ADELAIDE UNIVERSITY AUSTRALIA

Development of a Water Management Model for the Evaluation of Streamflow for Aquifer Storage and Recovery

Submitted by

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Table of Contents

List of Figures

List of Tables

List of Photographs

Abstract

Groundwater levels within the Willunga Basin, South Australia are declining due to excessive extraction of water for irrigation purposes. An alternative source of water is needed to supplement the declining levels to ensure the sustainability of the groundwater system. A model is developed to evaluate the potential for using aquifer storage and recovery in conjunction with the surface storage of streamflow as a possible alternative water source. The application of this model to the largest catchment in the Willunga Basin shows that sufficient streamflow is available to reverse the current overexploitation of the groundwater system.

Declaration

This work contains no material which has been accepted for the award of any other degree or diploma in any university or other tertiary institution and, to the best of my knowledge and belief, contains no material previously published or written by another person, except where due reference has been made in the test.

I give consent to this copy of my thesis, when deposited in the University Library, being available for loan and photocopying.

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Date. 2/4/2002..........

Acknowledgements

I would like to dedicate this thesis to my two sons, Chester Lawrence and Henley Edward. Without the birth of Chester, I would have not embarked upon this research. Henley through his many battles in the first year of his life has provided me with the determination to finish this research. I would like to thank my husband Scott for his continual support of my studies and chosen career.

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1 Introduction

Photo 1.0 'Pedler Divide', McLaren Vale South Australia

1.1 Introduction

The semi-arid climatic conditions that exist in many parts of the world coupled with the increasing demand for water supplies has induced stresses on surface and groundwater resources. Optimal water management practices are needed to meet the increasing demand for water, to alleviate the pressure on current supplies and to ensure the long-term sustainability of water resources.

The conjunctive use of surface and groundwater resources is critical to the development of rural and urban populations and water use industries (Watkins and Clark, 1997).

Sustainability of groundwater resources is dependent on groundwater recharge being equal to or exceeding groundwater extraction. Aquifer Storage and Recovery (ASR) using new sources of water such as treated stormwater and wastewater are methods that potentially may ensure the sustainability of the groundwater resources.

In this study, a computer simulation model is developed to evaluate the potential for ASR using streamflow. This is demonstrated for a case study area in South Australia. The model for ASR potential consists of three sub-models: a rainfall-runoff model, a surface storage model and a groundwater model. The ASR potential model is applied to Pedler Creek in the Willunga Basin, an important viticulture region in South Australia.

1.2 Study Objectives

The aims of this study are:

(a) To develop a general model that can be used to determine the quantity of streamflow that can be "captured" in a surface storage and made available for ASR; and

(b) To develop a better understanding of the interaction of streamflow, surface storage and aquifer storage in determining the performance of ASR schemes using streamflow.

A detailed data set is needed in order to determine the quantity of streamflow available for aquifer storage and recovery. This involves collecting and collating rainfall data from a number of locations, estimating a representative rainfall record for the whole catchment, determining the amount of runoff produced from this rainfall and hence the amount of water that could be diverted into an off-stream storage. A generic off-stream storage model is developed to provide an estimate of the actual quantity of water available for groundwater injection. Groundwater modelling has been used in conjunction with the developed surface storage model to ascertain the effect of pumping and extraction on the aquifer system.

1.3 Structure of Thesis

This thesis consists of three main components: an overview of artificial recharge, development of a general water management model and application of this model to a case study area.

Chapter 2 provides an introduction to artificial groundwater recharge and methods by which surface water is directed into the groundwater system. Artificial recharge is used in a number of countries throughout the world; this experience is discussed in Section 2.3 and is followed by a discussion of developments in artificial recharge in South Australia in Section 2.4. Prior to recharge, surface water often requires pretreatment. Various pretreatment methods are outlined in Section 2.5. An overview of possible constraints faced by operators of artificial recharge sites is introduced in Section 2.6.

The generic water management model developed in this study is described in Chapter 3 and includes discussion on modelling streamflow. The AWBM rainfall-runoff model development is outlined in Section 3.3 and the development of the surface storage model detailed in Section 3.4. Section 3.5 contains a brief discussion of how a groundwater model may be used to ascertain possible impacts of injected water on the groundwater system.

The water management model is applied to a case study area to determine the quantity of surface water available for artificial recharge into the groundwater system. Application and results of this model to the case study area are detailed in Chapter 4.

Concluding this thesis in Chapter 5 is a summary discussion of this research.

2 Artificial Groundwater Recharge

Photo 2.0 Injection and Extraction Well-heads, 'Pedler Divide', McLaren Vale, South Australia

2.1 Introduction

Artificial recharge is the deliberate redirection of surface waters into the groundwater system. It is defined as *"the purposeful redirection of excess surface waters into aquifers that provide storage for subsequent reuse"* (Pavelic and Dillon, 1997).

Artificial recharge of aquifers provides an important, cost effective water management tool by which alternative sources of water, e.g. surface water and recycled water, can be used to supplement and reduce demands on the groundwater system. Artificial recharge of aquifers may also reduce the impurities found in some surface waters by filtering the water as it passes through the porous media, removing a significant fraction of the suspended and colloidal load (Huisman and Olsthoorn, 1983). Recycled or reclaimed waters are a valuable resource and use of these waters via artificial recharge can reduce the volume of imported water required in a region. It can also reduce the environmental impacts associated with disposal of stormwater and wastewater to receiving ecosystems and reduce costs associated with water supply (Dillon et al., 1997). Surface water is a valuable water resource. Harvesting it during months when there is excess and using it for artificial recharge can provide a source of water during months when surface water availability is low and demand is high.

Factors influencing artificial recharge include the permeability of the aquifer into which the water is being recharged; size of the aquifer; availability, quality and quantity of recharge water; topography, surrounding land use and demand for recovered water. Aquifers targeted for recharge are predominately sedimentary or limestone and occasionally fractured rock. Aquifers with a high transmissivity are favoured for recharge as they can accept high rates of recharge and large volumes of water (Dillon and Pavelic, 1996).

The quality of the source water may affect artificial recharge. The relevant quality parameters include the quantity of organic matter present, suspended solids, colour and the quantity of nutrients present (Hatva, 1996). Source water may be natural waters from rivers and lakes or may be reclaimed water such as sewage effluent or urban stormwater.

Pretreatment of water prior to recharge is preferred as it reduces operational and environmental impacts. Pretreatment methods vary according to the type of source water, regulatory controls, and end use of the recovered waters. Methods range from advanced treatment such as tertiary treatment to wetlands to no treatment at all depending on the quality of source water.

Artificial recharge has an important function in water management and can be utilised in regions, which are highly dependent on the groundwater resource.

Recharge of the groundwater system is achieved by a number of different methods that are outlined in Section 2.2

2.2 Artificial Recharge Methods

Methods by which artificial recharge can occur include surface infiltration and injection wells (Pyne, 1995). The different methods depend on the hydrological characteristics of the aquifer and the availability and characteristics of the source water to be used for recharge (Pavelic and Dillon, 1997).

2.2.1 Infiltration Basins

For unconfined aquifers the most common method of recharge is surface infiltration via spreading basins or recharge trenches and channels. This involves the ponding of water over permeable soils. The infiltration basin method (Figure 2.1) is used for unconfined aquifers when the unsaturated zones are of suitable thickness, the topography is relatively flat and the transmissivity of the receiving aquifer is high enough to direct the water away from the ponding area (Fetter, 1994). This type of recharge method is often susceptible to high evaporation rates and requires an extensive area of land at reasonable cost (Gerges and Howles, 1996).

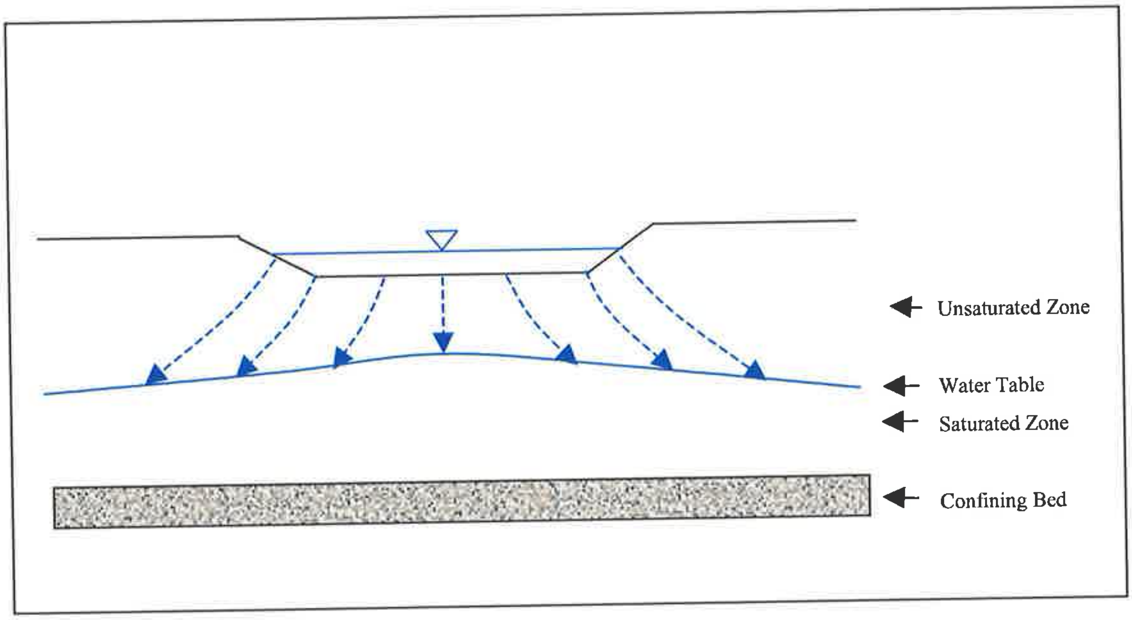


Figure 2.1 Artificial Recharge Using an Infiltration Basin

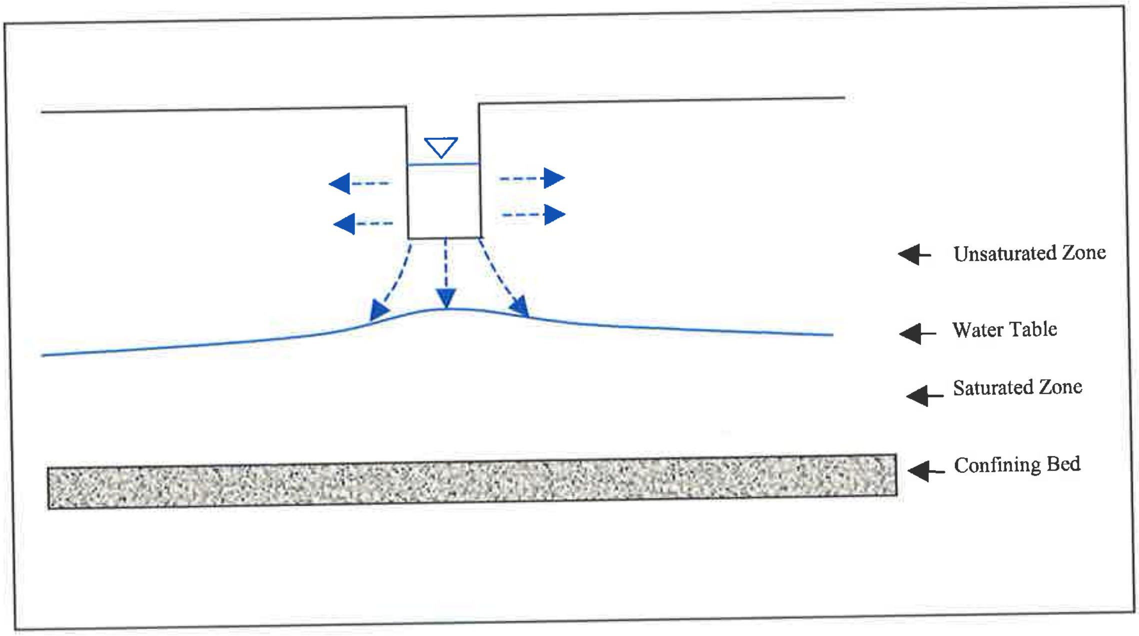


Figure 2.2 Artificial Recharge Using an Infiltration Trench

Recharge trenches (Figure 2.2) operate in a similar fashion to the spreading basins. This recharge method is used to recharge shallow unconfined aquifer systems. It requires less land than spreading basins and can be used in urban areas. However, foundation problems may arise if recharge trenches are located close to buildings (Gerges and Howles, 1996).

2.2.2 Recharge Wells / Aquifer Storage and Recovery

Artificial recharge via injection (Figure 2.3) is the preferred option for an aquifer that is semi or totally confined, or where the cost of land is high (Dillon and Pavelic, 1996). Injection wells may also be used to recharge unconfined aquifers by non-pressurised injection (Pavelic et al., 1992).

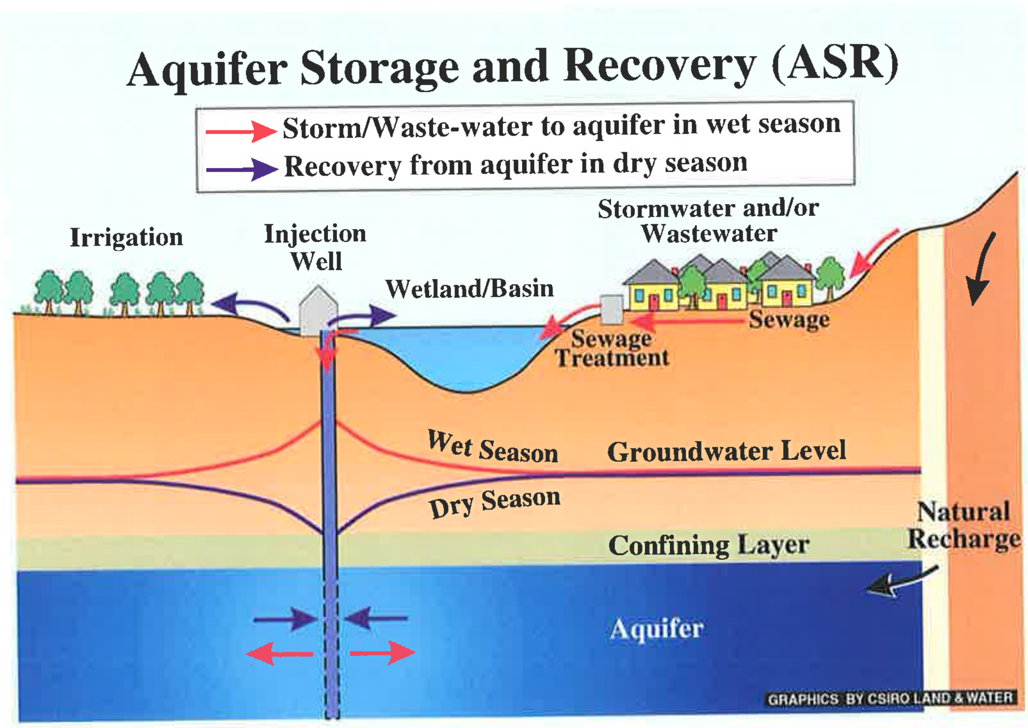


Figure 2.3 Schematic ASR Site (CSIRO, 2000)

Injection and extraction may occur from the same well or from separate wells. By utilising the same well, costs are reduced and the action of recharge followed by recovery may help to reduce clogging.

Pyne (1995) introduced the term aquifer storage and recovery (ASR), which he defined as *"the storage of water in a suitable aquifer through a well during times when water is available, and recovery of the water from the same well during times when it is needed."*

More generally ASR is considered to be the injection of water into an aquifer under pressure, either by a gravity head or a head maintained by an injection pump (Figure 2.4) (Fetter, 1994). The water is stored for a length of time and is recovered through one or more wells when desired.

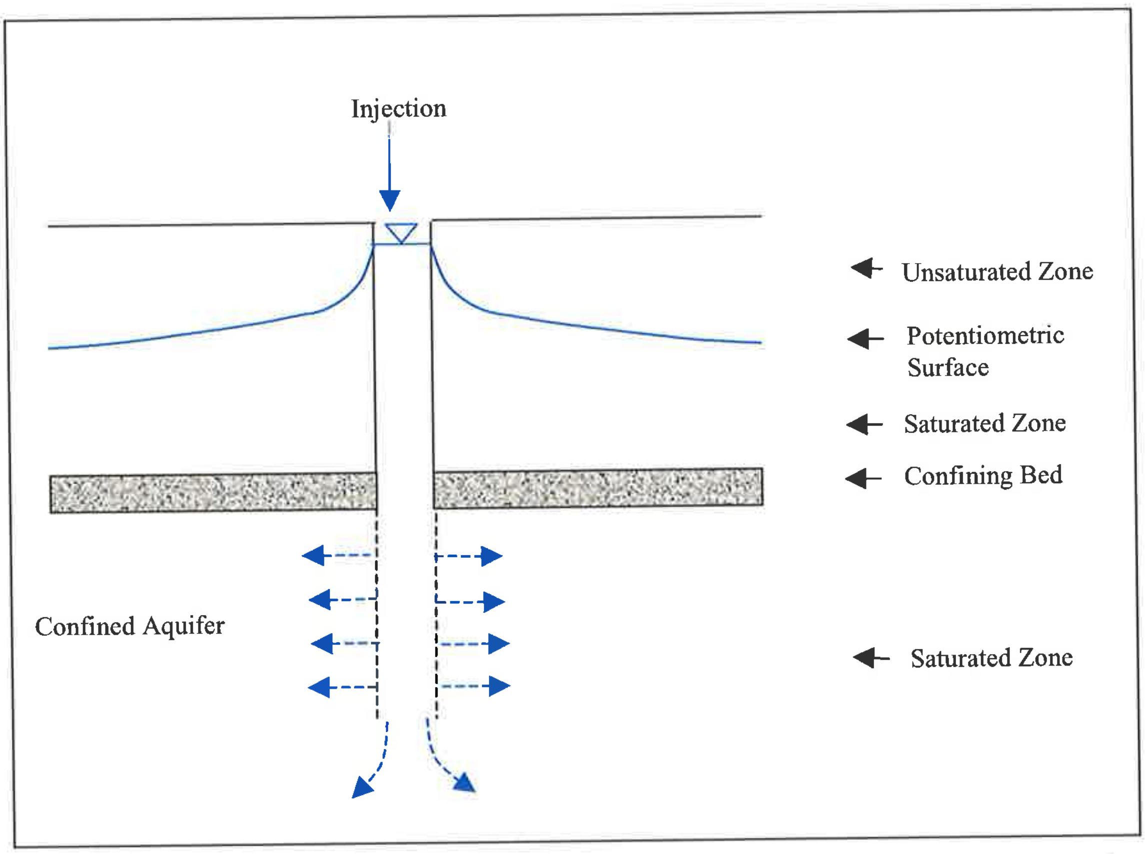


Figure 2.4 Artificial Recharge Using an Injection Well

Sites suitable for ASR are dependent on the hydrological characteristics of the aquifer and the availability and characteristics of the source water to be used for recharge

(Pavelic and Dillon, 1997). Identification of available water sources and key groundwater parameters are important to the identification of areas suitable for ASR. ASR has potential as a water management tool particularly in regions where groundwater is extensively utilised. ASR can supplement the natural recharge to the aquifer and recovery can be made easily via existing wells. The objectives of ASR include: supplementing groundwater supplies in times of shortage or high demand, reducing groundwater salinity and providing hydrological barriers to protect mixing of highly saline waters, e.g. seawater intrusion (Pavelic and Dillon, 1997), storing water underground to reduce evaporation and contamination, using underground storage in areas where topography is not suitable for surface storage, reducing the salinity of native groundwater and decreasing the outflow of stormwater and effluent to the marine environment (Gerges, 1996).

2.3 International ASR Development

Artificial groundwater recharge via injection wells has been used in a number of countries including USA, Israel, Canada, Kuwait, Netherlands, UK, Germany, Switzerland, Spain, India, Thailand and Australia (Pavelic and Dillon, 1997). In most cases it is an integral part of the water supply system to meet the demand for water from varying and limited supplies. Pavelic and Dillon (1997) provide a detailed summary of the international aquifer storage and recovery experience to date. There has been extensive international experience where the recharge waters have predominately consisted of natural waters, with more recent ASR sites using alternative sources of water including treated sewage effluent and urban stormwater.

Pavelic and Dillon (1997) reviewed international experience in ASR and identified 45 international cases of direct injection of ASR sites used for either research or operation. A summary of the primary purpose for artificial recharge at each of the 45 international sites is presented in Table 2.1.

Table 2.1 Primary Purpose for Artificial Recharge at 45 International Sites (Pavelic and Dillon, 1997)

Purpose	No. sites	%
Enhance groundwater supplies	20	45
Reuse reclaimed waters	5	11
Purify surface waters	5	11
Disposal of surface waters/flood control	4	9
Augment/supplement peak surface water demand	3	7
Control seawater intrusion	3	7
Protect surface water quality	2	4
Reduce groundwater salinity	1	2
Store thermal energy	1	2
Alleviate land subsidence	1	2

The primary purpose for artificial recharge to enhance groundwater supplies makes up 45% of these international cases. 11% of the sites were used to enhance or augment groundwater supplies with another 11% of the sites having, as their primary purpose, to purify surface waters.

Of the 45 international sites identified approximately 70% of the sites used river or lake water as the source for recharge, with the remaining 30% using sewage effluent or urban stormwater runoff (Pavelic and Dillon, 1997). Table 2.2 shows the types of pretreatment for the different types of source water at the international sites.

The main objective of pretreatment is to improve the quality of source water prior to injection. The level to which the source water is pretreated is dependent on a number of factors which include: the quality of the native groundwater, the end use of the recovered water, to minimise clogging and to prevent adverse biochemical reactions (Dillon and Pavelic, 1996).

Table 2.2 Pretreatment Methods for Varying Source Water (adapted from Pavelic and Dillon, 1997)

	Source Water % treated			
Types of pretreatment	**Urban Stormwater**	**River/Lake Water**	**Sewage Effluent**	**Groundwater**
Tertiary		31	56	20
Settling and Chlorination		28	11	
Filtration		18		20
Wetland	75	3		
Secondary and Chlorination		3	22	
No pretreatment	25	7		40
Unknown		10	11	20
TOTAL	**100**	**100**	**100**	**100**

2.4 ASR Development in South Australia

The use of aquifer storage and recovery is relatively new in Australia, although experience in other artificial recharge methods has been extensive (Dillon, et al., 1999).

South Australia has a very dry summer and cool winter. It is the driest state within Australia and relies heavily on a few reservoirs and the River Murray for most of its potable water supply. Research and development of artificial recharge using injection was first investigated in South Australia in the 1950's (Sibenaler, 2000). Since the first investigation, numerous experimental trials have been undertaken to assess the viability of storing and recovering recycled water or excess streamflow for potable and non-potable use.

Gerges et al. (1996) identifies a number of ASR objectives for South Australia:

- To develop ASR technology and identify where this technique can be used in South Australia.
- To improve local groundwater quality for irrigation and industrial use and reduce the reliance for imported water.
- To create low salinity lenses within saline aquifers for domestic water supply.

In areas where groundwater is of poor quality, ASR provides a means by which the groundwater resource can be utilised by artificially recharging with excess streamflow, urban stormwater or recycled effluent, which is of, or has been treated to a higher quality than the native groundwater. Using aquifers as a storage body for new sources of water, increases the benefits of water resource management by recycling more water and by reducing the amount of polluted water discharged to natural water bodies.

To date, the main source of raw water for ASR sites in South Australia is either urban stormwater or natural stream runoff, with pretreatment usually in the form of wetlands.

Artificial recharge via wells has taken place for over 100 years in Mt Gambier South Australia (Telfer and Emmett, 1994). Urban stormwater runoff for the area is recharged to the underlying limestone aquifer via 300 to 500 drainage wells to prevent flooding of the area. More recently it has been discovered that the limestone aquifer being recharged is hydraulically connected to the lake from which the town water supply is drawn. It is estimated that approximately 35% of the water received in the lake is from the recharge of urban stormwater (Telfer and Emmett,1994).

During the early 1950's, the SA Department of Mines investigated the potential for artificial recharge in the Adelaide metropolitan area when there was an excess of surface waters, i.e. when Adelaide's reservoirs overflowed. The initial findings were encouraging although the investigation ceased as it was considered at the time that reservoirs were a safer and more visible option (Sibenaler, 1996).

In 1974 an experimental artificial recharge site was established at Munno Para. The primary aim of the Department of Mines at this site was to gravity feed an existing well with river water to recharge the aquifer. Unfortunately the experiment folded due to lack of support and funding (Sibenaler, 1996).

Artificial recharge by ASR was attempted in the late 1970's far more successfully and was carried out in the Angas Bremer irrigation area (Figure 2.5). The area is a viable grape producing area and historically most of the water used for irrigation has been from groundwater and from floodwaters of the Angas and Bremer Rivers (Gerges et al, 1996). The main objective of this project was to reduce the overexploitation of groundwater and reduce rising groundwater salinity levels. Winter river flows were trapped in ponds adjacent to the Angas and Bremer Rivers and pumped into injection wells for recharge (Watkins and Clark, 1997). This provided the vineyards with improved quality irrigation water.

Scotch College (Figure 2.5) adjacent to Brownhill Creek injects creek water into a well during winter to increase the well production and lower the salinity of extracted water for irrigation on the school grounds during summer (Watkins and Clark, 1997). This operation has been ongoing since 1989.

More recently, Primary Industries and Resources SA (formally SA Mines and Energy) together with CSIRO and industry have conducted ASR trials in the Adelaide metropolitan area. One of the first ASR schemes to incorporate wetlands as a form of pretreatment and surface storage prior to recharge is located at the Paddocks site (Figure 2.5). Urban stormwater collected and treated in the wetland was injected into a number of wells at the Paddocks during winter and recovered during the summer months.

The suburban development of Andrews Farm located on the Northern Adelaide Plains site (Figure 2.5) was undertaken with the goal to recycle locally the stormwater produced from the peri-urban catchment consisting of residential areas and grazing farmland (Pavelic and Dillon, 1996). The urban stormwater is pretreated and stored temporarily in a wetland prior to injection. The trials indicate that the aquifer is

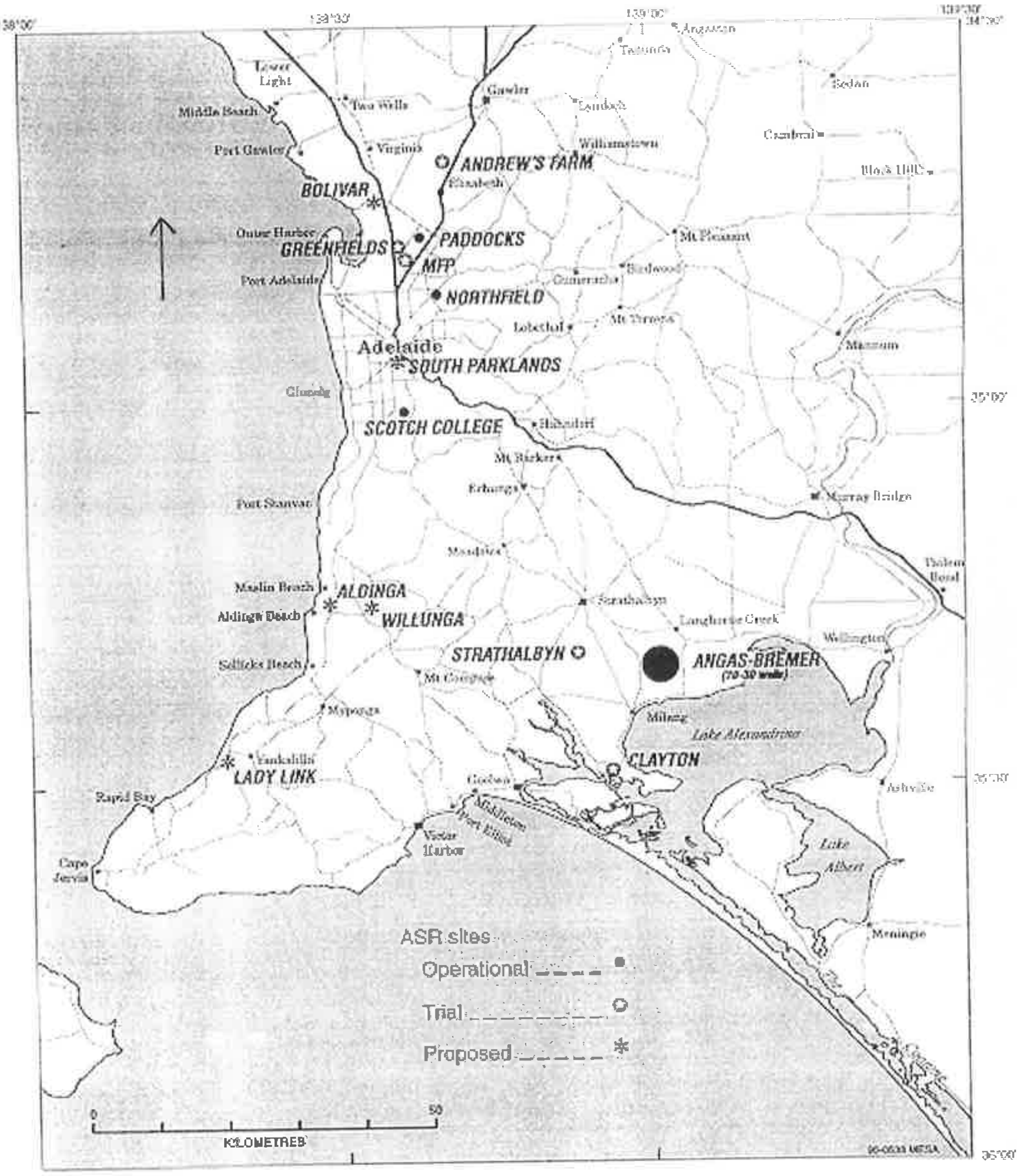


Figure 2.5 ASR Sites in South Australia (Sibenaler, 1996)

capable of storing the injected water and the salinity of the native groundwater is reduced to a quality suitable for irrigation (Mines and Energy SA, 1997).

An ASR site was commissioned at Regent Gardens, Northfield (Figure 2.5) in 1994 to dispose of urban stormwater generated by a housing development, which would have exceeded the capacity of the existing infrastructure (Sibenaler, 1996 and Emmet, et al., 1995). The water recovered during summer months is used to irrigate surrounding reserves and pumped back into the wetland to increase the level during summer.

The Greenfields site (Figure 2.5) was developed for the treatment of urban stormwater in wetlands followed by injection into an ASR site. The aquifer is recharged under gravity injection (Gerges et al., 1996).

The injection of water from Lake Alexandrina into a highly saline unconfined aquifer at Clayton (Figure 2.5) has provided the town with an emergency water supply. The main objective of this site is to store a backup supply of potable water in the event of toxic algal bloom outbreaks in the Lake during the summer months. Water is extracted from the Lake during winter when the algal bloom counts are the lowest and injected into the groundwater system. The aquifer also acts as a filter and purifies the lake water by removing some of the suspended solids.

The Bolivar trial site (Figure 2.5) is the first site in South Australia to test artificial recharge using treated effluent. The trial is testing the viability of treated effluent recharge to be recovered during months of high demand for irrigation of market gardens. The objective of the trial is to store the treated effluent in overexploited aquifers during the wet months and recover this water during the summer to meet the peak demand. An additional benefit of this system is that it may allow for expansion of the irrigated area (Dillon et al., 1999).

Table 2.3 summaries the ASR experience in South Australia and shows the level of pretreatment of source water prior to injection. Pretreatment options of source water are discussed further in Section 2.5.

Table 2.3 Summary of South Australian ASR Experience (adapted from Dillon et al, 1999)

Site/Year of Commencement	Source Water	Pretreatment
Mt Gambier – late 1800's	Urban Stormwater	None
Angas-Bremer Irrigation Area – mid 1970's	Streamflow	None
Scotch College Brown Hill Creek – 1989	Streamflow	None
Andrews Farm – 1993	Urban Stormwater	Wetland
Northfield – 1993	Urban Stormwater	Wetland
Greenfields – 1995	Urban Stormwater	Wetland
The Paddocks – 1995	Urban Stormwater	Wetland
Clayton – 1995	Lake Water	None
Bolivar – 1999	Effluent	Secondary disinfection

2.5 Pretreatment

Injection water is required to meet the ANZECC guidelines for irrigation (ANZECC, 2000) or be of no worse quality than the native groundwater. In order to achieve these objectives, pretreatment is generally required prior to injection of source water. The amount of pretreatment may range from a simple screen to passive treatment involving wetlands to advanced treatment with disinfection.

Wetlands are generally used to treat urban stormwater prior to injection. This form of treatment is used to remove contaminants by passive methods that are a function of the design, contaminant loading and residence time in the basin.

Treatment of effluent may be of preliminary, primary, secondary or tertiary standard. Preliminary treatment usually involves the removal of large floating objects, grit and sometimes grease. Racks and screens used in preliminary treatment remove any large suspended solids. This is usually followed by grit removal using gravity separators, which remove inert inorganic material, such as sand and metal fragments. Grease is removed by surface skimming devices or flotation processes.

Primary treatment is used to remove suspended solids; this usually involves fine screens followed by sedimentation using primary clarifiers. Addition of metallic coagulants and polymetric coagulants increases the removal of suspended solids in the primary clarifiers (McGee, 1991).

Secondary treatment processes remove soluble and organic matter from the wastewater by using biological processes. Biological treatment involves the addition of microorganisms, which remove soluble and colloidal organic matter from the wastewater. Biological systems can be separated into attached film growth and suspended growth processes. Attached growth techniques use a solid material on which bacterial solids concentrate. Types of attached film or surface growth processes include sand filters, trickling filters, rotating biological contactors (RBC), and fluidised beds (McGee, 1991). Clarifiers are utilised to remove large particles of bacterial slime. Suspended growth processes ensure sufficient bacterial population remains in suspension by using natural or mechanical mixers. This type of treatment includes activated sludge processes, oxidation ponds and sludge digesters.

Tertiary treatment may be used to treat effluent to a potable standard with the level required depending on the end use of the treated wastewater. Tertiary treatment aims to reduce the suspended solids, ammonia, organic nitrogen, total nitrogen, phosphorous and dissolved solids in the wastewater. The improvement in water quality is achieved by chemical coagulation, filtration and biological techniques (including nitrification and denitrification). Other advanced tertiary treatment techniques that may be used include ion exchange and reverse osmosis.

Metcalf and Eddy (1991) refer to disinfection as the *"selective destruction of disease causing organisms"*. Although not all organisms are destroyed by disinfection, the organisms of greatest concern to public health are bacteria, viruses, protozoa and amoebic cysts. Disinfection techniques aim to reduce the majority of these organisms in wastewater. Techniques include the use of chemical agents, physical agents, mechanical agents and radiation (Metcalf and Eddy, 1991). Chlorine is by far the most common form of chemical disinfectant. Physical agents used are heat and light (e.g. ultraviolet radiation). Four different mechanisms describe the action of a disinfectant: 1) damage to the cell wall; 2) alteration of cell permeability; 3) alteration of the colloidal nature of the protoplasm; and 4) inhibition of enzyme activity (Metcalf and Eddy, 1991).

The level of removal of organisms by disinfection is highly variable and depends on the technique employed. The virucidal resistance is generally higher than bacterial resistance and the resistivity of the organism to the disinfectant. Table 2.4 indicates the applicability of some disinfection techniques to wastewater.

Table 2.4 Comparison of disinfection techniques (adapted from McGee, 1991)

	Chlorination	**Ozone**	**Ultraviolet**
Level of treatment prior to disinfection	any	secondary	secondary
Complexity of technology	simple to moderate	complex	simple to moderate
Bactericidal	good performance	good	good
Virucidal	poor performance	good	good
Fish toxicity	toxic	none	nontoxic
Hazardous by products	yes	no	no
Persistent Residual	long	none	none
Contributes DO	no	yes	no
Increased dissolved solids	yes	no	no

Currently in the USA, disinfection together with advanced treatment of wastewater is necessary prior to injection of effluent into the groundwater system. Australia has developed guidelines for the use of effluent for injection and the types of treatments required. Depending on regulatory requirements and the end use of the recovered water, advance treatment and disinfection will achicve a high level of quality improvement for recharge water through a combination of physical, chemical and/or biological processes although the cost may be high.

2.6 Operational Constraints

One of the major constraints in the operation of injection wells used for aquifer storage and recovery is clogging or plugging. Artificial recharge of groundwater may result in increases in head near the well which is referred to as clogging (Pyne, 1995). The increase in the head in the well may result in a reduction in recharge efficiency due to clogging in the gravel pack, the screen wall and/or the area surrounding the well wall. Gerges (1996) identified the following physical, chemical and biological processes that may contribute to clogging:

- Microbial growth – dependent on the presence of carbon and nutrients in the source water. The end product is an impermeable slime that is deposited at or near the screen.
- Air entrapment and gas binding – when air bubbles become trapped within the aquifer pore spaces inhibiting water movement.
- Suspended solids – particles can physically block pore spaces in the filter media, which may lead to cake filtration, and then deteriorate into cake filtration with compression. These processes take varying amounts of time to develop, which contributes to varying rates of clogging.
- Geochemical precipitation – reactions that are a function of the quality of the injection water, native groundwater, aquifer mineralogy, pressure, temperature and redox potential may result in reduced velocity of recharge or recovered water.
- Clay swelling – swelling and spreading of montmorillonite in a clay aquifer may result in clogging. This is a common type of clogging and occurs by cation exchange between ions in solution and those present in the clay aquifer.
- Mobilisation of aquifer fines and particle rearrangement – occurs by the repeated reversal of direction due to injection and extraction.

If clogging is severe, redevelopment of the well may be necessary. The primary objective of redevelopment is to restore the well to the original hydraulic condition of the aquifer. Redevelopment techniques are either mechanical or chemical in nature. Mechanical methods include airlifting, pumping (at a rate higher than the recharge rate), or sectional flush pumping from tubes located within the gravel pack. Chemical methods include addition of acids, flocculants, disinfection and oxidising agents (Dillon and Pavelic, 1996).

Redevelopment may be undertaken daily or less frequently depending on the type of aquifer, clogging process and severity in reduction of the recharge rate. Table 2.5 provides a summary of clogging processes and redevelopment techniques.

Table 2.5 Summary of Clogging and Redevelopment Processes (Dillon and Pavelic, 1996)

Clogging	Minimisation	Redevelopment
Filtration of suspended solids	Minimise suspended solids	Pumping, surging, jetting
Microbial growth	Minimise organic matter, disinfection	Pumping, surging, jetting plus disinfection and acids
Chemical precipitation	Recharge water compatible with groundwater	pH changes
Clay swelling and dispersion	Use low clay aquifers	Add flocculants
Air entrapment	Avoid cascading, positioning of intake, high pressure feed	Pumping, surging, jetting
Gas binding	Prevent denitrification in porous media by disinfection, nitrate removal	Pumping, surging, jetting

3 Water Management Model

Photo 3.0 Off Stream Storage, 'Pedler Divide', McLaren Vale, South Australia

3.1 Introduction

The Water Management Model (WMM) developed in this study is a general model that aims to determine the quantity of water that is available for aquifer recharge for a given site using a particular water source. Currently the model considers natural streamflows as the water source, but it could easily be used in conjunction with urban stormwater or treated effluent.

The WMM consists of three submodels: a rainfall-runoff model, a surface storage model and a groundwater model (Figure 3.1). The WMM attempts to integrate the effects of natural streamflow, dam storage and groundwater storage.

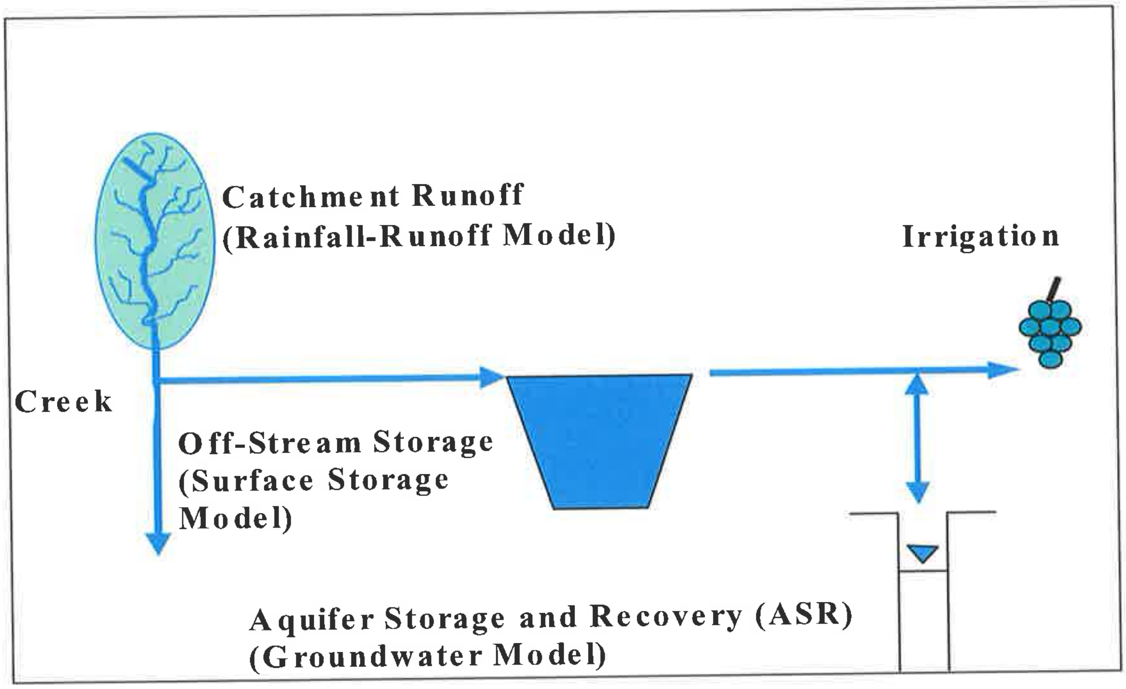


Figure 3.1 Schematic Diagram of the Water Management Model

3.2 Modelling of Streamflow

The modelling of streamflow is necessary in an ungauged catchment where streamflow data is not available. Where the catchment is gauged, and a long record of daily streamflow is available, the historical records can be used directly in the surface storage model. For an ungauged catchment, the first stage in the development of the Water Management Model involves the estimation of streamflow in the creek. In developing a model for runoff for a catchment, an understanding of the physical and hydrological characteristics of the catchment and their interactions is paramount in the model development. In the absence of streamflow data, the approach taken has been to use an established rainfall-runoff model to generate runoff for the study area.

The conversion of precipitation to streamflow as part of the hydrological cycle is illustrated in Figure 3.2, which demonstrates general flow of water from a watershed to a stream. The major inputs into streamflow are surface runoff or overland flow, interflow or subsurface flow and baseflow. Rainfall reaching the ground may collect to form runoff or infiltrate into the ground (Shaw, 1991) and may travel along subsurface pathways. Surface runoff (sometimes referred to as overland flow) can be defined as rain that drains across the land into a stream or channel (Fetter, 1994). Horton (1940) first introduced the concept of overland flow, which can be defined as flow which occurs during excess rainfall events; the infiltration capacity of the soil is exceeded and excess rainfall flows over the surface to a stream or lake. This type of flow is not commonly observed over the whole catchment and generally occurs in areas where the infiltration capacity is less than the rainfall intensity. Commonly this occurs once the soil is saturated, along stream channels which may be saturated by subsurface flow and in areas where soils have a low storage capacity (Singh, 1995). A large fraction of the surface runoff may be produced by a small section of the catchment. Interflow or subsurface flow is the flow which results from the rainfall infiltrating into the soil and draining into the river through the unsaturated zone.

Baseflow can be defined as the drainage from shallow unconfined aquifers (Boughton, 1993b) and the amount of discharge is dependent on the depth of the saturated zone and the hydraulic gradient towards the stream.

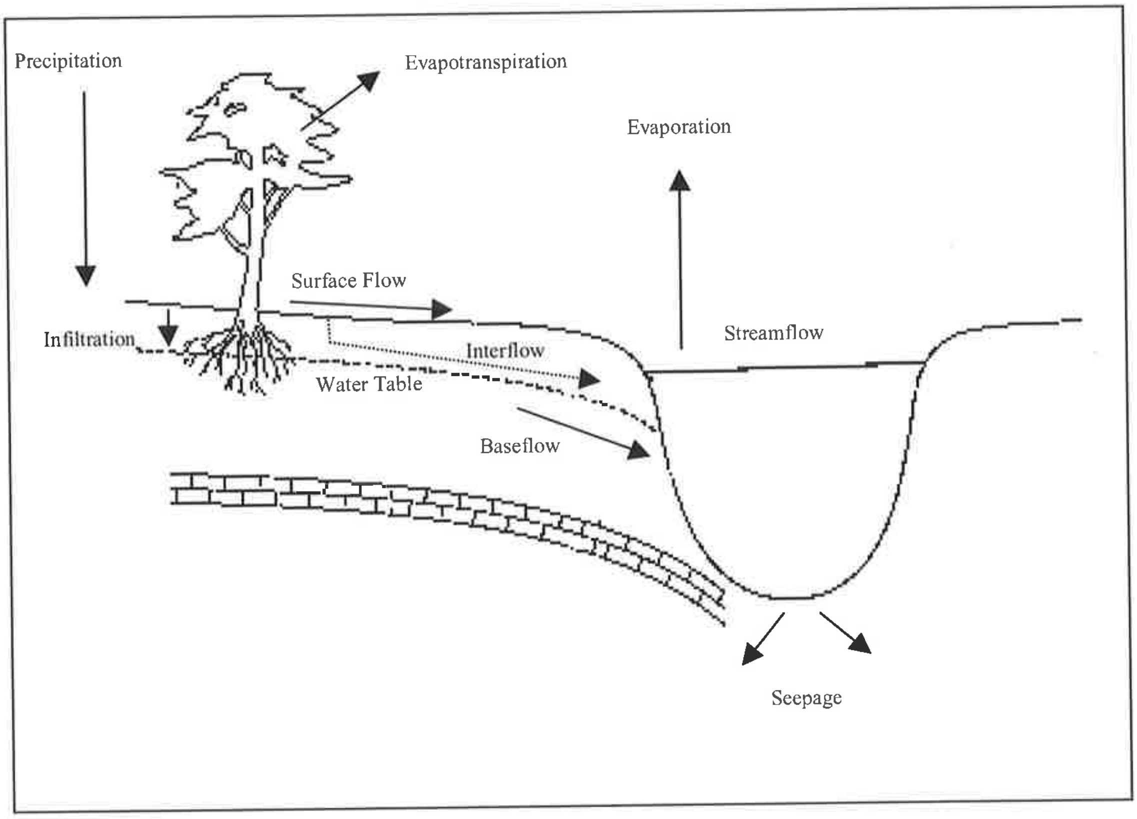


Figure 3.2 The Hydrological Cycle (adapted from Singh, 1995)

For ungauged catchments, the absence of streamflow data necessitates the need for using an established rainfall-runoff modelling technique. Numerous rainfall-runoff models have been developed to derive relationships between rainfall and streamflow for both gauged and ungauged catchments. The three main modelling approaches used (Chiew et al., 1993) are black box, process-based and conceptual models.

Black Box models use empirical equations to relate runoff and rainfall, so only the input and output have physical meaning (Chiew et al., 1993) and other catchment processes are not included. Examples of black box models include models with simple mathematical equations and time series methods such as the Tsykin equation (Tsykin, 1985) and IHACRES (Jakeman et al., 1990).

Process models simulate the hydrological processes in a catchment and use many partial differential equations for physical processes (Chiew et al., 1993). Examples of process models include the Institute of Hydrology Distributed model (IHDM, Bevan et al., 1987) and the Systeme Hydrologique Europeen (SHE) model (Abbott et al., 1986). The SHE model is an example of a physically based process model developed jointly in the UK, France and Denmark (Shaw, 1991). The SHE model attempts to incorporate the 3-dimensional processes of surface and subsurface flow into a general purpose catchment model. SHE uses finite difference methods to obtain solutions of non-linear flow equations. Submodels within SHE account for snowmelt, canopy interception, evapotranspiration, overland and channel flow, unsaturated and saturated subsurface flow (Boughton, 1988). Disadvantages of such process models include the numerous parameters required for their development, computer run time, data limitations and the application of theoretical equations describing laboratory scaled systems to real catchments (Chiew et al., 1993).

Conceptual models provide a simplistic representation of the hydrology of a catchment which can be treated as a series of interconnected storages which are described mathematically. The storages are considered to act as reservoirs and a water balance is performed on these. Examples of simple conceptual models include the SFB model and the AWBM model (Boughton, 1984, 1993, 1996). The AWBM model, developed in Australia, works on a daily timestep and attempts to model the important catchment processes. The model has been successfully applied in both gauged and ungauged catchments in Australia and can simulate runoff from gauged catchments with an accuracy equal to more complex models (Boughton, 1993) with parameters in the model directly evaluated from streamflow data sets without the need for trial and error optimisation. An example of a complex conceptual model is MODHYDROLOG (Chiew et al., 1993). MODHYDROLOG has 17 effective parameters and a runoff routine which routes the flow to the catchment discharge point (Chiew, 1993). Due to its numerous parameters, it can take considerable time and effort to calibrate MODHYDROLOG.

A relatively simple model is preferred to be used on an ungauged catchment and when more data is available it may be better to use a more complex model for calibration of model parameters (Boughton, 1988). The AWBM model has been used to generate runoff data for the ungauged catchment case study used in this project. The generation of runoff is described in Section 3.3. The model was selected for this project based on its successful use in ungauged catchments within Australia, its simplicity and the fact that it only requires rainfall, evaporation and basic soil data.

3.3 AWBM Model

The AWBM model (Boughton, 1993a, 1996) was developed for estimating water yield in gauged and ungauged catchments. The model simulates losses and runoff from partial areas within a catchment at either hourly or daily time intervals (Boughton and Hill, 1997). The capacity of the catchment to absorb some of the rainfall prior to producing runoff is modelled by three soil moisture stores (or "buckets") which allow for spatial variability within the catchment (Boughton, 1993b). The three buckets each represent a partial area of the catchment. Runoff is generated from the model when the storage capacity of one or more of the three soil moisture stores is exceeded. A water balance is performed independently over each of the buckets using a daily or hourly time-step. The structure of the model is shown in Figure 3.3. In this figure C1, C2, C3 represent the capacities of each of the three storage buckets and A1, A2, A3 (fractions) represent the partial areas of the catchment associated with each store. SS represents the surface storage and BS represents the baseflow storage.

The water balance at each time step involves the addition of rainfall to each of the soil moisture stores and the subtraction of evapotranspiration from each of the stores. If moisture in any storage bucket exceeds the capacity of the bucket, runoff results. The following water balance equation is applied to each bucket independently (Boughton, 1996).

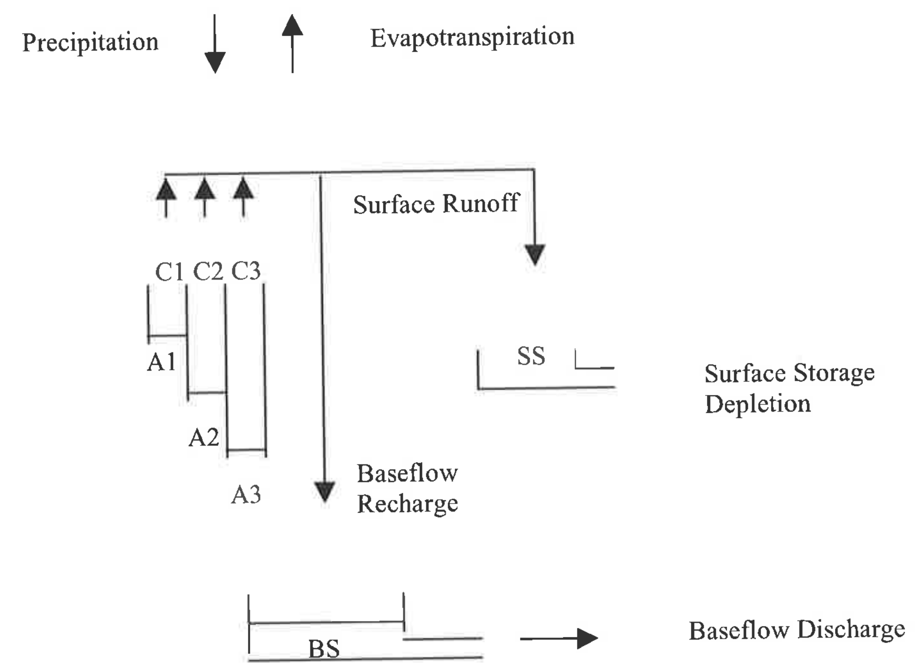
$$\text{store}_n^{t+1} = \text{store}_n^{t} + \text{rainfall}^{t} - \text{evaporation}^{t} \quad (n = 1, 2, 3), \quad (3.1)$$

where $store_n^t$ is the storage capacity of bucket n at time t.

If $store_n^{t=1} > capacity_n$,

then $runoff^{t+1} = store_n^{t+1} - capacity_n$

and $store_n^{t+1} = capacity_n$.



Key:

C1 = capacity of surface store 1, A1 = partial area associated with catchment C1

C2 = capacity of surface store 2, A2 = partial area associated with catchment C2

C3 = capacity of surface store 3, A3 = partial area associated with catchment C3

SS = surface storage and BS = baseflow storage

Figure 3.3 Structure of the AWBM Model (Boughton, 1996),

Runoff produced from the model is easily converted into volume for each partial area by multiplying runoff depth by the fraction represented by the partial area and multiplying by the total catchment area. The runoff from one or more of the surface storage buckets is partitioned into surface runoff and baseflow recharge using the baseflow index (BFI) which is the ratio of baseflow to total flow in the streamflow. The surface component of runoff is determined by the factor (1 – BFI) x Runoff.

Baseflow is modelled by a single moisture store which has two controlling parameters (Boughton, 1993b). Therefore, Baseflow Recharge is determined as follows:

$$\text{Baseflow Recharge} = \text{BFI} \times \text{Runoff.} \tag{3.2}$$

The baseflow recession constant controls the amount of discharge from the baseflow storage to the stream and the baseflow storage is depleted as:

$$\text{Baseflow Storage} = (1\text{-K}) \times \text{BS}, \tag{3.3}$$

where K is the baseflow recession constant for the timestep being used and BS is the current volume in the baseflow storage (Boughton, 1996). Similarly, the surface storage is depleted at the rate of (1-KS)*SS, where KS is the surface runoff recession constant for the timestep being used and SS is the current volume in the surface storage (Boughton, 1996).

The baseflow storage parameter (BS) represents the amount of water present in the baseflow store. The model assumes the partitioning of the runoff is constant for all events, although Sharifi and Boyd (1994) suggest that the fraction going into baseflow may not be constant and, in fact, more water goes into baseflow during small runoff events and less during large runoff events.

The capacity of the surface storage buckets is determined by an equivalent average soil storage capacity (SSC) which is then separated into capacities of the three storage buckets. These values may be calibrated if adequate streamflow data exists. The AWBM model has built-in default values for the proportions of the surface storage capacity and for the partial areas. The default values for the bucket capacities are defined in Table 3.1.

Table 3.1 Default Values for the AWBM Model, (C1 = capacity of surface store 1, C2 = capacity of surface store 2, C3 = capacity of surface store 3, A1 = partial area associated with the catchment of C1, A2 = partial area associated with the catchment of C2, A3 = partial area associated with the catchment of C3, SSC = average surface storage capacity)

Bucket Capacity	Partial Area
C1 = 0.5 * average SSC	A1 = 0.2
C2 = 0.75 * average SSC	A2 = 0.4
C3 = 1.5 * average SSC	A3 = 0.4

C1 corresponds to A1, i.e. the smallest bucket is set to represent 20% of the catchment area. Only two partial area parameters need to be defined with the third being a function of the others, i.e. A1 = 1.0 – A2 – A3 (since A1 + A2 + A3 = 1).

Surface runoff may be delayed by routing the runoff through a surface store. This is necessary in cases where the model is required to simulate a delay in surface runoff reaching the outlet of a medium to large catchment (Boughton, 1996).

The surface storage parameters are similar to the parameters used in the predecessor to the AWBM model, the SFB model (Boughton and Carroll, 1993). The difference between the AWBM model and the SFB model is that AWBM allows for spatial variability of the surface store capacity by using 3 stores of different sizes and the SFB does not allow for spatial variability. Another difference is between the baseflow parameters: the AWBM baseflow parameters are directly related to characteristics of the streamflow hydrograph and the SFB base flow parameters are determined by a mathematical derivation.

On a small ungauged rural catchment with no baseflow, the AWBM model can be used as a single parameter model (Boughton, 1993, 1989). The average surface storage capacity is the only AWBM parameter required as the program separates this value into the three default capacities and three default fractions of the catchment area.

Setting the baseflow index to zero and the daily recession constant to 1.0 nullifies the baseflow (Boughton, 1993, 1989).

The average surface storage parameter is very similar to the surface storage parameter S of the SFB model. Testing of the SFB model is well documented and can be used to estimated the average surface storage capacity of the AWBM model (Boughton, 1993). The SFB model was calibrated on 106 rural catchments in south eastern Australia with varying catchment areas by Nathan and McMahon (1990) and Figure 3.4 shows the calibrated vales of the SFB parameter S. The histogram shows that the median value of S based on rural Australian catchments is approximately 120mm. The AWBM model is preferred over the SFB model by practitioners as it can be used to estimate parameters directly from the recorded data (Sharifi and Boyd, 1994).

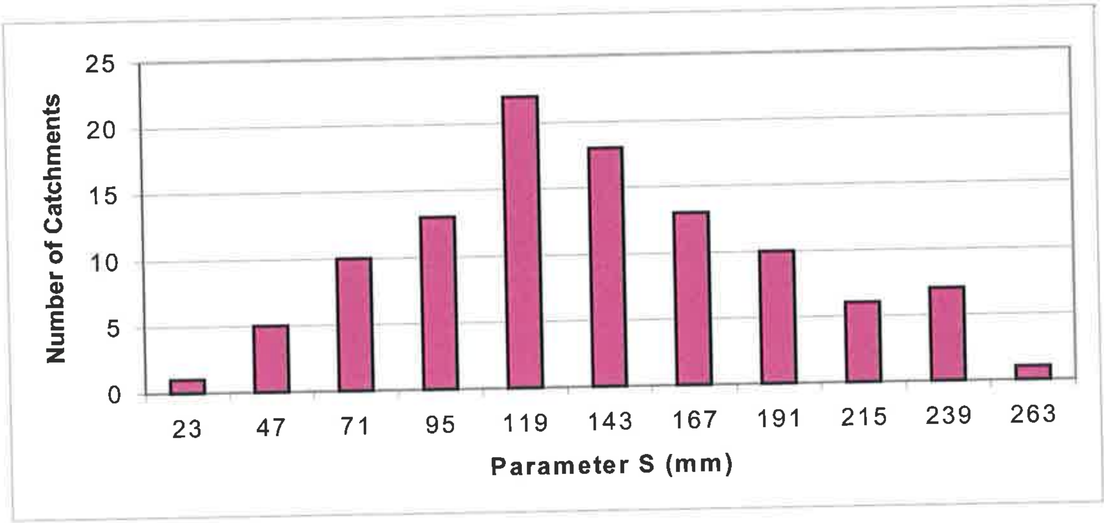


Figure 3.4 Histogram of Calibrated S Values of the SFB Model (Boughton, 1991; Nathan and McMahon, 1990)

3.3.1 Calibration of the AWBM Model

The parameters used in the AWBM model can be directly evaluated when daily streamflow data exists without the need for trial and error optimisation (Boughton, 1993a, 1993b). The calibration technique is based on the methods of hydrograph analysis and has been incorporated into the AWBM program. For surface runoff only, the purpose of the calibration is to determine the surface storage capacities and partial areas, whose combined excess closely resembles the actual runoff values (Boughton, 1996). The multiple linear relationship for the actual runoff is given by equation 3.4

$$\text{Actual } RO_j = e_{1,j}\, A_1 + e_{2,j}\, A_2 + e_{3,j}\, A_3, \qquad j=1,\ldots,12 \tag{3.4}$$

where the left hand side of the equation represents the actual runoff in the jth month, $e_{n,j}$ is the calculated runoff from store n for the jth month and A_n is the partial area of the catchment represented by capacity C_n (Boughton, 1996). The capacity of the smallest store C_1 is determined by trial and error testing a range of capacities until a surface runoff occurrence corresponds with the actual runoff occurrence (neglecting the volume). Once C_1 is determined to give the most accurate match to the actual runoff events, capacities C_2 and C_3 are determined by multiple linear regressions with the partial areas A_2 and A_3 being the regression coefficients.

Calibration involving baseflow and surface runoff is more complex as it is difficult to determine when rainfall excess is generated from the smallest store (Boughton, 1996). Details of calibration of the AWBM model are given in Boughton (1993b, 1996, 1990).

The AWBM model can be used with any number of parameters between one and nine, depending on the catchment type, catchment size and data availability. The model can be used as a 1- parameter model for a simple ungauged catchment with no baseflow to a 9- parameter model, which includes 2 surface routing parameters. Boughton (1996) illustrates the flexibility of the AWBM model.

- 1 - parameter model: using a single average value for surface storage capacity with the default disaggregation into 3 capacities and partial areas. BFI is set to zero for no baseflow store and KS is set to zero for no surface routing. This can be used on ungauged catchments with no baseflow.

- 3- parameter model: using a single average value for surface storage capacity with the default preset disaggregation into 3 capacities and partial areas; 2 baseflow parameters, BFI and K; and setting KS to zero (no surface routing store). Used on ungauged catchments with baseflow.

- 4- parameter model: using a single average value for surface storage capacity with the default preset disaggregation into 3 capacities and partial areas; 2 baseflow parameters, BFI and K; and using a daily surface routing parameter KS. This can be used on ungauged catchments with baseflow.

- 5- parameter model: used on small gauged catchments with no baseflow where runoff exists and the surface storage capacities and partial areas can be directly calibrated using a calibration program within the model.

- 7 - parameter model: this is incorporated into the original version of the AWBM model (Boughton, 1993) without surface routing. The parameters consist of 3 surface storage capacities, 2 partial area parameters and 2 baseflow parameters. The parameters are calibrated using a calibration program within the model.

- 8 - parameter model: includes a daily surface routing parameter as an addition to the original version of the AWBM model (Boughton, 1993). The parameters are calibrated using a calibration program within the model.

- 9 - parameter model: includes 2 surface routing parameters as an addition to the original version of the AWBM model (Boughton, 1993). This version of the model is used for a continuous hydrograph calculation using rainfall and runoff data of hourly time intervals.

3.4 Storage Model

A Surface Storage model is developed for the daily simulation for the off-stream storage. The storage dam is designed to provide a temporary surface storage system prior to irrigation or injection into the groundwater system.

Figure 3.5 shows the inputs required for the surface storage water balance. These include seepage of groundwater into (and out of) the dam, precipitation into the dam, streamflow diverted into the storage and other water diverted into the storage. Outputs include evaporation from the storage, spillage and water pumped out of the storage.

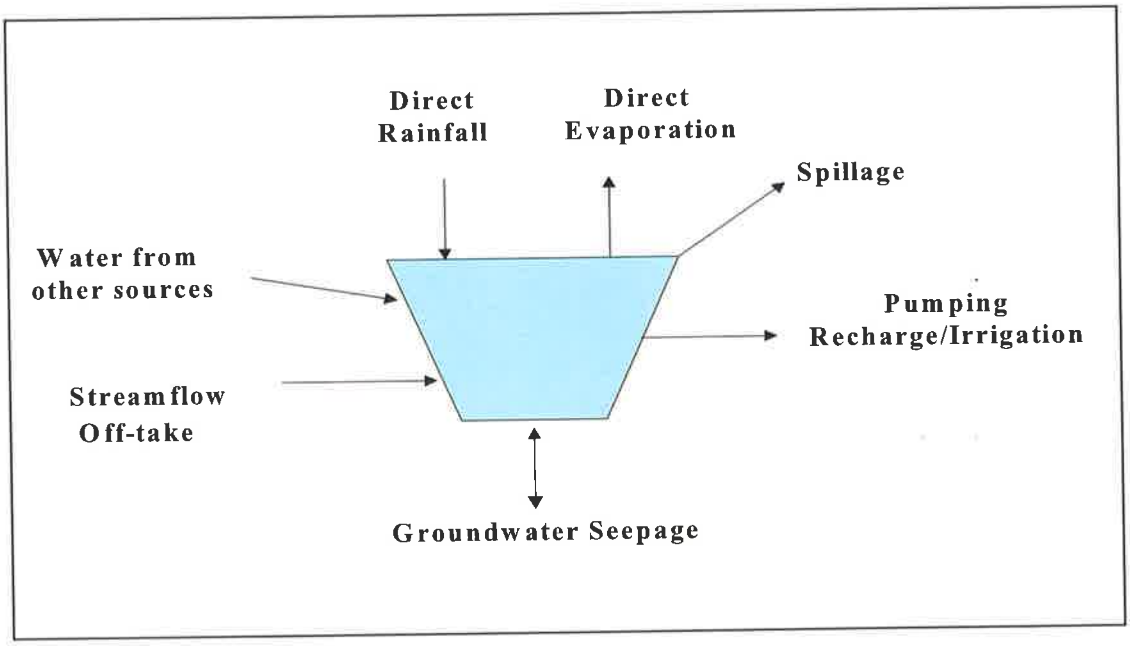


Figure 3.5 Surface Storage Model

A water balance is performed on a daily time step on the surface storage to determine the amount of water available in the storage on a daily basis for recharge or irrigation.

Stream water is diverted to an off-stream store by a diversion structure. In general, an off-take diversion structure allows water to pass from the stream into the storage dam. Commonly the diversion structure takes the form of a weir or a pipeline at a preset height above the streambed.

In order to determine the diversion into an off-stream storage, a hydrograph is needed to allow for short-term variations in streamflow. For gauged catchments where hydrographs have been measured, data can be used directly and the volume of water diverted to the off-stream storage calculated. In the absence of streamflow data, a synthetic hydrograph is required. This enables flow rates to be determined at various time increments and the associated diversion volumes to be calculated.

3.4.1 Estimation of Stream Depth

In order to estimate the volume of water passing into the storage via the diversion structure, the depth of water in the stream is required. For gauged catchments the depth of flow can be determined by the use of empirical formulae such as Manning's Equation together with field measurements of cross sectional area, bed slope and estimates of boundary roughness (Pilgrim, 1987).

To calculate the volume of water entering the diversion structure in an ungauged catchment, an estimate of the streamflow is required. From Section 3.3, the AWBM model estimates the daily runoff depth produced from the total catchment. For the purposes of this model the catchment area of the stream is estimated to the point of the off-take diversion structure.

The daily simulation of the AWBM model produces a daily runoff value in mm (Figure 3.6). The daily volume produced from a runoff event is calculated using the following equation:

$$\text{Total Volume (m}^3\text{)} = \text{AWBM runoff (mm)} \times \text{Catchment Area (km}^2\text{)} \times 1000 \qquad (3.5)$$

The variation in flow rate over the given day can be approximated by assuming a triangular synthetic hydrograph with constant rainfall intensity over the storm duration (McCuen, 1998). The area under the hydrograph represents the total daily volume as calculated using Equation 3.5.

As an approximation, the base of the isosceles triangle is taken to equal twice the time of concentration (McCuen, 1998), where the time of concentration (Tc), is the time required for rain falling at the farthest point of the catchment to flow to the measuring point of the river (Shaw, 1991). In this case, the measuring point of the creek is the point on the creek where the diversion channel starts.

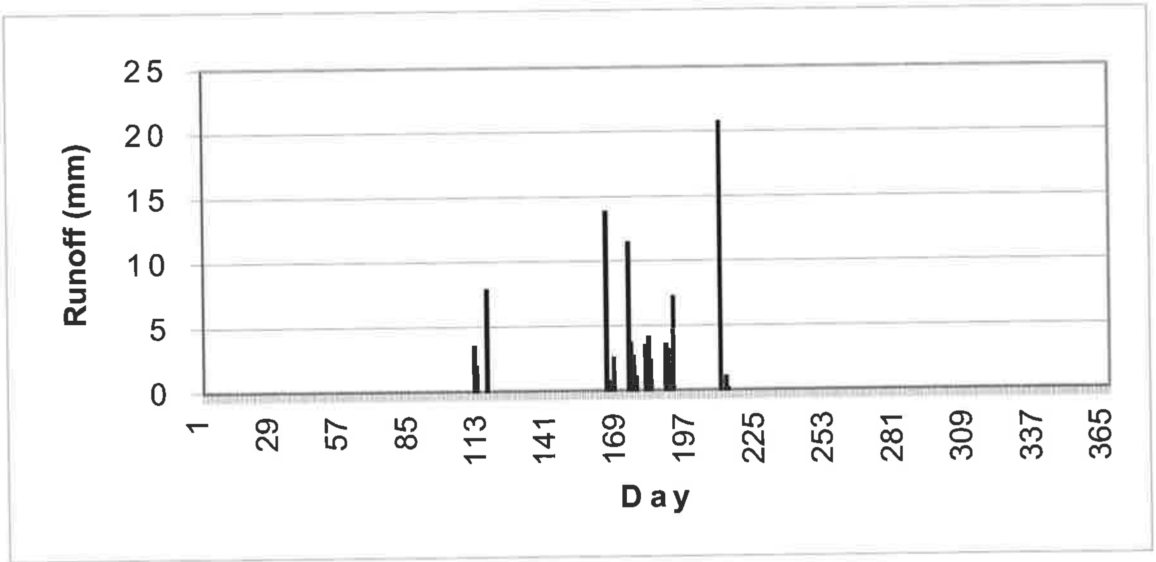


Figure 3.6 Example of Generated Runoff from the AWBM Model

For South Australia the time of concentration is estimated to be (Pilgrim, 1987):

$$Tc(\text{hour}) = 0.5\, a^{(2/3)}, \tag{3.6}$$

where:

a = catchment area (km^2).

From Figure 3.7, the peak of the triangular hydrograph occurs at the time of concentration.

Knowing the time of concentration and the total daily volume, the maximum flowrate of the triangular hydrograph can be estimated using the following equation:

Total Daily Volume (m^3) = Tc (hours) x $Q_{(max)}$(m^3/s) x 3600sec/hour (3.7)

To approximate the discharge volume that passes through the diversion per unit time interval, the hydrograph is divided into suitable time increments.

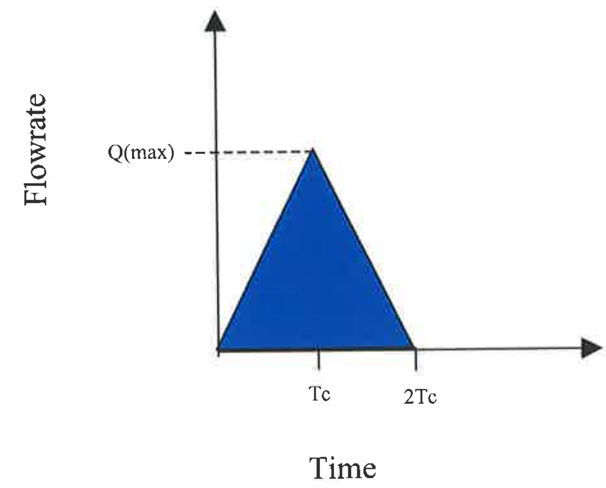


Figure 3.7 Triangular Hydrograph

By assuming the main stream channel is trapezoidal in cross-section (Figure 3.8), the depth for each flowrate in the stream can be determined using Manning's Equation.

Manning's Equation: $Q = \frac{1}{n} A R^{\frac{2}{3}} S^{\frac{1}{2}}$, (3.9)

where:

Q = discharge (m^3/s)

A = cross sectional area (m^2) = $(b + zy)y$

R = hydraulic radius (m) = $((b + zy)y)/(b + 2y\sqrt{(1 + z^2)})$

S = channel longitudinal slope

n = Manning's Coefficient

b = channel bottom width (m)

y = depth of flow (m)

$\frac{1}{z}$ = side slope

By applying Manning's equation, the stream discharge is expressed as a polynomial function of the depth of flow in the stream. The depth of water in the stream can be determined from the known discharge by solving this equation using the mathematical secant method (Kreyszig, 1988).

Once the depth of water is determined, the height of water in the diversion channel can be calculated, so that for a given time increment the volume of water flowing into the surface storage can be determined.

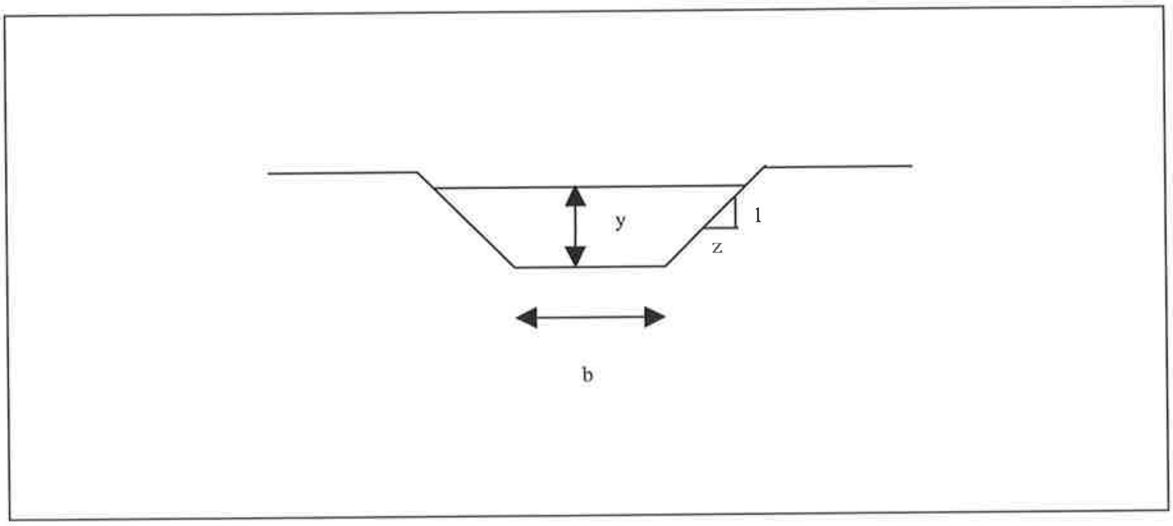


Figure 3.8 Trapezoidal Channel Cross Section, where b is the channel width, y is the depth of flow and 1/z is the side slope of the channel

3.4.2 Flow in Off-take Diversion Structure

The off-take diversion structure allows water to pass from the stream into the off-stream storage. The off-take structure most commonly takes the form of an open channel or pipeline at a preset height above the streambed. Determination of the flow in the open channel structure used in this model is discussed in Section 3.4.2.1 and alternative diversion structures are outlined in Section 3.4.2.2.

3.4.2.1 Flow in an Open Channel Off-take Diversion Structure

The depth of water passing into the open channel diversion is estimated for each of the time increments used in the triangular hydrograph generation for an ungauged stream or actual hydrograph for a gauged stream. From Figure 3.9, y is the depth of flow in the channel for each time increment. Subtracting the height of the diversion structure, P, from the depth of water in the channel, the depth of water flowing in the diversion channel (h) for each time increment can be determined.

For the purpose of this surface storage model, the open channel diversion structure is assumed to act as a rectangular broad crested weir. French (1994) defines a broad-crested weir as *a structure with a horizontal crest above which the fluid pressure may be considered hydrostatic.* To satisfy the definition of a broad crested weir, the following inequality must be met (French, 1994):

$$0.08 \leq h/L \leq 0.50, \tag{3.10}$$

where:

h = height of water passing over the weir,

L = width of the weir.

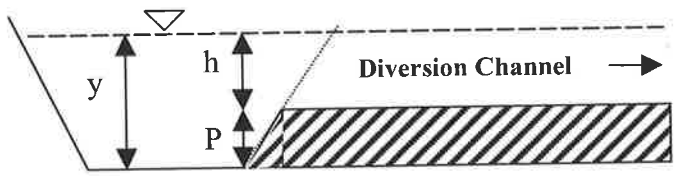


Figure 3.9 Cross-sectional View of Channel and Weir Diversion Structure (P is the height of the diversion channel, h is the depth of water flowing in the diversion and y is the depth of water in the channel)

The ratio h/L must be greater than or equal to 0.08 for the energy losses over the weir crest to be neglected. In addition h/L must be less than or equal to 0.50 for the assumption that the streamlines over the weir block are horizontal and the pressure is hydrostatic (French, 1994).

French (1994) gives the theoretical discharge over a broad crested weir as:

$$Q = 2/3C_DC_V\,(2/3g)^{1/2}Lh^{3/2} \qquad (3.11)$$

where:
C_D = discharge coefficient,
C_V = velocity coefficient,
g = acceleration due to gravity (m/s^2).

The broad crested weir supports the nape so that the pressure variation is hydrostatic over the weir. By applying Bernoulli's equation up stream of the weir and on the weir and neglecting the approach velocity, the following equation is obtained (Streeter and Wylie, 1981):

$$Q = CLh^{3/2}\,, \tag{3.12}$$

where:

$C = 2.0$.

Hence the diversion, Q, can be determined.

Using 15 minute time-steps, the triangular hydrograph can be discretised and the height of water in the diversion channel calculated based on the assumption that the diversion channel acts as a broad crested weir and the weir inequality is satisfied.

3.4.2.2 Alternative Diversion Structures

The current model uses an open channel diversion structure; alternative diversion structures may take the form of a diversion pipe or a mid-channel diversion structure. These are briefly discussed below.

For incompressible flow through an off-take diversion pipe, the flow rate is determined using Bernoulli's equation and the Darcy-Weisbach formula.

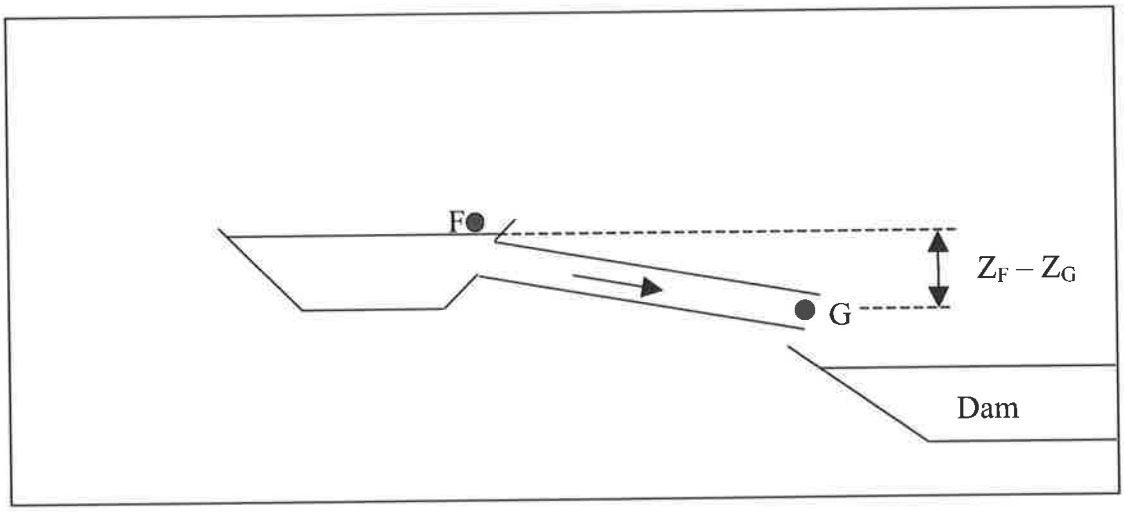


Figure 3.10 Pipe Diversion Structure (FG is the pipe length, Z_F is the elevation at F and Z_G is the elevation at G).

From Figure 3.10, the diversion from a stream through a pipe of diameter d and of length FG which discharges to the atmosphere satisfies the equation:

Total Energy at F = Total Energy at G + friction loss

The loss due to friction in the pipe FG is given by the Darcy-Weisbach formula:

$$h_f = fl/d\ (\bar{v}^2/2g). \qquad (3.13)$$

Neglecting entry and exit losses, the discharge through a pipe diversion structure can be estimated from:

$$Z_F - Z_G = \frac{\bar{v}^2}{2g} + \frac{fl}{d}), \qquad (3.14)$$

where

Z_F = Elevation at F (m),

Z_G = Elevation at G (m),

$\bar{v}$ = mean velocity in the pipe (m/s),

f = friction factor,

l = length of pipe (m),

d = diameter of pipe (m).

Hence from the mean velocity and the pipe's cross-sectional area, the diversion flowrate Q can be estimated.

An alternative vertical structure that partitions the flow into two, a diversion flow and a bypass flow as shown in Figure 3.11, may be used (Dandy, 2000).

The vertical structure may be positioned in a suitable location across the channel to allow a predetermined fraction of the streamflow to pass into the storage. The diversion flowrate in this case is simply the predetermined fraction of the streamflow.

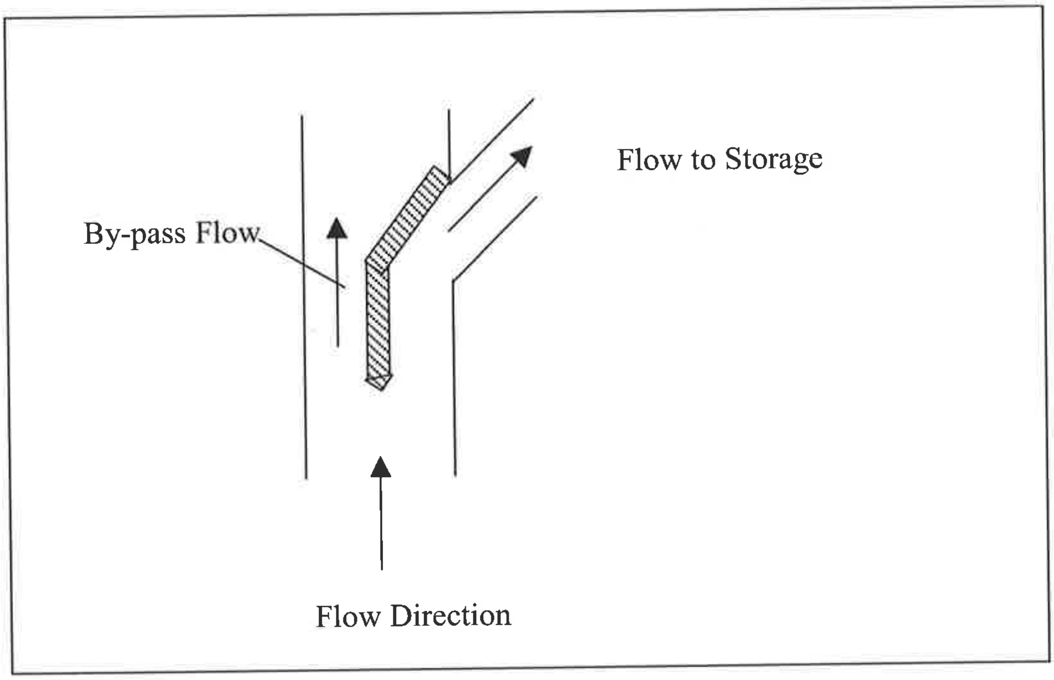


Figure 3.11 Vertical Partition in Stream

3.4.3 Surface Storage

The storage is assumed to be rectangular in plan, with known side slope, top length, width and maximum depth. The bottom dimensions of the storage can be derived from the known top dimensions and side slope. The maximum surface area of the storage is estimated using the bottom dimensions of the storage and the maximum depth and side slopes (Figure 3.12).

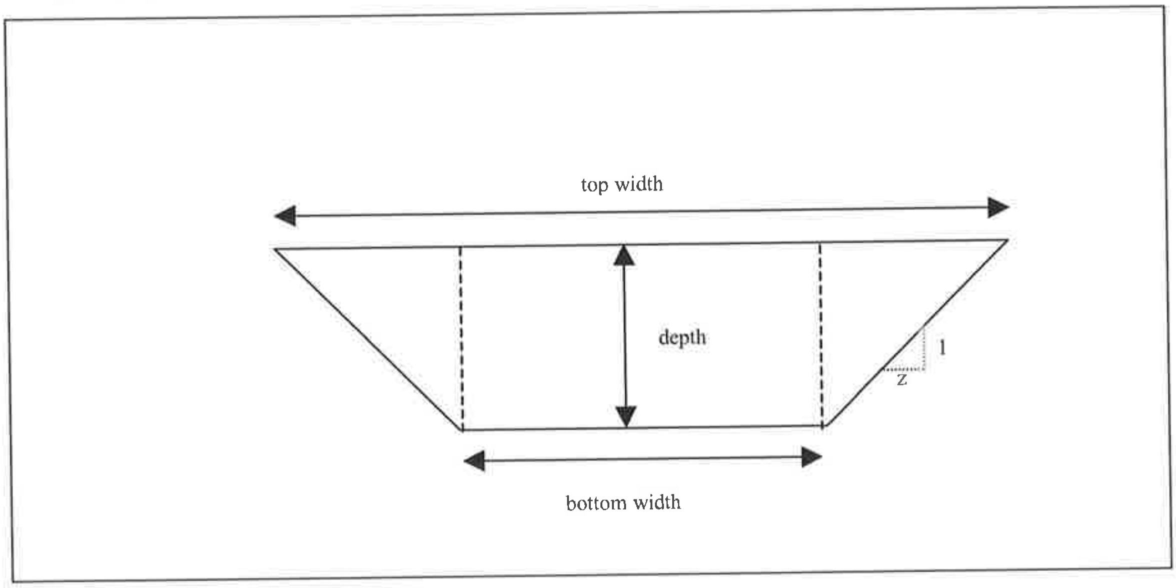


Figure 3.12 Storage Cross-section with side slope 1/z

The volume contributed by rain falling directly onto the storage is calculated by multiplying the maximum surface area of the storage by the daily rainfall recorded. This is added to the water balance of the storage on a daily time step.

The volume of water removed from the storage by evaporation is a function of the actual surface area of the storage. As the depth of water in the storage varies, the surface area will vary as a function of depth. The depth may be determined for a known volume using Newton's Method (as an alternative to the secant method used in Section 3.4.1)(Kreyszig, 1988). Once the variable depth is derived, the surface area can be calculated and daily evaporation loss from the storage determined. The surface area multiplied by the daily depth of evaporation determines the volumetric evaporation loss.

The model accounts for other water entering the storage. This may include mains water pumped in, recovered water, groundwater seepage or water transferred from other storages.

3.5 Groundwater Model

Potentially available surface water can be injected into the groundwater system, to provide a temporary storage that can be utilised at a later time. A groundwater model is used in conjunction with the rainfall-runoff model and the surface storage model to simulate the injection process and predict behavior of the injected water in the sub-surface environment. The daily volume available for artificial recharge is determined from the surface storage model, and is input into the groundwater model. The groundwater model used is a numerical model and for comparison an analytical solution is also used. A discussion of the theoretical assumptions behind the numerical model and the analytical solution follows with the application of this theory to the study area presented in section 4.9.

The theoretical considerations required to simulate aquifer head distribution near injection wells and extraction wells are equivalent (Fetter, 1994). During the pumping of a well, the head in the aquifer is drawn down, whereas during injection the head in the aquifer increases (Figure 3.13). Therefore, the same mathematical equations are used for injection and extraction, with a negative value given for the pumping rate of an injection well (Fetter, 1994).

The governing equation for modelling non-steady state groundwater flow is:

$$\frac{\partial^2 \phi}{\partial x^2} + \frac{\partial^2 \phi}{\partial y^2} = \frac{S}{T}\frac{\partial \phi}{\partial t} - \frac{Q}{T} \tag{3.15}$$

where T is transmissivity (m^2/day)

ϕ is the groundwater head (m),

S is the storage coefficient,

t is the time (day),

Q is inflow or outflow (m/day).

To solve the above equation, the groundwater model incorporates a numerical method in order to obtain an approximate solution. The groundwater model solves a set of algebraic equations which are derived by approximating the partial differential equation 3.15 together with boundary conditions and initial conditions. The approximating technique used in this groundwater model is the finite difference method.

When simplified equation 3.15 can be solved analytically. A simplified form of Equation 3.15 in polar coordinates for the effect of a single pumping well on a homogeneous confined aquifer with radial symmetry is:

$$\frac{\partial^2 \phi}{\partial r^2} + \frac{1}{r}\frac{\partial \phi}{\partial r} = \frac{S}{T}\frac{\partial \phi}{\partial t}. \tag{3.16}$$

An analytical solution to Equation 3.16 is provided by the Theis Equation which describes the drawdown effect in a confined aquifer (Fetter, 1994):

$$\phi_0 - \phi_h = \frac{Q_p}{4\pi T}\int_u^\infty \frac{e^{-u}}{u}du, \tag{3.17}$$

where the argument u is given by:

$$u = \frac{r^2 S}{4Tt}, \tag{3.18}$$

where

Q_p = constant pumping rate (m3/day),

ϕ_h = hydraulic head after pumping(m),

ϕ_0 = initial hydraulic head (m),

$\phi_0 - \phi_h$ = drawdown (m),

T = aquifer transmissivity (m^2/day),

t = time since pumping began (days),

r = radial distance from the centre of the pumping well (m),

The integral in equation 3.17 is defined as the well function W(u). Using the well function notation, the Theis equation is expressed as (Fetter, 1994):

$$\phi_0 - \phi_h = \frac{Q_p}{4\pi T}W(u). \tag{3.16}$$

The hydraulic parameters of an aquifer are determined by means of aquifer tests. In an aquifer test a well is pumped and the rate of decline of the water level in nearby observation wells is recorded. The hydraulic properties of the aquifer are determined from the time-drawdown data (Fetter, 1994).

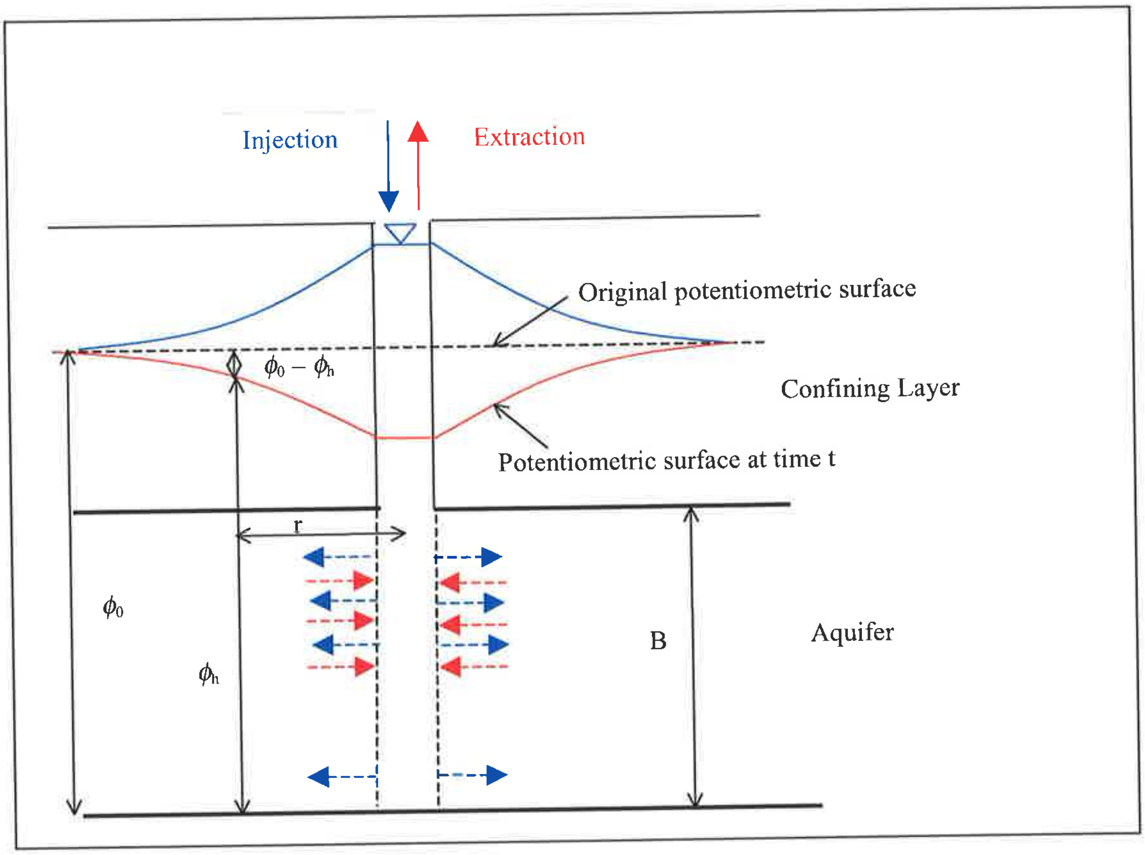


Figure 3.13 Well Pumping/Injection in a Confined Aquifer, where B is the aquifer thickness, r is the radial distance from the pumping well, ϕ_0 is the initial hydraulic head and ϕ_h is the hydraulic head after pumping

The above equations assume the aquifer is homogeneous and isotropic, including sedimentary or "porous media" aquifers. However, the Pedler Divide injection site is contained within the basement aquifer, which is a fractured crystalline rock. Groundwater flow through porous media aquifers occurs between individual mineral grains and assemblages, whereas flow through fractured rock aquifers occurs primarily through fractures and joints which occupy only a small proportion of the entire aquifer. Therefore, application of the above equations for radial flow around wells in porous media may not be appropriate in a fractured rock environment unless the fractured rock is assumed to act as a porous media on a regional scale (Harrington, 2001). For this analysis, the assumption of the fractured rock aquifer acting as a porous medium is made.

Simulation of aquifer head distribution around an injection well, and migration of injected water in the sub-surface environment can also be estimated with the aid of numerical groundwater flow models. There are a number of computer codes commercially available for modelling groundwater flow in porous media, including MODFLOW (McDonald and Harbaugh, 1988) and PLASM (Prickett and Lonnquist, 1971) which could easily be integrated with the developed surface storage model for prediction or system interpretation. There also exist a limited number of codes for simulating groundwater flow in fractured rock environments, but these all require careful assignment of discrete fracture properties (eg. FRAC3DVS, Therrien and Sudicky, 1996).

Rasser (2000) has developed a 3-dimensional regional groundwater model of the Willunga Basin. Application of this model and an analytical solution to the case study area is discussed briefly in Section 4.8.3.

4 Willunga Basin

Photo 4.0 Vineyards at Pedler Divide, McLaren Vale

4.1 Introduction

The Willunga Basin is located approximately 35 km south of Adelaide (Figure 4.1) and covers an area of 26 000 hectares. The basin comprises a variety of rural and urban land-uses and is best known for its viticulture and almonds.

Groundwater levels within the Willunga Basin are declining due to excessive extraction of water for irrigation purposes (PIRSA and OCWMB, 1998). An alternative source of water is needed to supplement the declining levels to ensure the sustainability of the groundwater system and the local wine industry. The water management model developed is used to evaluate the potential of using aquifer storage and recovery (ASR) in conjunction with surface storage of streamflow as a possible alternative water source.

4.2 Hydrology

The boundaries of the Willunga Basin watershed are delineated to the north by the watershed of the Onkaparinga River, to the south and east by the Sellicks Hill Range and to the west by the coastline of Gulf St Vincent (Newman, 1994, Figure 4.2). Five ephemeral streams are located within the basin: Pedler Creek, Maslin Creek, Port Willunga Creek, Washpool Drain and Sellicks Drain (Figure 4.2). The creeks are ephemeral and only flow for short durations after heavy rainfall. The creeks rise in the Sellicks Hill Range and flow down the escarpment westward to Gulf St Vincent. A number of steep-sided escarpment ephemeral streams flow onto the plain and terminate within a short distance of the range in alluvial outwash fans (Bowering, 1979).

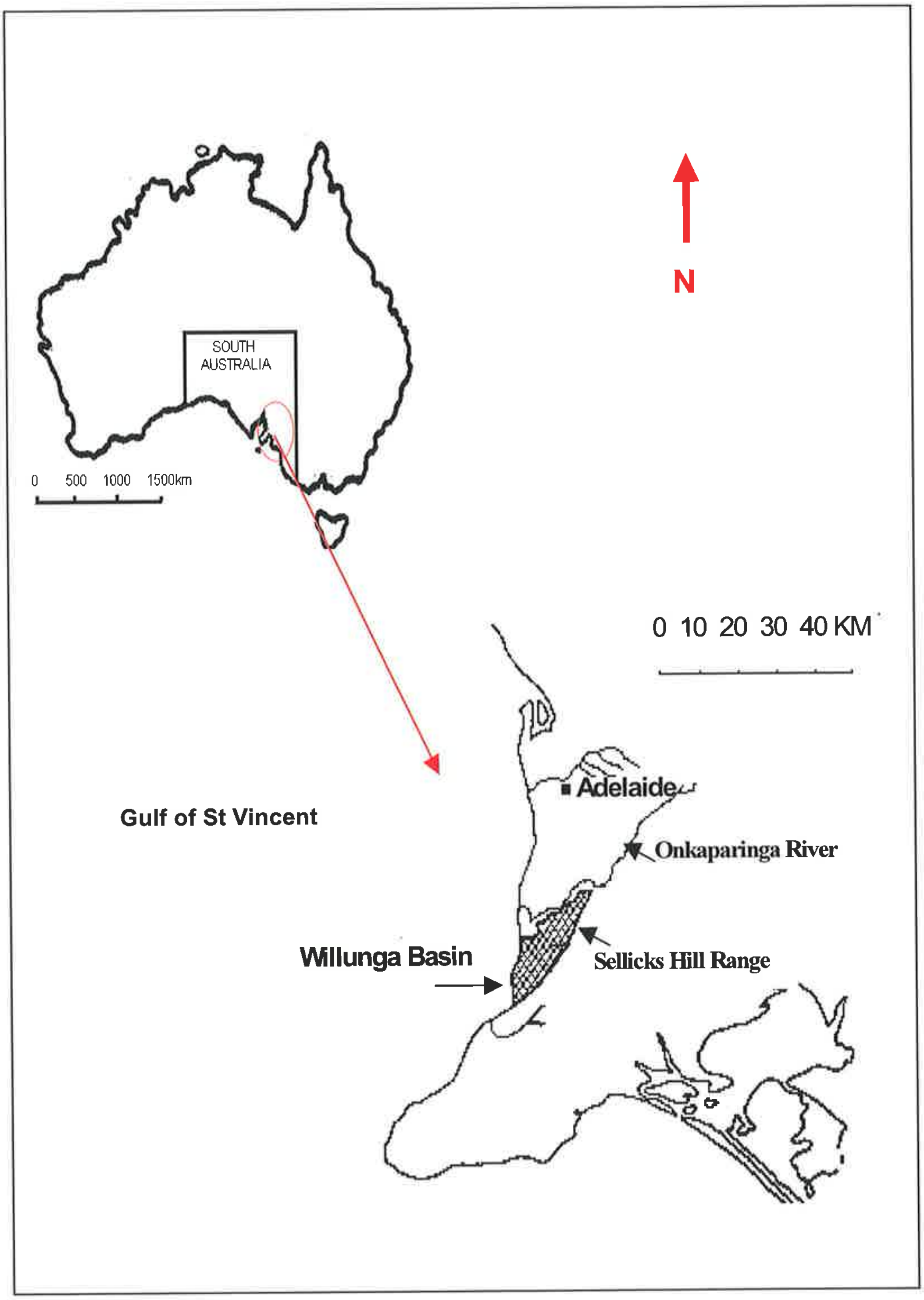


Figure 4.1 Location of Willunga Basin (adapted from Cresswell, 1994)

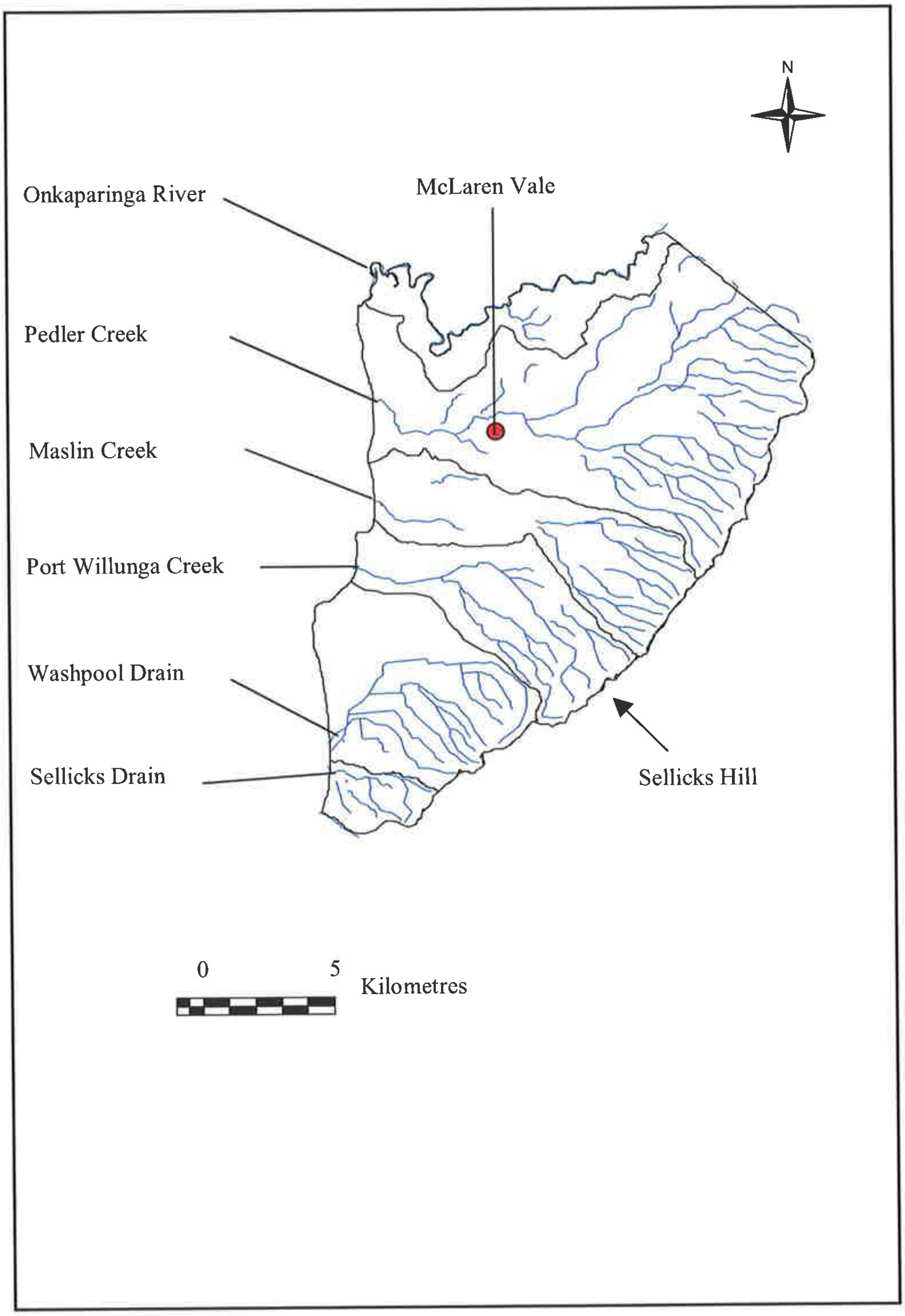
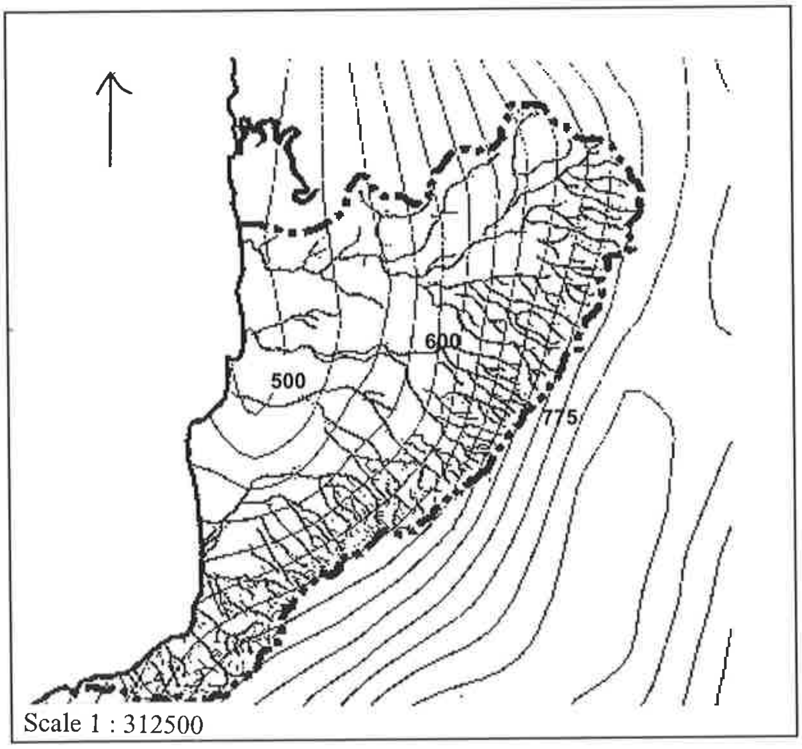


Figure 4.2 Catchments within the Willunga Basin

The climate of the basin is typically Mediterranean, with warm dry summers and cool moist winters (Bowering, 1979). Temperatures range from an average daily minimum of 9.4°C to an average daily maximum of 27.9°C in January with the corresponding values in July being 2.1°C and 14.9°C (respectively) (Overton, 1993). Average annual rainfall ranges from 500mm near the coast to over 800 mm in the foothills to the east, most of which occurs during winter (Figure 4.3). Summer rainfall is low and during this time the demand for water for irrigation is high. The summer period is generally hot and dry with evaporation exceeding 8mm/day. Prolonged dry periods in excess of 2 months often occur in summer, with the occasional thunderstorm activity during January and February producing some summer rain (Newman, 1994).

Figure 4.3 Rainfall Isohyet Map for Willunga Basin (Cresswell, 1994)



4.3 Hydrogeology

The Willunga Basin incorporates both a hydrogeologic basin and a boundary of surface catchments based on the watershed for the area. Although both the watershed and the geologic basin overlap, they do not coincide perfectly (Newman, 1994). In this study, the term Willunga Basin is used to refer to the watershed area.

The Willunga geologic basin is part of the much larger St. Vincent Basin formation commenced in the early Tertiary period where the uplift of the Willunga Fault resulted in a topographic depression bounding the basin to the south and east (Cresswell, 1994). The basin is a thin vertical wedge comprising mid to late Tertiary and Quaternary sediments deposited on the western or downside of the Willunga Fault (Newman, 1994, Figure 4.4). The sedimentary deposits are thickest to the south and west and taper towards the north (Cresswell, 1994). The whole sedimentary sequence dips toward the south-east (Watkins and Telfer, 1995). A number of aquifers exist within the Willunga geologic basin. These are the Port Willunga Formation, the Maslin Sands aquifer and the Basement aquifer. The Blanche Point Formation aquitard separates the Port Willunga Formation and the Maslin Sands aquifer (Watkins and Telfer, 1995, Figure 4.5). The Basement aquifer lies beneath the Maslin Sands aquifer. The Port Willunga Formation is separated by an overlying clay layer deposited in the Quaternary period, the Quaternary aquitard (Cresswell, 1994). Groundwater flows from the north-east corner of the geologic formation toward the coast as depicted in Figure 4.5. All the major aquifers outcrop at the surface. This provides a natural avenue for recharge to the aquifers by streamflow and rainfall.

The estimated sustainable yield from groundwater for the Willunga Basin is 5700 ML/a, while the current average usage is estimated at 7380 ML/a (PIRSA, 1998).

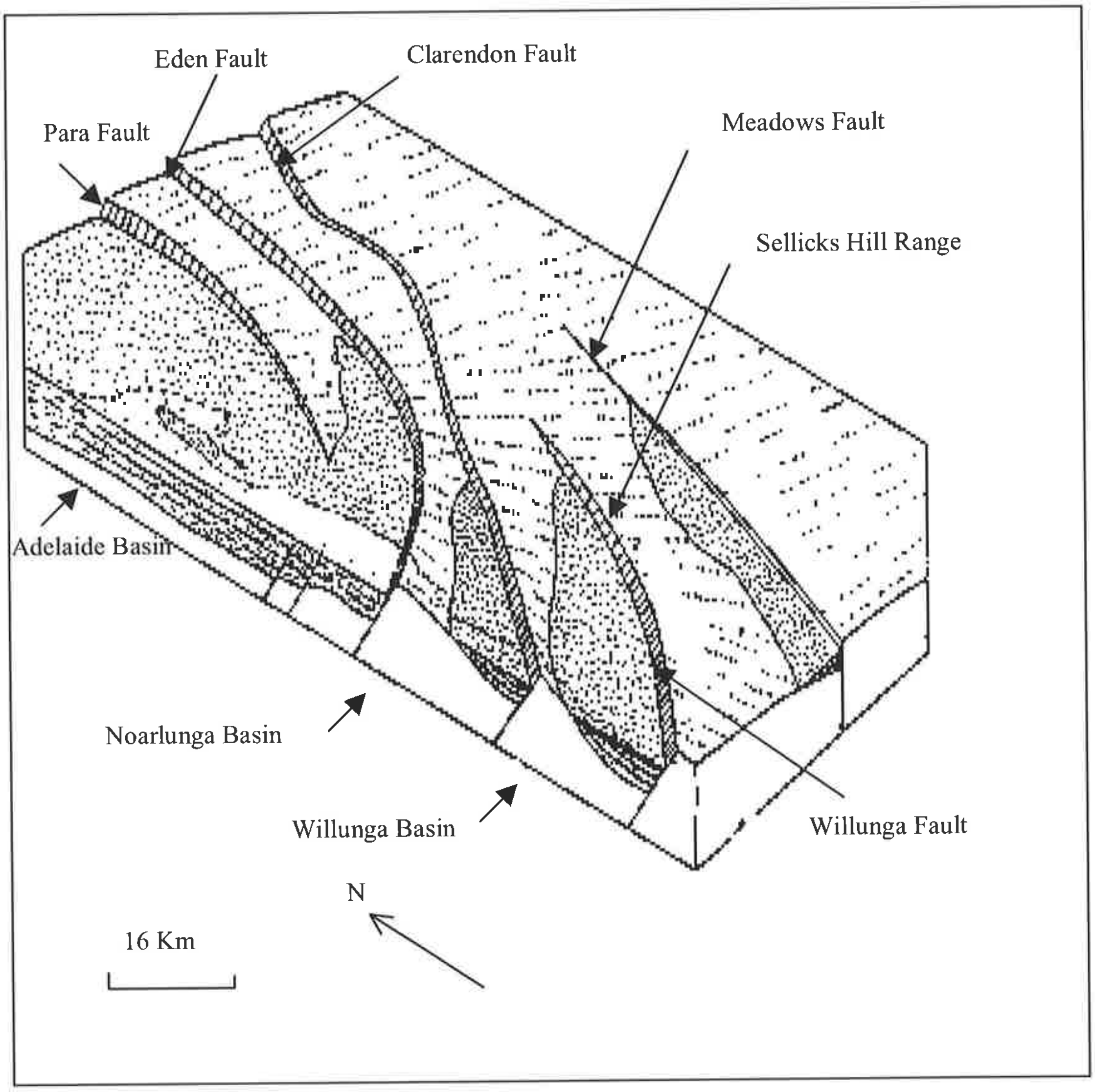


Figure 4.4 Block Diagram of Adelaide, Noarlunga and Willunga Basins Showing the Downthrown Side of the Faults (Watkins and Telfer, 1995)

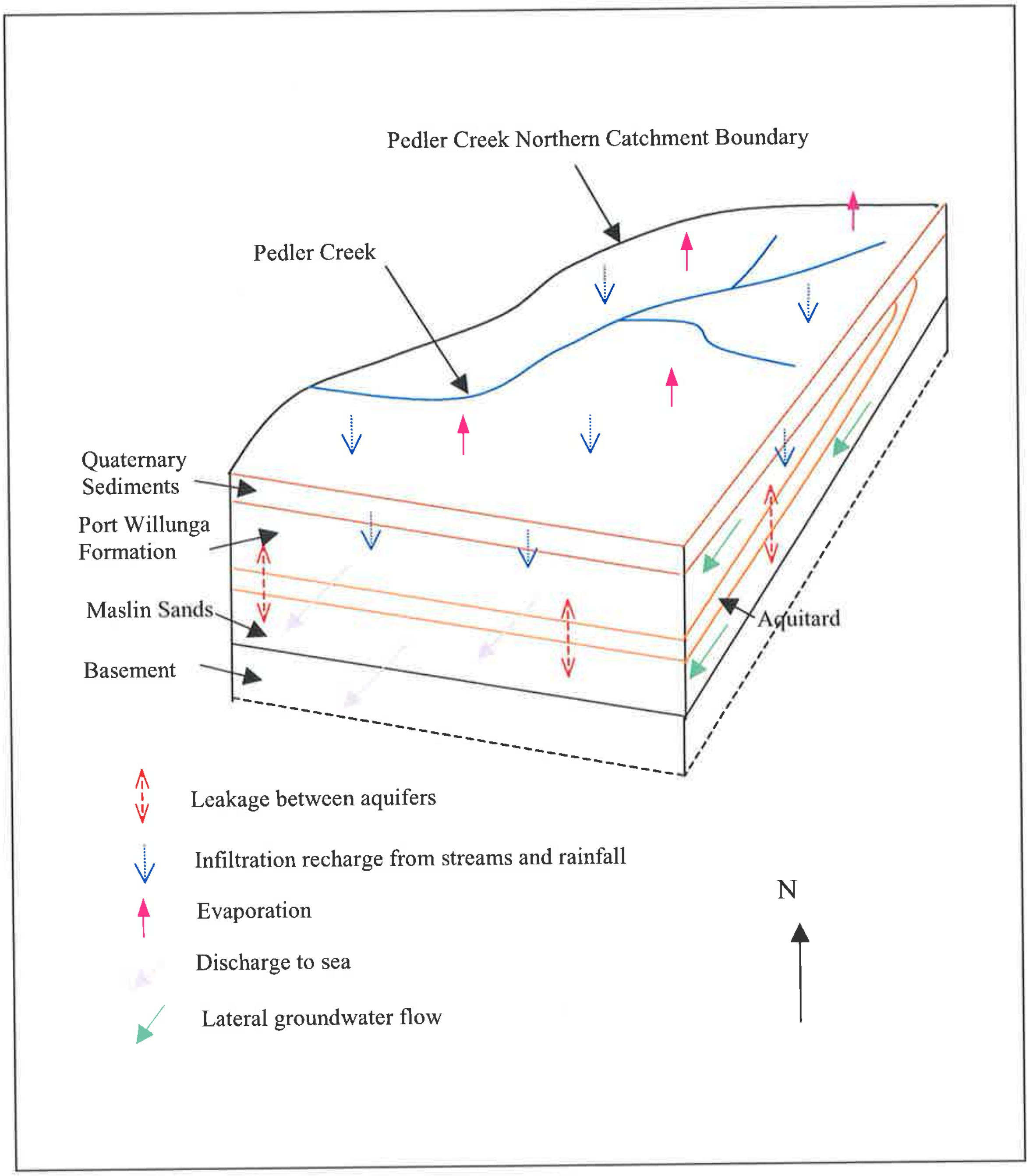


Figure 4.5 Willunga Basin Aquifer Sequence (adapted from Watkins and Telfer, 1995)

4.4 Land Use and Groundwater Use

Agriculture dominates the land use within the basin. This consists predominantly of grazing on the upper reaches (up to 400 m above sea level) and vineyards on the lower reaches (from sea level to 200 m above sea level). Much of the basin has been cleared of natural vegetation to enable agricultural activities. The basin supports some 4450 ha of irrigated crops. Figure 4.6 shows the distribution of crops by area with vines comprising 73% of the irrigated area and almonds 18%. The remaining 9% is made up of pasture, fodder, fruit trees, olives and other crops (Woodward-Clyde, 2000).

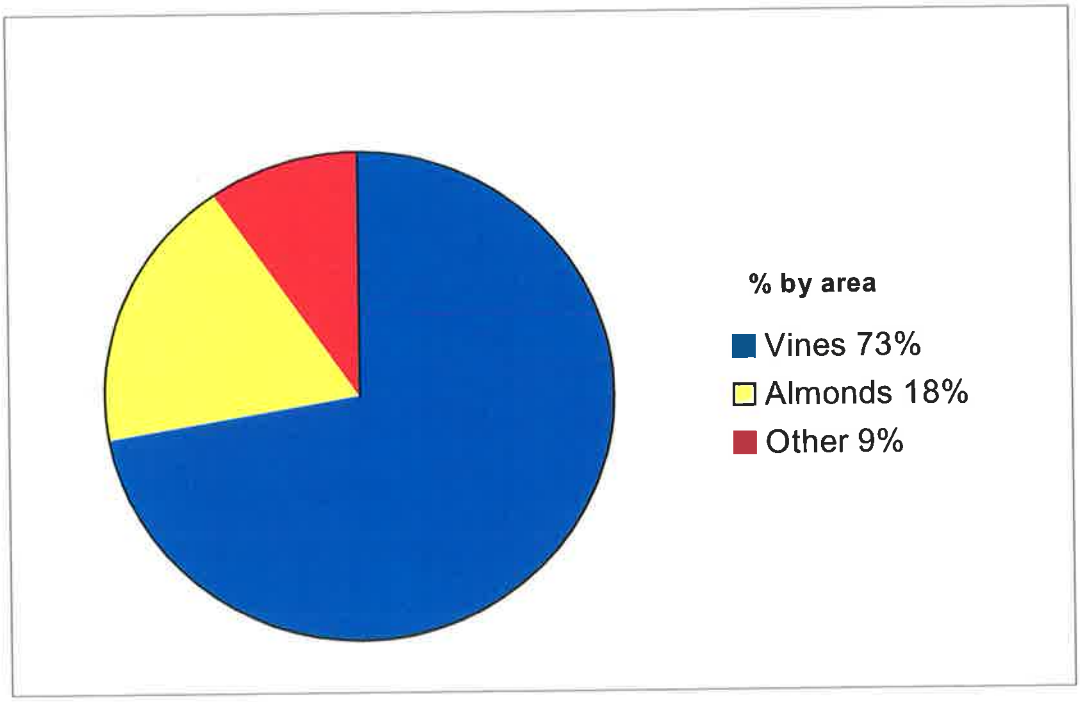


Figure 4.6 Irrigated crops of the Willunga Basin

Groundwater usage within the basin is estimated at 7 380 Ml/year (PIRSA and OCWMB, 1998). Current agricultural practices, predominantly irrigation, are depleting the groundwater from the two main aquifers in the Willunga Basin (Maslin

Sands and the Port Willunga Formation) resulting in a lowering of the standing water level of approximately 0.8m/year. It is estimated that salinity levels are increasing by 10 – 50 mg/L per year within the three main aquifers and within 10 years may exceed the tolerable limit for vineyard irrigation of 1350 mg/L (PIRSA, 1998). This will impact on agricultural practices within the basin.

To protect the groundwater resource within the basin, new water management practices need to be adopted. Alternative sources of water can supplement the groundwater system by artificially recharging the aquifers. These alternative sources include harvesting natural streamflows, harvesting urban stormwater, using treated effluent and transferring winter surplus water from surrounding reservoirs (PIRSA and OCWMB, 1998).

4.5 Pedler Creek

Excess winter streamflow has been identified as a potential source of water to recharge the Willunga Basin groundwater system, and Pedler Creek is used to illustrate the application of this model.

Pedler Creek Catchment is the northern most and largest catchment within the Willunga Basin; it covers an area of approximately 113 km^2 and consists of two main tributaries that join near McLaren Vale (Figure 4.2). Limited water budget data is available for Pedler Creek. Prior to 1999, the creek was ungauged and only one official rainfall gauge existed between 1938 and 1996 within the catchment boundary. The Bureau of Meteorology daily rainfall gauge 23726 was located at McLaren Vale Post Office (PO), and daily rainfall data exists for 1938 – 1996; the gauge was closed in early 1997. In 1993 the Bureau of Meteorology opened a second daily rainfall gauge at Pirramimma Winery located approximately 1 km south of the old gauge (Figure 4.7). Data from this gauge exits from November 1993 to the present.

Pedler Creek has two main tributaries: Pedler Creek north and Pedler Creek south (Figure 4.7). Pedler Creek north has a sub-catchment of approximately 45 km^2

upstream of the junction with Pedler Creek south. Pedler Creek south has a sub-catchment of approximately 15 km^2.

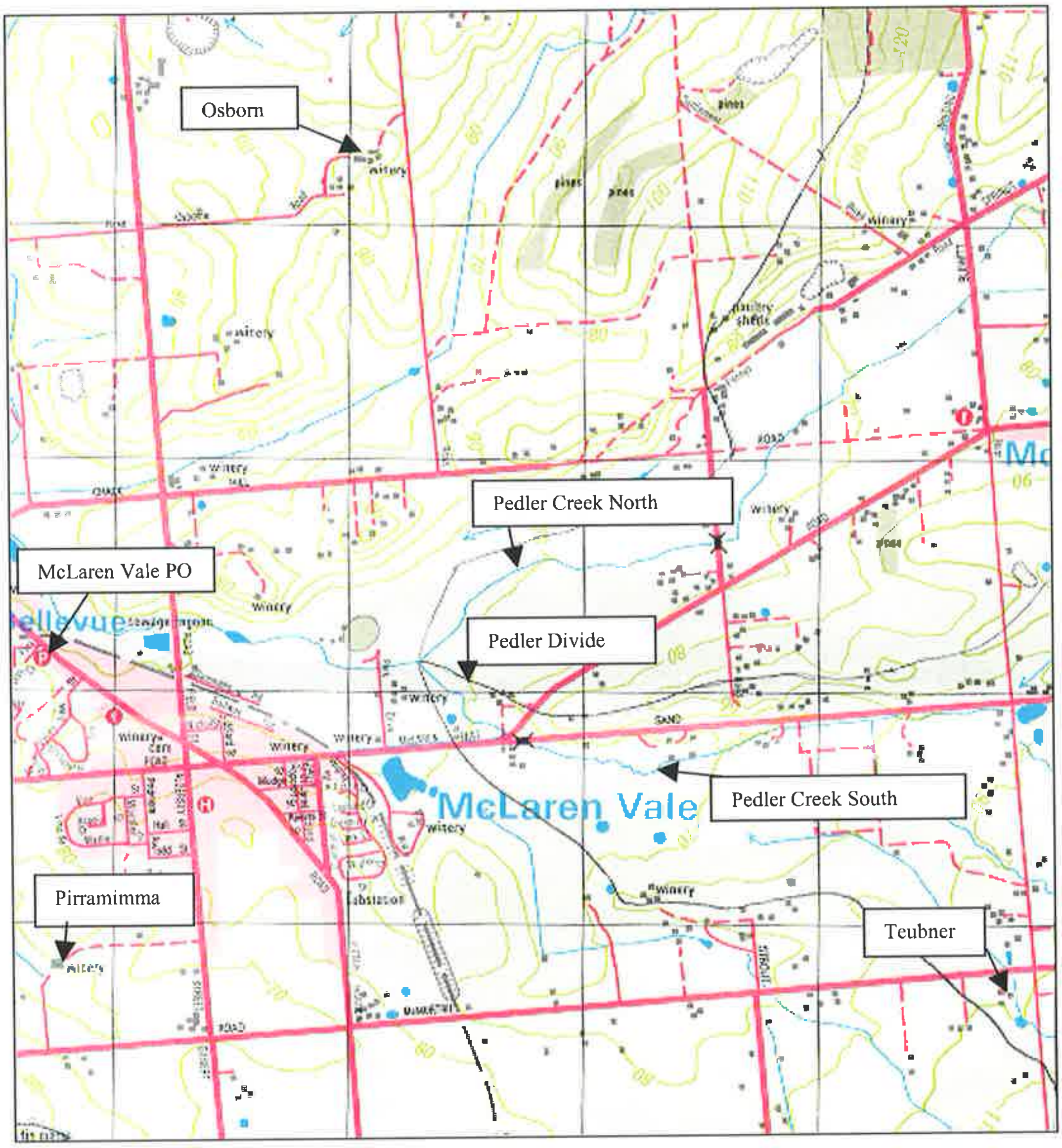


Figure 4.7 Location of Daily Rainfall Gauges and the North and South Arms of Pedler Creek (adapted from Government of SA Noarlunga South Topographic Map)

The water management model is applied to the study area ("Pedler Divide") within the Pedler Creek catchment, which is situated between the two main tributaries and covers an area of approximately 57 hectares. Two off-stream storages are situated on the property: the original storage with a capacity of 1.5 ML can receive water from Pedler

Creek south through an off take pipe. The newer and larger storage (capacity 8.5 ML), is connected to Pedler Creek north by an off-take open diversion channel (Photos 4.1 and 4.2). The southern tributary flows quite spasmodically and only one event between 1996 and 2000 produced enough water for diversion into the smaller storage. The larger storage has filled to capacity on a number of occasions since it was developed in 1998. Artificial recharge from the larger storage into the aquifer started in July 1998. The larger storage is modelled in this study using data up until the end of 2000.

Photo 4.1 8.5 ML Off Stream Storage Storage

Photo 4.2 Off-take Diversion Channel

4.6 Data Collection and Analysis

4.6.1 Rainfall and Evaporation Data

A number of daily rainfall gauges exist within the Pedler Creek catchment. Some are official gauges, from which data is forwarded to the Bureau of Meteorology (BoM) and some unofficial daily rainfall gauges are situated on private properties. The locations of rainfall gauges are shown in Figure 4.7.

The commencement date and the period of operation of the rainfall gauges used in this study are indicated in Table 4.1.

Table 4.1 Daily Rainfall Gauges in Pedler Creek Catchment used in this study

Gauge Location	Period of Record	Length of Record Analysed	Estimated Height Above Sea Level (m)
Mc Laren Vale PO	1938 - Feb 1997	1976-1996	55
Pirramimma	Nov 1993 – present	1994-2000	60
Osborn	1992 – present	1992-1998	110
Teubner	1995 – present	1995-2000	100
Pedler Divide	Oct 1997 – present	1998	70

The McLaren Vale PO gauging station provides the longest historical record for the catchment; unfortunately, the post office was only open Monday to Friday and daily records for Saturday and Sunday rainfalls were only included in the cumulative total on Monday. The Pirramimma rainfall gauge (BoM) is read 7 days per week, which provides a more representative indication of daily rainfall for the area.

The Osborn rain gauge is located on the main property of d'Arenberg winery and is read 7 days per week. The Teubner rain gauge is located south of the southern arm of Pedler Creek and is closest to the Sellicks Hill Range. This gauge is read 7 days per week.

The rain gauge situated on the Pedler Divide property is owned by the Onkaparinga Institute of TAFE and is part of a weather station, that is used to analyse conditions for viticultural purposes at the Pedler Divide Property.

The annual rainfall for McLaren Vale PO for the period 1977 – 1996 is given in Table 4.2 with the overall average for the 20 year period being 541mm.

Table 4.2 Annual Rainfall for McLaren Vale PO for 1977 – 1996

Year	Annual Rainfall (mm)
1977	468.4
1978	573.3
1979	617.2
1980	501.1
1981	573.0
1982	344.0
1983	646.1
1984	486.8
1985	478.6
1986	659.7
1987	560.1
1988	488.8
1989	545.2
1990	535.6
1991	510.6
1992	811.6
1993	599.0
1994	343.0
1995	502.1
1996	576.4
Average over 20 Years	**541**

Table 4.3 Annual Rainfall 1992 - 2000

Station	1992	1993	1994	1995	1996	1997	1998	1999	2000
McLaren Vale PO	811	599	343	502	576				
Pirramimma			327	524	580	484	529	532	643
Osborn	861	583	390	569	679	537	581	N/A	N/A
Teubner				684	718	503	580	607	787

N/A is not available

Table 4.3 shows the annual rainfall for the four gauging stations over the period 1992 to 2000. These values show quite distinct differences over fairly close locations, although McLaren Vale PO and Pirramimma are relatively similar for 1994-1996 and Osborn and Teubner are similar for 1996-1998.

Evaporation data used in this study is from the Mt Bold Reservoir located approximately 18 km north east of the catchment and 23 km south-southeast of Adelaide. This is the only data available in the vicinity of the Pedler Creek catchment. Evaporation data used is monthly evaporation for 1976 to 2000. The pan evaporation data from Mt Bold Reservoir is based on a monthly average; data has been divided by the number of days per month to obtain a daily value in mm/day to provide a format suitable for the AWBM Model.

4.6.2 Streamflow Data

Before the end of 1999, no streamflow gauging stations existed in the Pedler Creek Catchment. At the end of 1999, two stations were installed: one on the northern arm of Pedler Creek and one on the concrete drain along the southern arm of Pedler Creek. The two streamflow gauging stations were installed for the Onkaparinga Catchment Water Management Board to determine the flow in Pedler Creek and to obtain daily streamflow values.

The hydrometric station on the northern arm of Pedler Creek is designed to record continuous flow at all ranges. The station is located approximately 30 m upstream of the off-take channel. The weir is a triangular flat vee weir with a 1:10 cross slope and 1:2 upstream and down stream slopes (Water Data Services, 1999, Photo 4.3). Photo 4.4 shows the creek downstream of the weir. The off-take diversion channel is located next to the tree stump in the photo.

The hydrometric station on the southern arm of Pedler Creek has also been designed to record continuous flow at all ranges. The southern station is located close to the off-take structure. The weir is a triangular low profile flat vee weir with 1:10 cross slopes and 1:2 upstream and downstream slope (Water Data Services, 1999).

Photo 4.3 Stream Gauging Station Pedler Creek North

Photo 4.4 Downstream of the Gauging Station

The primary aim of obtaining data from streamflow gauging stations is to calibrate the rainfall-runoff model to improve the estimation of the surface storage parameters. The usefulness of streamflow records for calibration of the AWBM model increases with the length of the streamflow record. Ideally the data records should include a sustained dry period in which each of the surface stores is empty and a wet period that is sufficient to fill all of the surface stores, producing runoff from all of the catchment (Boughton, 1993a).

Since the completed installation of the streamflow gauging station at the end of 1999, a number of runoff events have occurred. A verification of the rainfall-runoff model has been attempted using one year of data from 2000, and is discussed in Section 4.7.1.

4.7 Development of the Rainfall- Runoff Model and Application to Pedler Creek

The AWBM model was applied to the Pedler Creek Catchment. Daily rainfall data for 1977-1996 from the McLaren Vale PO rainfall station and monthly evaporation data for 1977-1996 from the Mt Bold Reservoir were initially used as input. The average surface storage capacity (SSC) in the model was set to 120mm throughout this Chapter. This is the default value for the AWBM model. It is also the value estimated by Nathan and McMahon (1990) based on catchments in eastern Australia calibrated using the related SFB model as discussed in Section 3.3.

The surface storage parameters were set using the default values as follows:

Smallest Store	$C_1 = 0.5 * 120 = 60$ mm over 0.2 of the catchment,
Middle Store	$C_2 = 0.75 * 120 = 90$ mm over 0.4 of the catchment,
Largest Store	$C_3 = 1.5 * 120 = 180$ mm over 0.4 of the catchment.

As Pedler Creek is ephemeral and has a small catchment, baseflow was assumed to be negligible and all runoff was assumed to be surface runoff. To allow for zero baseflow, the BFI parameter was set to zero and K to 1. For small catchments there is no need to rout daily values of surface runoff; therefore the surface recession constant

(KS) was set to zero, which was the default value. Thus the AWBM Model was reduced to a one- parameter (SSC) model.

The average annual runoff generated from the AWBM model for Pedler Creek based on rainfall from McLaren Vale PO for the period 1977-1996 with an average annual rainfall of 541 mm/a is 4000 ML. This is obtained using a catchment area of 113 km^2. As shown in Table 4.4 this is comparable with results obtain by Cresswell (1994) using runoff estimates based on regional relationships developed for catchments of similar characteristics, but is quite low in comparison with the estimate of 8418 ML obtained by the EPA (1998). The EPA estimate was based on the correlation of 19 gauged catchments within the Mt Lofty Ranges, from which a relationship was derived between catchment characteristics and annual runoff and this was applied to ungauged catchments such as Pedler Creek.

Cresswell (1994) used regional relationships developed for catchments of similar characteristics within the Adelaide Hills, South Australia. Rural runoff was based on two areas representing the different soil types and winter rainfall. The Pedler Creek catchment was divided into a steep zone area and plain zone area. The steep zone was assumed to be a more efficient catchment because of its steepness and thin soils and was assumed to generate runoff similar to the more efficient catchments in the Mt Lofty Ranges. Efficiency was defined as the ratio of mean annual runoff to the mean annual rainfall recorded in the catchment, with the plain zone being less efficient because of its significant soil depth and smaller slope. Catchment characteristics for the Inverbrackie Creek Catchment were adopted for the steep zone and characteristics for Echunga Creek Catchment were adopted for the plain zone. Cresswell derived an equation representing the runoff for the Inverbrackie and Echunga Creek Catchments and using the winter rainfall within the Pedler Creek Catchment estimated runoff for Pedler Creek. Both Inverbrackie and Echunga Creeks are on the eastern side of the Mt Lofty Ranges, so they may not be a true representation of the Pedler Creek Catchment, which is on the western side. Another difference in the runoff estimation technique was the use of winter rainfall. In a number of unseasonally wet years, high levels of rainfall occurred during the summer months, which would not be used in the estimation of runoff using Cresswell's technique.

Table 4.4 Comparison of runoff results for Pedler Creek Catchment

Pedler Creek Catchment	This Study	Cresswell (1994)	EPA (1998)
Estimated Annual Runoff (ML)	4000	4400	8418
Annual Depth of Runoff (mm)	35	40	74

To provide a more realistic approximation of runoff across the catchment area, an improved representation of annual rainfall for the catchment was required. Figure 4.3 shows an isohyet map for the Willunga Basin. It is evident that rainfall ranges from 500 mm/a near the coast to 775 mm /a in the upper reaches of the catchment. To account for the higher rainfall received in the upper reaches, the daily rainfall values from the McLaren Vale PO for the period 1977 to 1996 were scaled using two ratios. This enabled the evaluation of the effect of increased rainfall over the catchment. The two ratios were 750/541 to examine a 750 mm average annual rainfall scenario, and 650/541 for a 650 mm average annual rainfall scenario.

The AWBM model was re-run with scaled daily rainfall inputs corresponding to 541 mm/a, 650 mm/a and 750 mm/a and the surface storage capacity parameter was set at 100mm, 120mm and 140mm for each ratio. This allowed a sensitivity analysis to be performed using realistic variations in both rainfall and surface storage capacity. The results are given in Table 4.5.

Table 4.5 shows that as the SSC is reduced, the runoff increases, as is expected. As an example, for the 650 mm rainfall scenario, reducing S by 17% from 120 mm to 100 mm, increases the runoff by 19%. Conversely, increasing rainfall increases runoff. Using S=120mm with rainfall increasing 16% from 650 mm to 750 mm, the runoff increases by 60%. This indicates that, using the AWBM model, runoff is much more sensitive to variations in rainfall than SSC. Obtaining an accurate estimate of the rainfall, therefore, is more important to develop an accurate assessment of the runoff, with thc storage capacity being less important. An examination of the isohyets in Figure 4.5 suggests that 650 mm/a is a good representative average rainfall value for the Pedler Creek catchment. Using the average SSC value of 120 mm, together with this rainfall value, gives an estimated average annual runoff of 9400 ML for the Pedler Creek catchment, which is comparable with the EPA result shown in Table 4.4.

Table 4.5 Results of AWBM for Various Rainfall (R, mm) and Surface Storage (S, mm) Values

	R =541 S= 100	R= 650 S= 100	R = 750 S= 100	R =541 S= 120	R = 650 S= 120	R = 750 S= 120	R = 541 S= 140	R= 650 S= 140	R = 750 S= 140
Annual Runoff (ML)	5 100	11 100	17 700	4 000	9 400	15 700	3 100	7 800	13 800
Mean Runoff Depth (mm)	45	99	160	35	83	140	28	69	120

As the AWBM model runs on a daily time step it is imperative that the daily rainfall records are as accurate as possible. The consequence of cumulative readings on Monday of the McLaren Vale PO's weekend rainfall is that it gives the model inaccurate daily rainfall values, that may adversely affect the runoff results. The annual runoff difference between Pirramimma and McLaren Vale rainfall is given in Table 4.6. The total annual rainfall difference between Pirramimma and McLaren Vale for the period 1994-1996 was less than 5% for any year. A consistent difference of 8 – 20% between the Osborn and Pirramimma annual totals is evident for 1994-1998 which would be expected as Osborn is located at a greater height above sea level (110m compared with 60m for Pirramimma). The Osborn station is situated on a hill and is likely to be subjected to slightly different weather patterns and an orographic effect. Similarly the difference of annual rainfall averages between Osborn and McLaren Vale is in the 2-18% range. There is a large difference between the annual rainfall totals at Teubner and McLaren Vale (and Pirramimma) for 1995 and 1996. This may be due to weather patterns from the south-west which would result in an increase in orographic precipitation. Similarly for 1995 a comparison between Osborn and Teubner annual rainfall average shows a difference of 20%. This again may be due to weather patterns and the prevailing direction of storms. For 1996 – 1998 the difference between Osborn and Teubner annual rainfalls is 6% or less.

Table 4.6 Annual Rainfall Difference Between the Various Rain Gauges

	1992	1993	1994	1995	1996	1997	1998
Pirramimma – McLaren Vale (mm)			-15.4	22.7	4.2		
%			-4.5	4.5	0.7		
Osborn – McLaren Vale (mm)	50.2	-15.7	47.2	67.1	103.1		
%	6.2	-2.6	13.8	13.4	17.9		
Teubner – McLaren Vale (mm)				181.9	142.1		
%				36.2	24.7		
Osborn – Pirramimma (mm)			62.6	44.4	98.9	52.8	52.4
%			19.1	8.5	17.0	10.9	9.9
Teubner – Pirramimma (mm)				159.2	137.9	19.3	51.3
%				30.3	23.8	4.0	9.7
Teubner – Osborn (mm)				114.8	39.0	-33.5	-1.1
%				20.2	5.7	-6.2	-0.2

Due to the inconsistent daily readings from the McLaren Vale PO daily rain gauge, a comparison was made between this and the other daily rain gauge readings using linear regression analysis. This was compared to the results of regressing monthly totals of rainfall from McLaren Vale PO against the Pirramimma station. The results are given in Table 4.7. The closest correlation of daily rainfall exists between the Osborn and Pirramimma stations ranging from 0.7033 to 0.9405, which suggests in the absence of a historical record for Pirramimma station, the Osborn station is a reasonable substitute. The monthly rainfall totals of the Pirramimma and McLaren Vale stations correlated extremely well as expected, ranging from 0.9857 to 0.9981 for the period of record 1994-1996. In comparison a poor correlation exists for the daily rainfall readings, which probably reflects the absence of readings on Saturdays and Sundays (which were left as zeros) for the McLaren Vale station.

Although runoff analysis has been carried out using the 20 years of rainfall records for McLaren Vale PO, the daily runoff values obtained may not give an accurate indication of runoff for that period due to the cumulative Monday readings of rainfall. Comparing this with the difference in annual rainfall between Pirramimma and McLaren Vale PO suggests that on an annual and monthly basis there is good correlation between the data sets. So, using the historical records prior to Pirramimma rainfall station commencement suggests that a good estimate of **annual** runoff received in Pedler Creek will be obtained, but on a **daily** basis, the estimated runoff may be inaccurate.

Table 4.7 Rainfall Regression Analysis (Value Shown is the Correlation Coefficient)

Comparison Stations	1992	1993	1994	1995	1996	1997	1998	1999	2000
Pirramimma vs.McLaren Vale Monthly Rainfall Daily Rainfall			 0.986 0.527	 0.996 0.553	 0.998 0.509				
Osborn vs.McLaren Vale PO Daily Rainfall	0.436	0.384	0.502	0.552	0.490				
Teubner vs.McLaren Vale PO Daily Rainfall				0.236	0.371				
Osborn vs.Pirramimma Daily Rainfall			0.941	0.883	0.796	0.703	0.928		
Teubner vs.Pirramimma Daily Rainfall				0.503	0.706	0.954	0.776	0.817	0.912
Teubner vs.Osborn Daily Rainfall				0.478	0.584	0.673	0.698		

Table 4.8 Estimated Runoff (mm) for 1994-2000 using the AWBM Model

Station	1994	1995	1996	1997	1998	1999	2000
McLaren Vale	0	43.5	71.8	N/A	N/A	N/A	N/A
Pirramimma	0	64.9	77	0	36	3	56
Pirramimma data scaled to an annual rainfall of 650mm.	46.1	151.1	120.4	25.3	94.5	22	142
Osborn	0.7	72.9	122.5	11	40	N/A	N/A
Teubner	N/A	152.2	119.9	0	58.7	21	155

The estimated annual runoff using the Pirramimma, Osborn and Teubner rainfall stations is given in Table 4.8. A more realistic estimation of annual runoff has been calculated using the daily rainfall values for Pirramimma scaled so that the annual average is 650mm. The average scaled Pirramimma estimated runoff for 1994 – 1998 equates to 87 mm of runoff per annum. This is comparable with the mean runoff depth of 83 mm given in Table 4.5 for an annual rainfall of 650 mm and a surface storage capacity of 120 mm, using the McLaren Vale PO daily data.

Table 4.9 Runoff Regression Analysis (Value Shown is the Correlation Coefficient)

Comparison Stations	1994	1995	1996
Pirramimma vs. McLaren Vale Daily Runoff	No runoff	0.352	0.479

A runoff regression analysis between the daily runoff produced from the Pirramimma rainfall data and McLaren Vale PO data is given in Table 4.9. The regression analysis for the daily rainfall (Table 4.7) and runoff (Table 4.9) for McLaren Vale PO and Pirramimma indicates that rainfall is slightly better correlated than runoff.

By conducting a histogram analysis of the daily rainfall recorded for both the Pirramimma rainfall gauge and the McLaren Vale PO rainfall gauge for 1994 –1996 and plotting the cumulative percentage for each rainfall range, it is evident that the curves are quite similar except for the lowest rainfall range. This is shown in Figure 4.8.

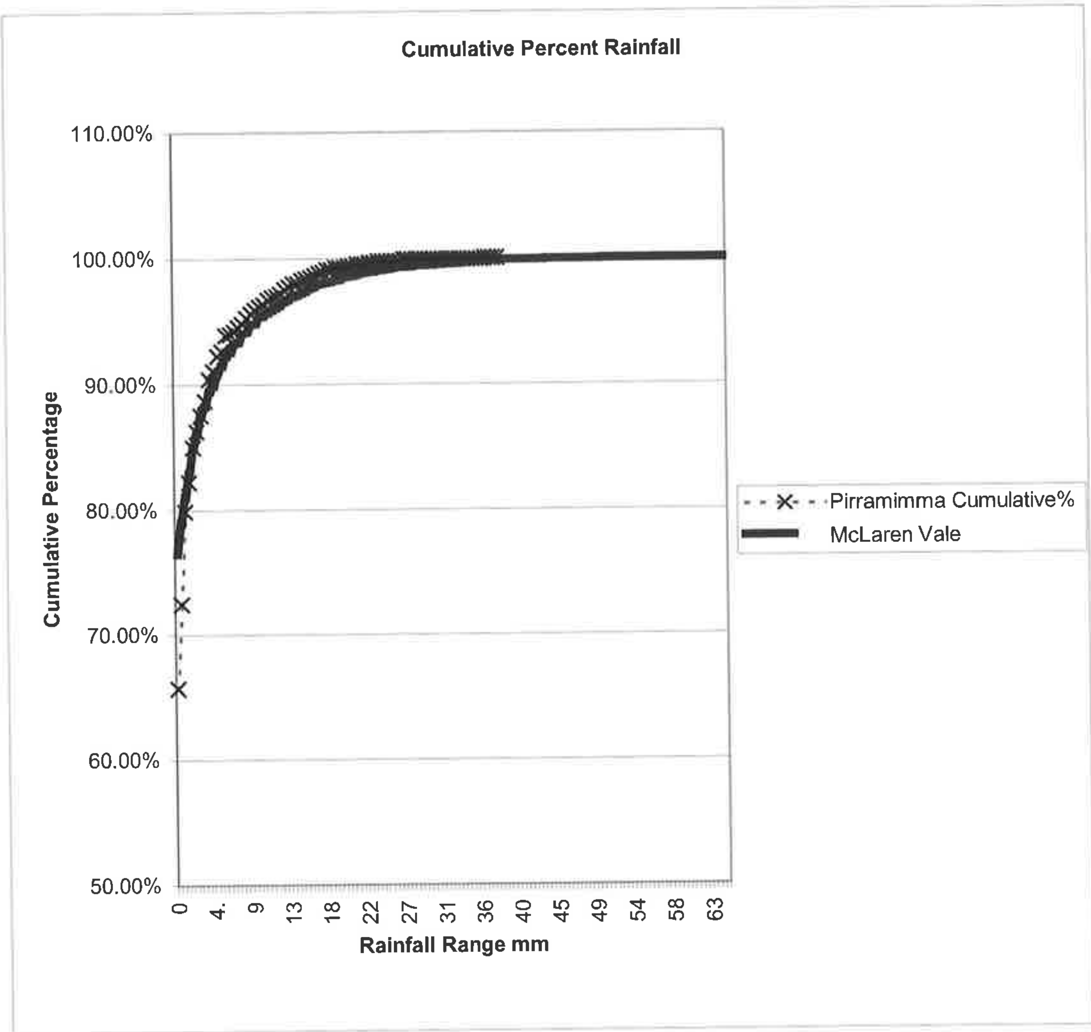


Figure 4.8 Cumulative Percentage Plot for McLaren Vale PO and Pirramimma Rainfall Gauges

Figure 4.8 shows that the Pirramimma data has a higher percentage of exceedance for daily rainfall of 9 to 22 mm. The general agreement of the two curves suggests that the data from McLaren Vale PO from 1977 to 1996 provides a good approximation for historical analysis as input to the rainfall-runoff model. To check the error of reading the rainfall 5 days per week versus 7 days per week, the rainfall-runoff model was run using data from McLaren Vale for 1992, the wettest year over the 1977-1996 period, during which a few large storms produced a large amount of rainfall and runoff. Data was altered to spread rainfall readings equally over weekends and public holidays, thus approximating a full record of daily readings. Comparing the results using the daily-modified data set against the measured (incorrect) data set produced monthly runoff results that were almost identical (Table 4.10). Scaling the daily rainfall by 650/541 produced a greater amount of runoff compared with the runoff produced from the actual rainfall recorded at McLaren Vale PO for 1992.

Table 4.10 Monthly runoff (mm) for 1992 McLaren Vale PO produced from scaled 650/541 rainfall, altered rainfall (to account for weekends and public holidays) and runoff produced from actual recorded rainfall

Data Set	J	F	M	A	M	J	J	A	S	O	N	D
Scaled 1992	0	0	0	0	0	12.4	7.5	21.5	55.2	15.5	0	0
Altered 1992	0	0	0	0	0	1.4	1	4.6	25.2	4.4	0	0
Actual 1992	0	0	0	0	0	1.4	1	4.7	25.3	4.4	0	0

This indicates that the filling of the three stores in the rainfall-runoff model is not sensitive to the spread of rainfall over a few days but is purely a function of the volume entering the buckets. For example, if a reading of 24mm was recorded on the Monday where, in actual fact, rainfall of 8mm per day was received on Friday, Saturday and Sunday, the cumulative addition to the three buckets over three days is the same as adding all of the rainfall on Monday. Therefore it can be assumed that during the winter months when evaporation is low, 7-daily readings of rainfall versus 5-daily readings of rainfall produces similar runoff. If this occurred during the summer months, high evaporation may influence the estimated runoff. As most of the rainfall is received during the winter months, it is assumed that the 5-daily readings from the McLaren Vale PO for the period 1977-1996 are sufficient to provide a historical rainfall data set for use with the rainfall-runoff model.

Table 4.11 shows the runoff produced using the Pirramimma rainfall station data for the period of record 1994-2000. The bottom row gives estimated annual runoffs using the daily rainfalls scaled by the ratio 650/541. A total annual average runoff of 9,700 ML is estimated from the scaled rainfall data from Pirramimma station using the default values of the AWBM model. This figure is comparable with the figure of 9400 ML/year obtained using scaled McLaren Vale data (Table 4.5) and the figure of 8418 ML/year obtained by the EPA. The estimate of 4400 ML/year of Cresswell appears to be low, although calibration of the AWBM model is required before a true comparison can be made.

Table 4.11 Estimated Runoff (ML) for Pedler Creek Catchment

Station	1994	1995	1996	1997	1998	1999	2000	Average ML/year
Pirramimma	0	7330	8700	0	4070	339	6328	3820
Using scaled Piramimma data (650/541)	5210	17074	13605	2860	10680	2486	16046	9700

4.7.1 Attempted Calibration of AWBM Model Using 2000 Data

The purpose of calibrating the AWBM model for surface runoff is to determine the surface storage capacity and partial area parameters in order for the excess to closely match observed runoff data. As discussed previously in section 3.3, the water balance at each time step requires the addition of rainfall and the subtraction of evapotranspiration to each of the stores. Runoff results when the capacity of any store is exceeded. In order to calibrate the model, a number of years of streamflow data is required covering at least one wet and one dry year to confidently estimate the surface storage capacity and partial area parameters.

The streamflow data used for calibration is the limited data available for the year 2000 provided by Water Data Services (WDS). This data is only provisional, as the streamflow calibration performed by WDS has been extrapolated based on theoretical equations. These flow equations may change and to ensure confidence in the calibration, several years of streamflow data is required so that high and low flows are captured (Water Data Services, 2000). Until such time as calibrated streamflow data is available, the existing data should only be used as a rough guide (Water Data Services, 2000).

In an attempt to verify the results from the AWBM model a provisional calibration was carried out together with a flow comparison analysis.

Using the scaled Pirammima data for 2000 (as discussed in section 4.6) with the provisional streamflow data from 1 January to 31 December 2000, the calibration program SURF.PAS within the AWBM model was run. The aim of using the limited data collected from the stream gauge station was to try to gain a rough estimate of the capacities and partial areas of the surface stores used in this study. Using a multiple linear relationship the set of surface storage parameters (C_1, C_2, C_3 and A_1, A_2, A_3) which most closely matched the actual runoff values for 2000 was determined. From Boughton (1996),

$$RO_j = e_{1,j}\, A_j + e_{2,j}\, A_2 + e_{3,j}\, A_3, \qquad j=1, \ldots, 12 \tag{4.1}$$

where RO_j is the actual runoff in the jth month, $e_{n,j}$ is the calculated excess (runoff) from capacity C_n for the jth month and A_n is the partial area of the catchment represented by capacity C_n for n = 1 to 3.

Runoff results from the smallest storage capacity prior to or at the same time as it occurs from the larger capacities. The smallest capacity is determined by trial and error. This involves testing a range of values for the capacities over the whole catchment to find the best match for the months in which actual runoff occurs. A full description of the calibration process of the AWBM model can be found in Boughton (1996).

The SURF.PAS program within the AWBM calibrates the surface storage capacities and the partial areas of the three stores in the AWBM model (Boughton, 1996) to a streamflow runoff data set in which runoff only consists of surface runoff. The output from running the calibration program gave the following results:

C_1 – a soil storage capacity of 0 mm over 0.046 of the catchment area,
C_2 – a soil storage capacity of 124 mm over 0.223 of the catchment area,
C_3 – a soil storage capacity of 269 mm over 0.731 of the catchment area.

A plot of the runoff calculated using the calibrated parameters is given in Figure 4.9.

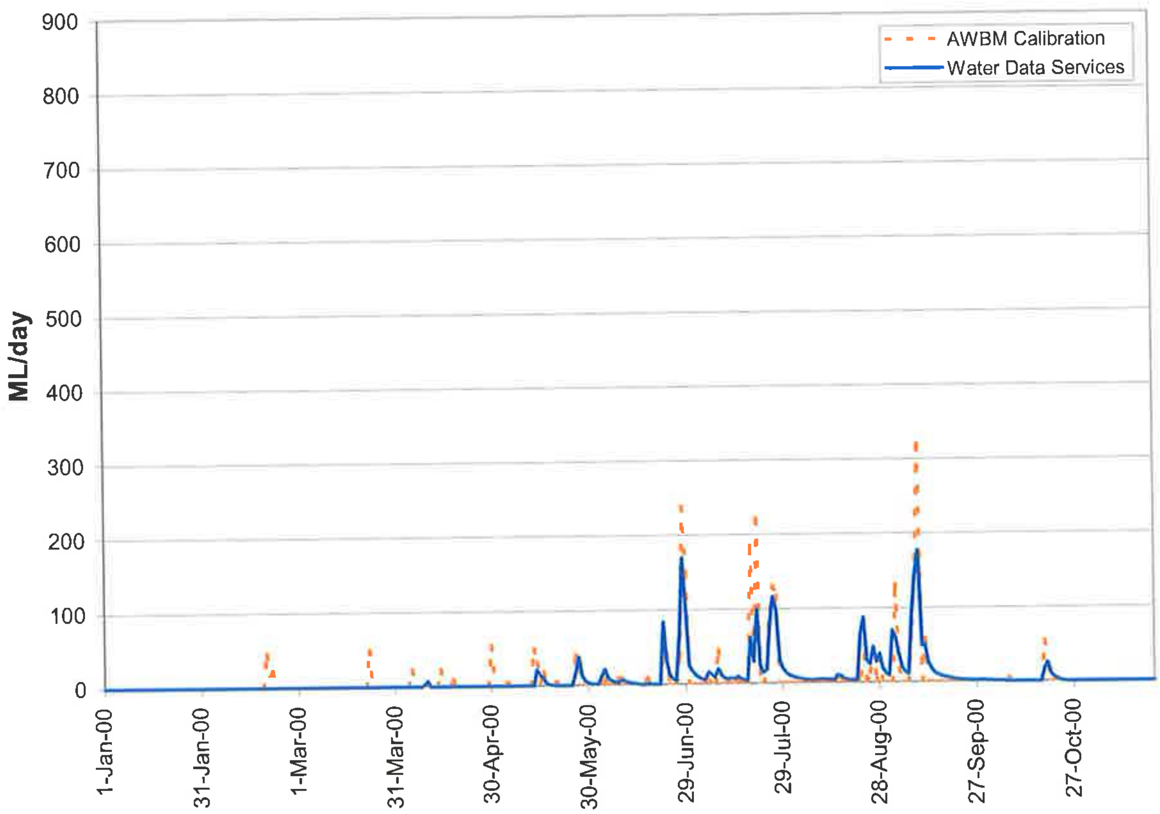


Figure 4.9 Plot of AWBM runoff (calibrated) and field data

It is evident that using the calibrated parameters, runoff is calculated for the period February to the middle of May whereas flow has not been recorded in the field for this period. In addition, the calibrated parameters overestimate the peaks for June to September. The total annual flow for 2000 estimated using the calibrated parameters is 2844 ML, which compares well on an annual basis with 2904 ML from the field data. Due to such limited data and 2000 being a very wet year, the values obtained from the calibration were not used in the running of the model, as it was preferred to use the average value of 120 mm for the soil storage capacity as discussed in Section 4.7.

In another attempt to verify the calculated runoff produced from the AWBM model in this research, a flow comparison analysis was performed and a plot of calculated flow values and measured flow values was constructed for 2000. Figure 4.10 shows the runoff for Pedler Creek North. The AWBM results using both scaled Pirammima rainfall and Teubner rainfall produce corresponding peaks with similar amplitudes with the total annual flow of 6390 and 6975 ML respectively. Comparing this with the measured data from Water Data Services, it is apparent that the time of the peaks is somewhat similar but the amplitudes and total annual volume are not. This would suggest that the runoff calculated by the AWBM model is being over-estimated and a possible increase in the SSC parameter is required. Changing the SSC parameter from 120 mm to 150 mm and rerunning with the scaled Pirammima data produced less runoff, as shown in Figure 4.11 with a number of missing peaks (compared with field data) during May and August. The total annual flow calculated using the SSC value of 150 was 1975 ML, which is comparable with the total annual flow measured in the field. Another explanation for the lack of daily similarity between calculated and measured hydrographs is the unreliability of the field data as discussed previously.

At this stage attempting to calibrate the AWBM model using 1 year's data is inconclusive. Comparing the results obtained by running the calibration program within the AWBM model and those obtained by the flow comparison analysis suggests the current AWBM model with SSC=120 mm may be over predicting the runoff. However, the present analysis has shown the potential applicability of the AWBM model in being able to predict runoff.

Until such time as accurate field data is available, and the calibration of the AWBM model can be achieved and be able to be used with confidence, the default values for the AWBM model will be used in this model.

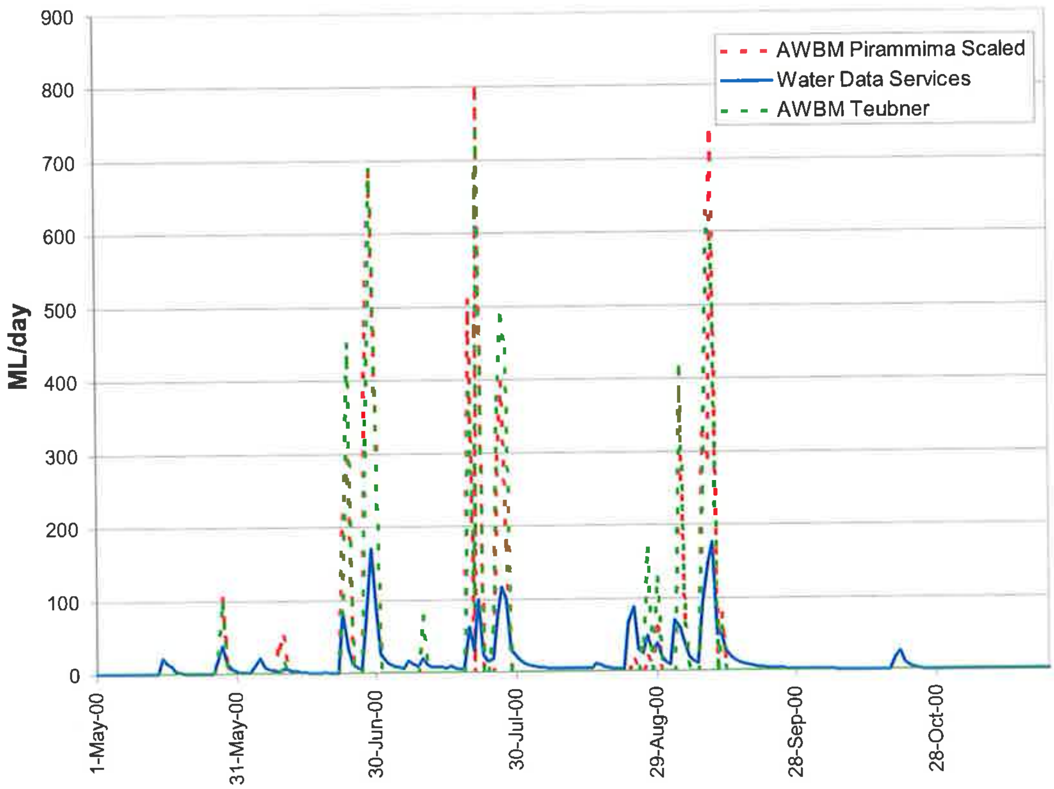


Figure 4.10 **Calculated AWBM runoff using Teubner rainfall, scaled Pirammima rainfall and field data.**

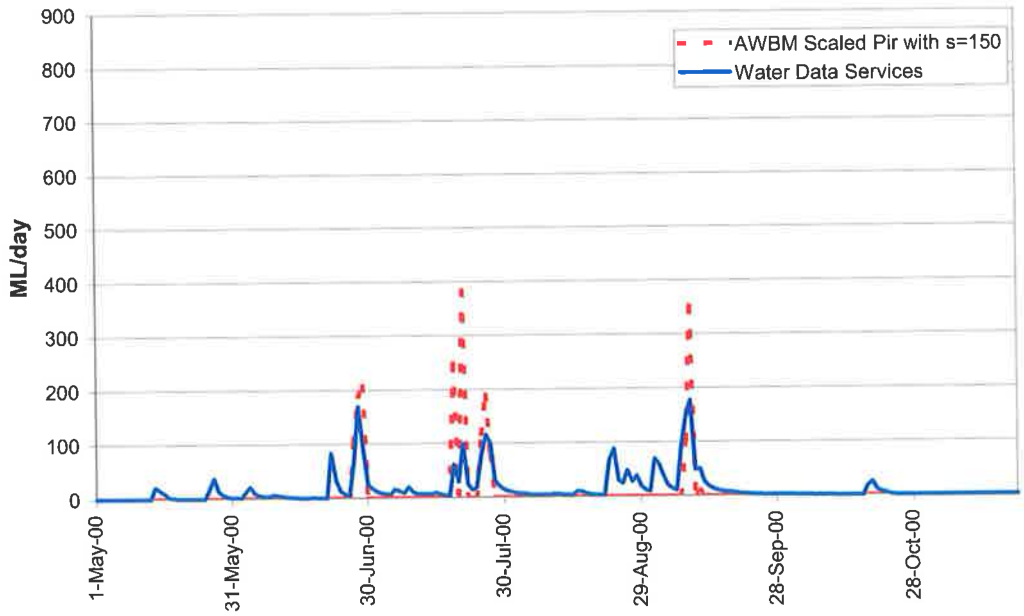


Figure 4.11 **Plot of calculated AWBM runoff using scaled Pirammima rainfall with SSC = 150 mm and field data.**

4.8 Application of the Surface Storage Model to Pedler Divide

The surface storage model is applied to the off-stream storage dam. The dam provides temporary storage for diverted streamflow prior to injection.

Runoff from Pedler Creek is diverted via an open diversion channel (Photo 4.5) into the 8.5 ML storage on the property at Pedler Divide (Figure 4.12 and Photo 4.6).

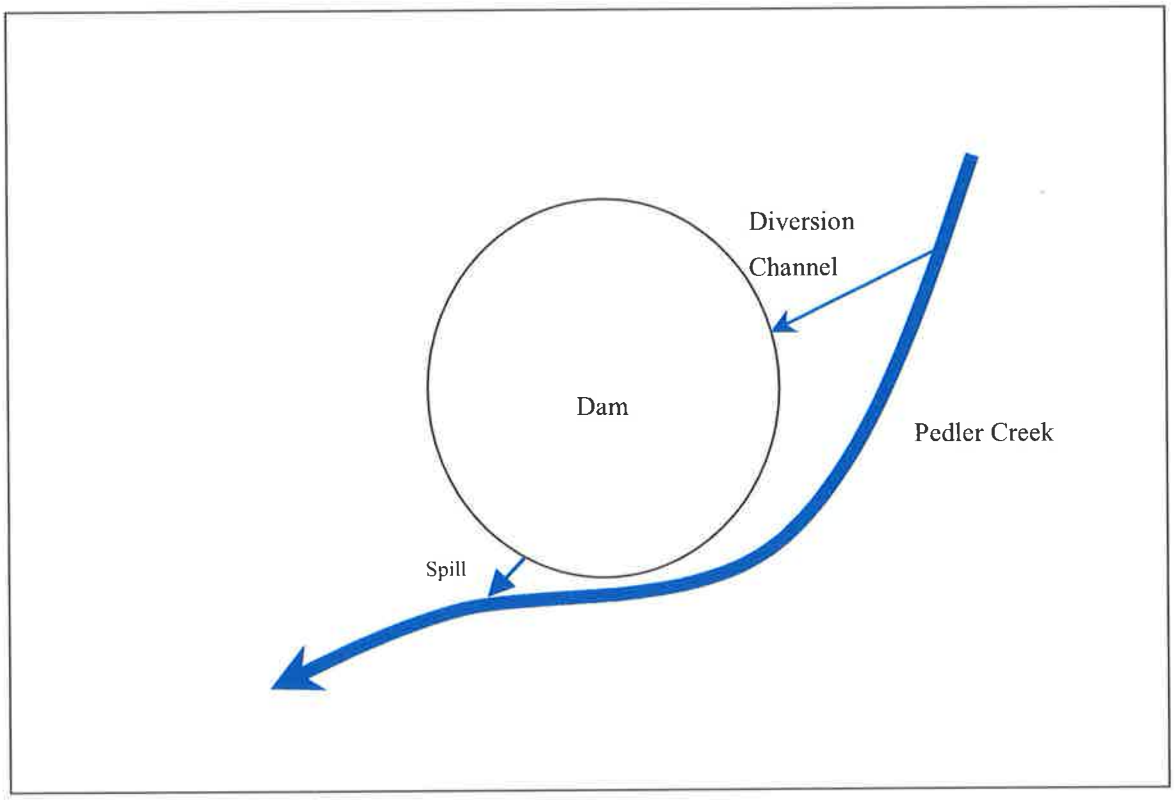


Figure 4.12 Schematic of Pedler Creek and Diversion Channel

Prior to recharge commencing on the property, irrigation needs were met primarily from groundwater. The groundwater was extracted and pumped into a smaller storage where it was subsequently pumped through sand filters and out through the irrigation system to irrigate the vines. The irrigation requirements for the viticulture property are approximately 13 ML/year (based on 97/98 data) (pers. comm. d'Arenberg Winery).

Photo 4.5 Off Stream Diversion Channel at Inlet to Storage

Photo 4.6 8.5 ML Storage at Pedler Divide

4.8.1 Estimation of Flow into the Storage

Water flowing into the storage is estimated using the triangular hydrograph approach described in Section 3.4.2

The AWBM Model is used to estimate the daily runoff depth produced from the total catchment. The catchment area of Pedler Creek north of the off-take diversion structure at Pedler Divide is estimated to be 45 km^2.

The daily simulation of the AWBM model produces daily runoff in mm from the scaled rainfall data of Pirramimma station. The daily volume produced from the runoff events is calculated using the following equation:

Total Volume (m^3) = runoff (mm) x Catchment Area (km^2) x 1000, (4.2)
where Catchment Area = 45 km^2.

As an example, the estimated daily runoff volume for 1996 is shown in Figure 4.13. In this Figure day 1 is January 1st. A total volume of 5418 ML of runoff is estimated for 1996 for the 45 km^2 catchment. Twenty three runoff days occurred during 1996, starting at day 179 with the last runoff event occurring on day 240. By estimating the depth of flow in Pedler Creek for the 23 days, the flow entering the diversion channel can be estimated.

The variation in flow rate over each day has been approximated by assuming a triangular hydrograph as discussed in Section 3.4.1. The area under the hydrograph represents the total daily volume. The maximum flowrate can be determined simply by using Equation 4.3.

Total Daily Volume (m^3) = Tc (hours) x $Q_{(max)}$(m^3/s) x 3600sec/hour (4.3)

Rearranging Equation 4.2 in terms of Q(max) gives:

Q(max) = Total Daily Volume/(Tc x 3600)

Therefore Q(max) = Total Daily Volume/21600

The peak of the triangular hydrograph occurs at the time of concentration.

The time of concentration,

$$Tc(hour) = 0.5A^{(2/3)}, \tag{4.4}$$

where A = 45 km^2.

Therefore Tc = 6 hours.

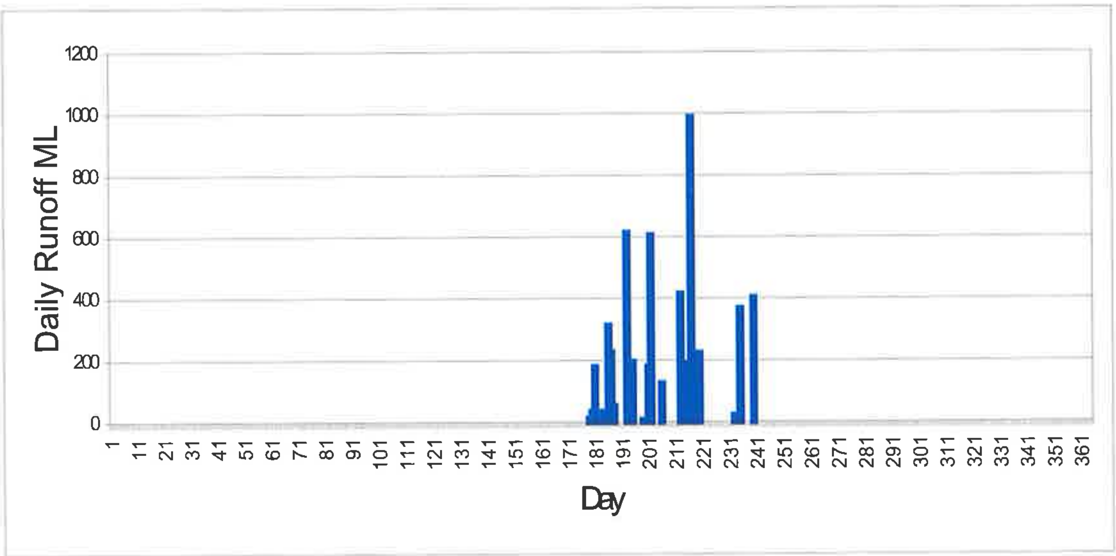


Figure 4.13 Total Daily Runoff (ML) for Pedler Creek in 1996

For 1996, the maximum flowrate for each runoff event is tabulated in Table 4.12.

To approximate the discharge volume that passes through the diversion every 900s (15 minutes), the hydrograph is divided into time increments of 900 seconds. For a time of concentration of 6 hours or 21600 seconds, the number of time increments is equal to 48 increments. By using the triangular hydrograph, the peak of the hydrograph corresponds to increment 24.

Table 4.12 1996 Maximum Flowrate for Runoff Events

Day	Runoff (mm)	Total Daily Volume (m^3)	Q(max) (m^3/sec)
179	0.2	9000	0.42
180	0.7	31500	1.46
181	3.9	175500	8.13
183	0.1	4500	0.21
184	0.7	31500	1.46
186	6.9	310500	14.38
187	5.0	225000	10.42
188	1.1	49500	2.29
193	13.6	612000	28.33
194	3.9	175500	8.13
195	4.3	193500	8.96
199	0.1	4500	0.21
201	3.9	175500	8.13
202	13.4	603000	27.92
206	2.7	121500	5.63
213	9.2	414000	19.17
214	4.2	189000	8.75
216	2.5	112500	5.21
217	21.9	985500	45.63
220	4.9	220500	10.21
233	0.4	18000	0.83
235	8.1	364500	16.0
240	8.9	400500	18.54

The flowrate for each time increment is given by:

$$Q_i = \text{slope} \times 900 \times i, \qquad i = 0, \ldots, 48 \tag{4.5}$$

with the slope of the triangular hydrograph given by:

$\text{Slope} = (Q_{(max)} - Q_0)/(T_c - T_0)$

$\text{Slope} = Q_{(max)}/(21600)$

For day 202 the maximum flowrate is 27.92 m^3/sec. Assuming the stream flows for 12 hours (twice Tc), the flowrate and volume for each 15-minute increment can be estimated. The results are tabulated in Table 4.13.

Table 4.13 Fifteen minute intervals of flowrate for day 202, 1996

Increment	Q(i) m^3/sec	Volume (i) m^3
0	0	0
1	1.16	1047
2	2.33	2094
3	3.49	3141
4	4.65	4188
5	5.82	5235
6	6.98	6282
7	8.14	7329
8	9.31	8376
9	10.47	9423
10	11.63	10470
11	12.80	11517
12	13.96	12564
13	15.12	13608
14	16.28	14652
15	17.45	15705
16	18.61	16749
17	19.77	17793
18	20.94	18846
19	22.10	19890
20	23.26	20934
21	24.43	21987
22	25.59	23031
23	26.75	24075
24	27.92	25128
25	26.75	24075
26	25.59	23031
27	24.43	21987
28	23.26	20934
29	22.10	19890
30	20.94	18846
31	19.77	17793
32	18.61	16749
33	17.45	15705
34	16.28	14652
35	15.12	13608
36	13.96	12564
37	12.80	11517
38	11.63	10470
39	10.47	9423
40	9.31	8376
41	8.14	7329
42	6.98	6282
43	5.82	5235
44	4.65	4188
45	3.49	3141
46	2.33	2094
47	1.16	1047
48	0	0
	Total	603000

Note: Volume(i) = Q_i x 15 minutes

By assuming the main stream channel is trapezoidal in cross-section, the depth for each flowrate in the stream can be determined using Manning's Equation (Equation 3.9). The bottom width of the Pedler Creek channel is 3.30 m with a side slope of 1 vertical to 2.3 horizontal, a longitudinal slope of 0.025 and maximum depth of creek of 1.5 m. Manning's n is set at 0.035.

By applying Manning's equation together with the estimated incremental stream discharge, a polynomial is derived in terms of the depth of the stream. The depth of the stream is determined using the secant method (Kreyszig, 1988).

4.8.2 Flow in Open Channel Off-take Diversion Structure

The depth of water passing into the open channel diversion (width 1m) is estimated for each of the time increments used in the triangular hydrograph generation. If the depth of water in the stream channel is greater than the height of the diversion channel above the creek bottom (150 mm), water will flow into the diversion channel.

For the purpose of this surface storage model, the open channel diversion structure is assumed to act as a rectangular broad crested weir as discussed in Section 3.4.2.

By applying Equation 4.6, the flowrate in the diversion channel for each time increment is calculated using the equation:

$$q = CLh^{3/2}, \qquad (4.6)$$

where:

$C = 2.0$,

L = width of weir (m) = 1m,

h = height of water over weir (m).

The volume of water entering the storage is calculated for each diversion channel incremental flowrate. Thc volume of water entering is limited by the maximum depth of the channel and hence the diversion.

Using a daily time-step the surface storage program calculates the total daily volume of water entering the storage through the diversion channel from streamflow in Pedler Creek. The estimated daily volume of water entering the storage via the diversion is given in Table 4.14. The model indicates on sixteen days, water was at a sufficient depth in the creek to flow into the diversion channel and into the storage. From Table 4.14 it can be seen that the total volume of water diverted is approximately 165ML. The total volume diverted from Pedler Creek into the storage for 1996-2000 is given in Table 4.15. For an annual flow of 5418ML/year for Pedler Creek north in 1996, approximately 3% of the flow passes through the diversion channel.

Table 4.14 Estimated Diversion Volume from Pedler Creek 1996

Day	Diversion Volume m^3
181	2580
186	9820
187	5116
193	24546
194	2580
195	3463
201	2580
202	24162
206	487
213	15313
214	3237
216	268
217	38268
220	4874
235	12740
240	14626
Total	**164660**

Table 4.15 Estimated Diversion Volume from Pedler Creek 1994-2000

Year	Diversion Volume ML/year
1994	67
1995	227
1996	165
1997	11
1998	116
1999	19
2000	211

4.8.3 Other Inputs/Outputs

The storage model is used to estimate the volume of water available for groundwater recharge. The initial volume of the case study storage on day 1 (1^{st} January) is set at 0.5 ML. Inputs included in the surface storage model are the rain falling directly into the storage and seepage of groundwater. Outputs included are the volume of water lost from the storage by evaporation and the volume of water pumped out for artificial recharge.

The volume contributed by rain falling directly into the storage is calculated by multiplying the maximum surface area of the storage by the daily rainfall. The surface area of the case study storage is $3060m^2$.

Groundwater seepage is estimated from the amount of seepage water pumped out over a given time period. Based on field data from 1998, groundwater seepage has been estimated to be 16.4 m^3 per day (Hunt, 1999). This is the amount pumped out to keep the storage at a constant level given no inflow from Pedler Creek. It is assumed that groundwater seepage is constant throughout the year, as the driving head is the sand hills located 500m north of the storage at an elevation of approximately 50 m above the storage.

Output from the storage includes spill; if the net inflow volume exceeds the capacity of the storage, excess water spills back into the creek (Figure 4.9).

The volume of water lost by evaporation is a function of the depth of water in the storage, as discussed in Section 3.4.3. The evaporation loss is calculated by multiplying the surface area of the storage by the evaporation rate. The maximum evaporation area occurs when the dam is at its maximum capacity, i.e. when the surface area is to $3060m^2$.

In the surface storage model, recharge to the aquifer starts at the end of the irrigation season for the Pedler Divide property, which is set to day 121 (1^{st} May). Providing the depth in the storage in greater than 1m, water is recharged for 24 hours per day at 3L/s for a set number of days or until the storage reaches 1m depth.

Three different recharge scenarios are evaluated, to determine the largest total recharge volume. After each recharge period a day with no recharge occurs to allow the aquifer to recover. Scenario 1 involves 24 hours of injection continuously for 4 days followed by 1 day of no injection. Scenario 2 involves 24 hours of injection continuously for 3 days followed by 1 day of no injection. Scenario 3 involves 24 hours of injection continuously for 2 days followed by 1 day of no injection. The total volume of water removed for each scenario for 1994 to 2000 is given in Table 4.16.

Scenario 1 produces the largest total recharge volume available for all years. From Table 4.16 the maximum volume available for recharge under scenario 1 is in the order of 24ML. 1997 was an unusually dry year and Table 4.16 reflects the available volume for recharge being only 5ML for all three scenarios. The average recharge under scenario 1 was 19ML/year.

The maximum volume available for recharge under scenarios 2 and 3 is 23 and 22 ML respectively. The average recharge under scenario 2 was 18 ML/year and the average under scenario 3 was 17 ML/year. The small decrease in maximum volume and average recharge from scenario 1 (4 days on, 1 day off) to scenario 3 (2 days on, 1 day off) suggests that the amount of water injected is primarily limited by stream discharge. Given that injection usually cannot be sustained for long periods of time because of clogging etc (see Section 2.6) it is more likely that scenario 3 can be sustained, with only a small decrease in injection.

Plots of the water level in the storage over time are given in Figure 4.13 and the total volume of water diverted and recharged is shown in Figure 4.14.

Table 4.16 Aquifer Recharge Volume based on Recharge Rate of 3 L/s.

Year	Scenario	Scenario Description	Total Number of Recharge Days	Total Recharge Volume (ML/a)
1994	1	4 days recharge 1 day off	75	19.4
1995	1	4 days recharge 1 day off	79	20.4
1996	1	4 days recharge 1 day off	83	21.5
1997	1	4 days recharge 1 day off	20	5.2
1998	1	4 days recharge 1 day off	90	23.3
1999	1	4 days recharge 1 day off	73	18.9
2000	1	4 days recharge 1 day off	92	23.8
1994	2	3 days recharge 1 days off	75	19.4
1995	2	3 days recharge 1 days off	77	19.9
1996	2	3 days recharge 1 days off	78	20.2
1997	2	3 days recharge 1 days off	20	5.2
1998	2	3 days recharge 1 days off	88	22.8
1999	2	3 days recharge 1 days off	71	18.4
2000	2	3 days recharge 1 days off	87	22.5
1994	3	2 days recharge 1days off	71	18.4
1995	3	2 days recharge 1days off	75	19.4
1996	3	2 days recharge 1days off	71	18.4
1997	3	2 days recharge 1days off	20	5.2
1998	3	2 days recharge 1days off	84	21.7
1999	3	2 days recharge 1days off	67	17.3
2000	3	2 days recharge 1days off	79	20.5

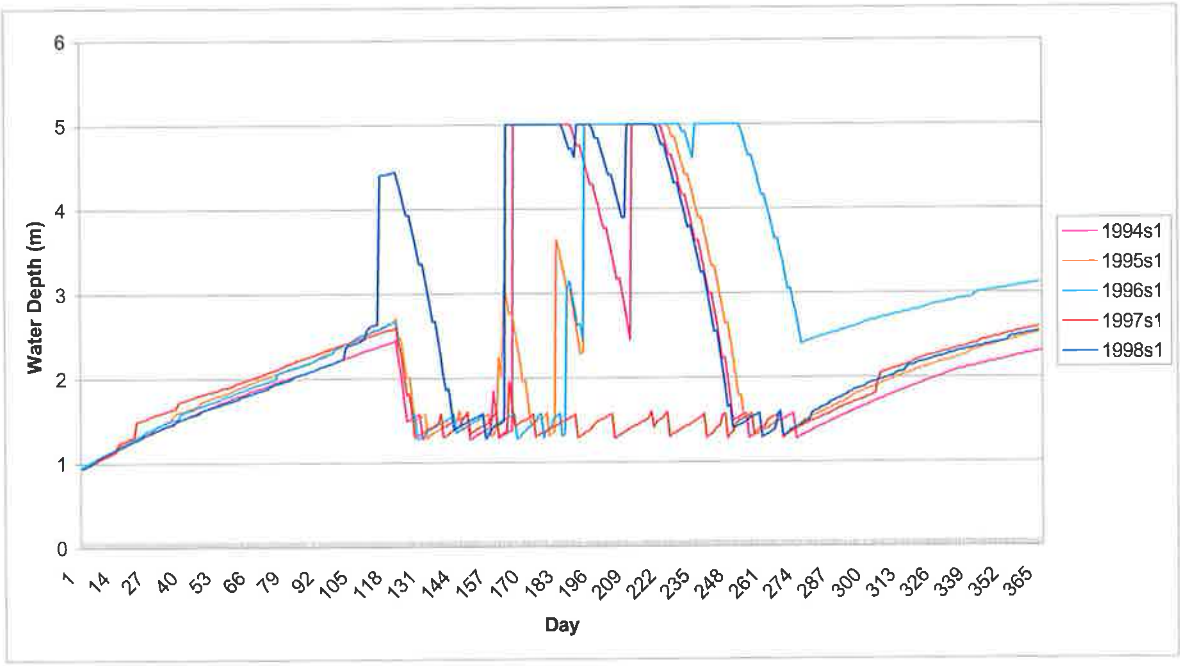


Figure 4.14 Daily Water Depth in Storage under Scenario 1

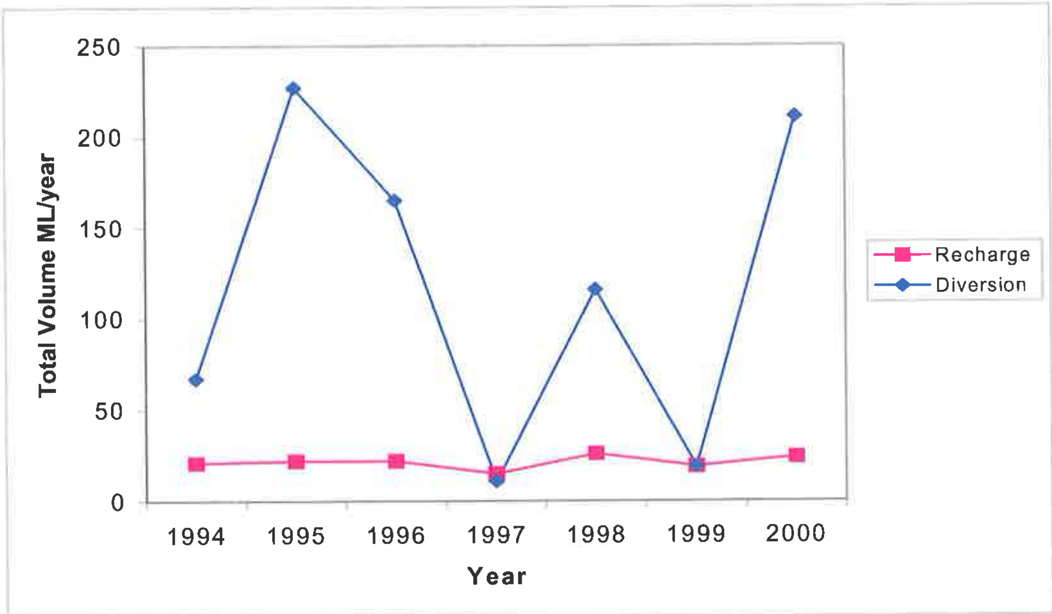


Figure 4.15 Total Recharge and Diversion Volumes for Scenario 1

4.9 Application of Groundwater Model to Pedler Divide

To assess the impact of injecting surface water into the groundwater system, a numerical model has been used. The numerical model is a regional groundwater flow model of the Willunga Basin (Rasser, 2000). For comparison an analytical solution provided by the Theis Equation as discussed in Section 3.5 is used.

4.9.1 Numerical Model

The Rasser (2000) model is a three dimensional regional model which uses a 500 m by 500 m grid spacing. The graphical output produced by the model represents an average groundwater response over each grid area, so that any localised variability in the potentiometric surface may be smoothed to a large extent.

The Willunga Basin Regional Groundwater Model considers the regional hydrogeology of the main aquifers in the area (Port Willunga Formation, Maslin Sands, and Basement aquifers), together with regional extraction, natural recharge and discharge to the sea.

Using Scenario 1 for 1995 (Section 4.8) together with the groundwater model, the effect of injecting water into the Basement aquifer within the groundwater system is evaluated. The locations that are evaluated using the groundwater model are: (1) the injection site on the Pedler Divide property, (2) 500 m west of the injection site and (3) 707 m south west of the injection site (Figure 4.16).

The effect of regional pumping is shown in Figures 4.17, 4.18 and 4.19 (from Rasser, 2000) with large drawdown occurring during the irrigation season. In Figure 4.17 at approximately day 335 there is a sharp decline in the potentiometric surface (which corresponds to the beginning of the irrigation season) to day 62 for both the Maslin Sands aquifer and the Basement aquifer which is the result of irrigation pumping. The decline in the potentiometric surface of the Maslin Sand aquifer over this period is approximately 2.8m and the Basement aquifer approximately 2.7m.

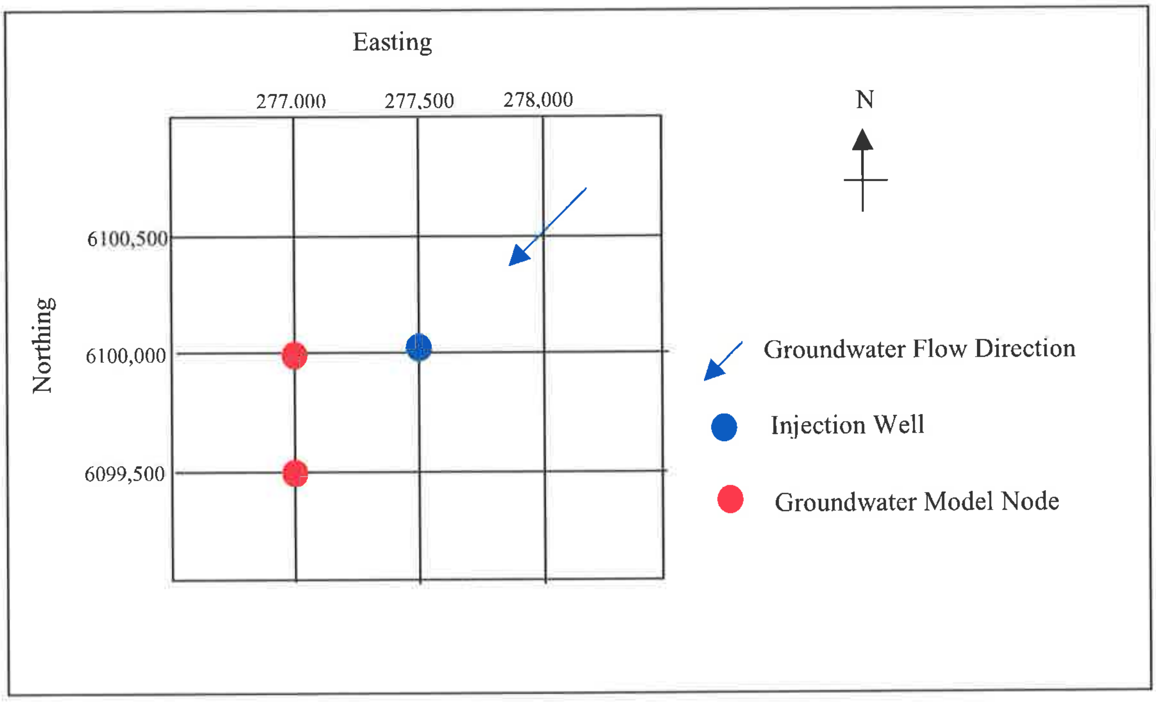


Figure 4.16 Location of the Nodes Analysed by the Groundwater Model

Figure 4.17 shows the response of the groundwater system to the injection of recharge water at the injection site. Over the four days of injection, the potentiometric surface in the Basement aquifer increases and then falls during the recovery day. From day 190 to day 260, there is a minimum increase in the potentiometric surface of 7cm and a maximum of 17cm. At the end of the recharge period (day 261), the potentiometric surface returns to the original non-injection potentiometric surface over a period of 40 days. The hydraulic connectivity between the Maslin Sands aquifer and the Basement aquifer is shown by a corresponding (although reduced) increase in the potentiometric surface in the Maslin Sands aquifer over the injection period. This suggests that some of the injected water may leak into the Maslin Sands aquifer as well as flowing within the Basement aquifer. Graphically, there is less evidence of recovery in the Maslin Sands potentiometric surface after each four-day injection period compared with the Basement aquifer.

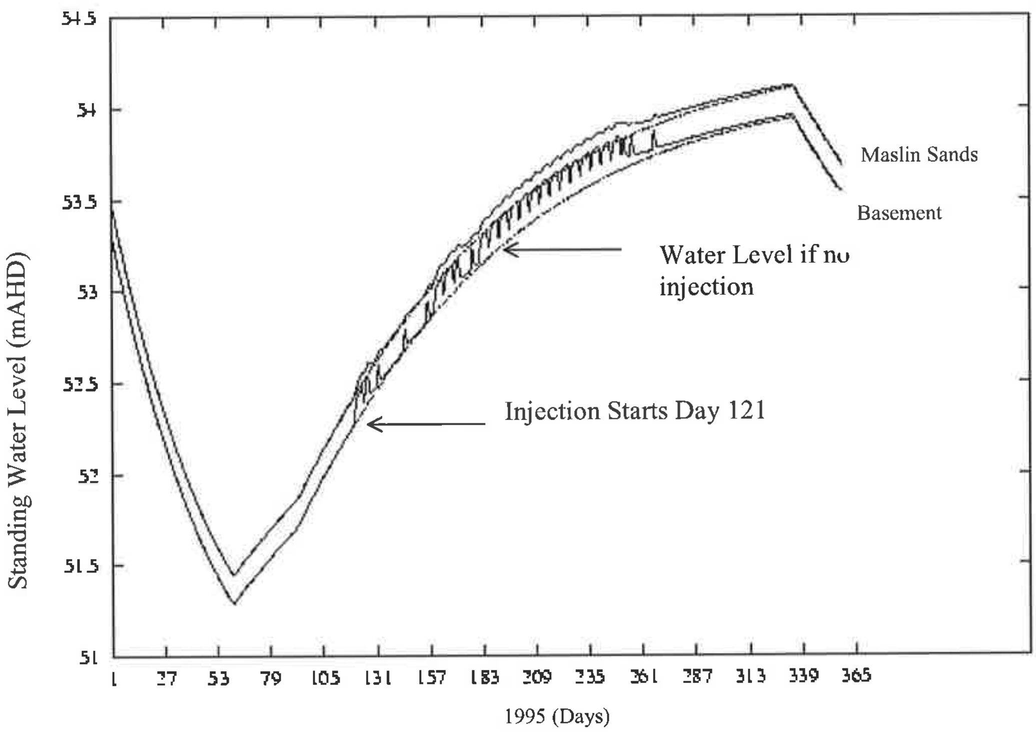


Figure 4.17 Groundwater Levels at Injection Site as a Result of Scenario 1, 1995, Injection. (data for Pt Willunga not provided)

Figure 4.18 shows the response of the groundwater system to the injection at a site 500 metres west of the injection site. There is a relatively small effect locally from the artificial injection 500m away, with a maximum increase in the Basement aquifer potentiometric surface of 11 cm and the Maslin Sands aquifer potentiometric surface of 5 cm. The Basement aquifer shows a similar response at this location as that at the injection site although the recovery after the fourth day of injection is not as sharp. There appears to be no effect on the potentiometric surface of the Port Willunga formation although there is a large decline of up to 6.5 m in the potentiometric surface due to irrigation pumping.

Figure 4.19 shows the response of the groundwater system 707 metres south west of the injection site. Similarly to Figure 4.17, there is a small increase of 5cm in the potentiometric surface of the Maslin Sands and Basement aquifers. Drawdown due to irrigation pumping is evident from Figures 4.18 and 4.19 with a maximum decline in the Port Willunga formation of 6.5 m, and declines of 2.6 m in the Maslin Sands and Basement aquifers.

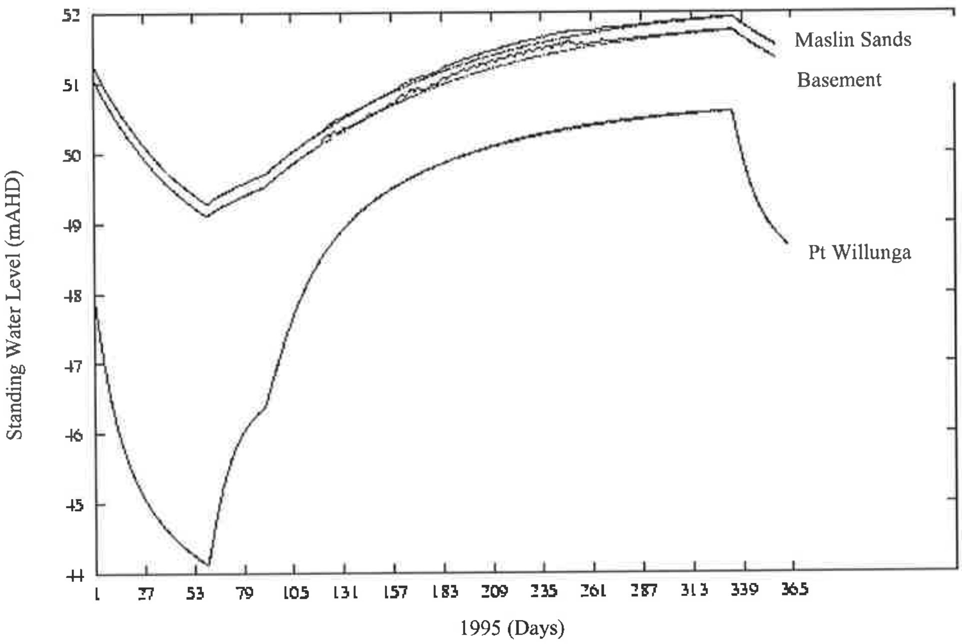


Figure 4.18 Groundwater Levels 500m West of Injection Site, as a Result of Scenario 1, 1995, Injection

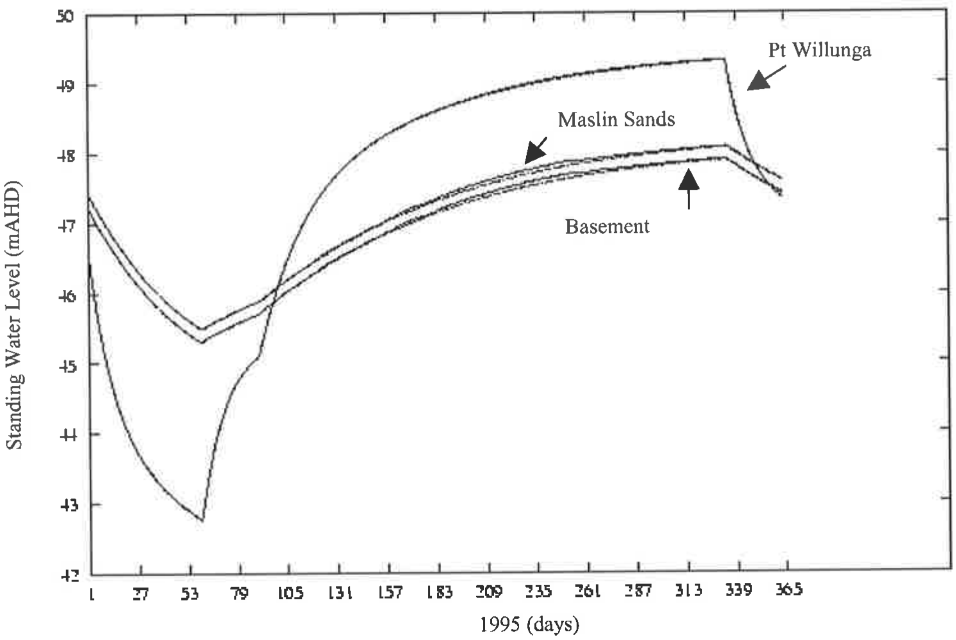


Figure 4.19 Groundwater Levels 707 m South East of the Injection Site, Scenario 1, 1995.

It is evident from Figures 4.17, 4.18 and 4.19 that the effects on the regional groundwater levels associated with the injection of surface water into the Basement aquifer are minimal. This would suggest that the water is absorbed into the groundwater system with little effect on the groundwater system and groundwater flow.

4.9.2 Analytical Solution

The Theis solution for drawdown at a well for unsteady radial flow in a confined aquifer was applied to the pumping well using equations (3.17) and (3.18). Storativity was set at 0.00015, transmissivity at 44 m^2/day (based on Rasser, 2000, for comparison with the regional model) and the injection rate at 259 m^3/day (3 L/s). The change in head over time is given in Figure 4.20. The greatest change in head occurs at 1 m from the injection well, compared with less than a 3m change in head 10m from the injection well. As expected the smallest change in head occurred at 1000m from the injection well.

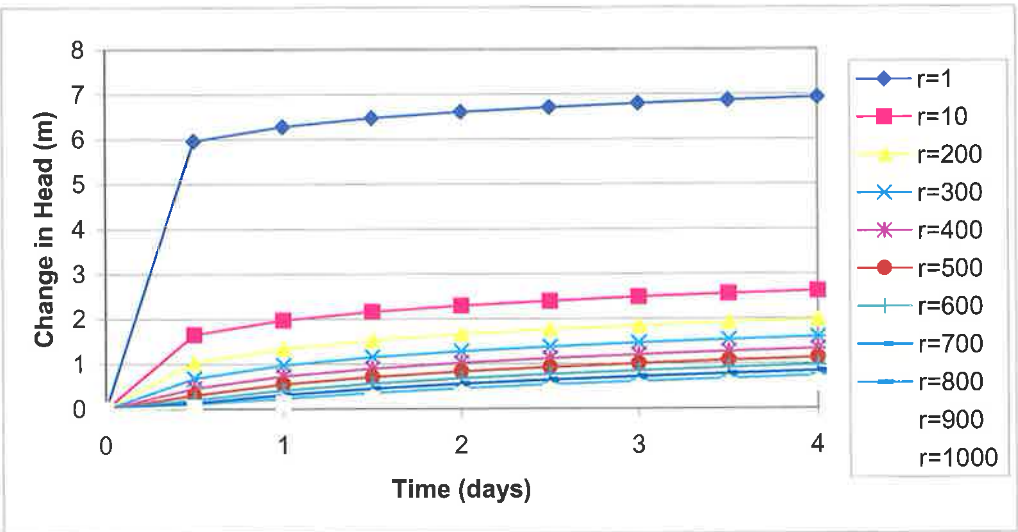


Figure 4.20 Plot of Change in Head versus Time at Various Distances from the Injection Well

These results suggest an increase in head of approximately 1m at distances 500m to 700m from the injection well, quite different from increase of the regional groundwater model results, which is in the order of 11cm at 500m and 5cm at 707m. However, the regional groundwater model calculates an average of the potentiometric surface 250m on either side of a node (Figure 4.16).

This analytical solution suggests that even though the increase in head very close to the injection well is somewhat significant, the head decreases quickly away from the well. In addition, the increase of 7m at a radial distance of 1m from the well at the end of 4 days injection under scenario 1 is not deemed significant to affect the groundwater system beyond the immediate vicinity of the well.

5 Conclusion

5.1 Conclusion

The aim of the research was to develop a generic water management model to identify the quantity of surface streamflow in an ungauged catchment potentially available for aquifer recharge. A surface storage model was developed to perform a daily water balance on an off-stream storage. This model was linked to a rainfall-runoff model, to estimate the volume of water available from the ungauged catchment. The volume available for ASR estimated from the surface storage model was entered into a groundwater model to ascertain the impact of recharge on the regional groundwater system.

A viticulture property within the ungauged Pedler Creek Catchment of the Willunga Basin was used to demonstrate the capabilities of the water management model and to quantify the potential surface water available for ASR from the catchment.

The average yield of 9700 ML/a has been estimated from the AWBM model for Pedler Creek, without considering the other four streams in the Willunga Basin. The estimated sustainable yield from groundwater for the Willunga Basin is 5700 ML/a, while the current average usage is estimated at 7380 ML/a (PIRSA, 1998). This suggests that if 1680 ML/a could be injected into the groundwater system through ASR, then a balance of extraction and recharge will have been reached for the Willunga Basin. If only 20% of the Pedler Creek flow estimated from the AWBM model (approx. 2000 ML/a) could be diverted to off-stream storage for use in ASR systems, an acceptable water balance may be achieved.

An attempt at calibrating the AWBM model was performed using streamflow and rainfall data for one year, 2000. The attempted calibration proved to be inconclusive, it was decided to run the model using the default values for SSC and partial area parameters. When streamflow data is available covering a number of years including at least one wet and a dry year, it is recommended that the AWBM model be calibrated and the total water balance model rerun to provide a more accurate estimation of water available for injection.

Application of the model to the case study area has shown that runoff from the Pedler Creek catchment is sufficient in most years to provide adequate quantities of water during winter for use as a water source for ASR. The average recharge volume estimated from Pedler Divide under various pumping scenarios is 18 ML/a. This recharge would easily satisfy the irrigation requirements for the viticulture property (approximately 13 ML/year based on 97/98 requirements (pers. comm. d'Arenberg Winery) as well as provide a surplus for the groundwater system.

Application of the regional groundwater flow model to Pedler Divide shows that the injection of surface water into the basement aquifer has little effect on the groundwater system and groundwater flow; this was also verified using an analytical Theis solution.

Groundwater in the Willunga Basin is currently being overexploited for the irrigation of vines and almonds. Using the water management model, together with a representative value of rainfall for part of the Pedler Creek north catchment, it has been shown that substantial water is potentially available for groundwater injection at the Pedler Divide site given the current infrastructure. At this time, an exhaustive examination of existing storages within the Pedler Creek catchment has not been conducted to determine the possible reduced volume of flow in Pedler Creek. It is recommended that this examination be performed as part of an extension to this work.

There is potential for capture of streamflow from Pedler Creek but further analysis is required to assess environmental needs, and requirements of downstream users. Further accurate streamflow data is required and is paramount to the calibration of the existing analysis.

5.2 Recommendations

The following recommendations are suggested as an outcome to this research to improve the estimation of determining the potential availability of surface water for artificial recharge.

General recommendations:

- Research into the construction of fixed structures at the inlet of the off-take diversion channel to overcome the problem of erosion of inlets of excavated earthen diversions. This ensures environmental flow requirements are satisfied.
- Application and calibration of the AWBM model in nearby gauged catchments will provide an improved estimation of the parameters.
- Apply the model developed in this study on a total catchment scale to analyse the likely impact on users downstream.

Specific recommendations:

- Calibration of the rainfall-runoff model using at least two years of streamflow data to obtain a more accurate estimation of its parameters.
- Daily data collection of dam depth would provide a more accurate estimation of the seepage into the dam.
- Further groundwater modelling using a smaller grid spacing of 100m by 100 m is required to assess the local movement of groundwater from the injection site.
- Inclusion of the impact of farm dams in the upper reaches of the catchment and the effect they have on streamflow.
- Accurate rainfall data is paramount in estimating runoff volumes. Increased spatial coverage of rainfall stations and evaporation stations within the catchment to assess the spatial variable of data.

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ADDENDUM

Page 37 below paragraph 2

MacDonald and Baker (1986) investigated a number of catchments in the Mt Lofty Ranges in an attempt to derive an empirical relationship for estimating time of concentration (Tc, hours). The formula they developed, based on catchment area (a, km^2), was $Tc= 1.0\ a^{0.49}$. However the authors felt that the basis for this relationship did not warrant its adoption over the previously accepted formula documented in the Australian Rainfall Runoff Guide (Pilgrim, 1987). Because of this, equation $Tc = 0.5a^{(2/3)}$ is used to determine the time of concentration.

Page 53

Figure 4.2 was created by the author using 2000 spatial data from the Department for Water Resources spatial database.

Page 54

The caption for Figure 4.3 should appear below the figure not above as shown.

Page 107 Add to reference list:

MacDonald, P. M. and Baker, T. Derivation of an Empirical Equation for the Estimation of the Time of Rise for Small Rural Catchments in South Australia. E&WS Report 6178/84. 1986.