# Optimal Design of Stormwater Wetlands

by

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Master of Engineering Science
in
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# Statement of Originality

This thesis contains no material that has been accepted for the award of any other degree or diploma at any other tertiary institution and, to the best of my knowledge and belief, this thesis contains no material previously published or written by another person, except where due reference is made in the thesis text.

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# Chapter 1 Introduction

## 1.1 Need for Research

Until recently urban stormwater wetlands were constructed with one function or objective in mind. In general, this was either flood mitigation or water quality improvement. Increasingly they are now being constructed to perform a range of different functions including: -

- Flood mitigation
- Water quality improvement
- Stormwater harvesting for reuse
- Providing habitats for flora and fauna
- Recreation
- Public education and research

Each wetland function has specific physical, biological and hydraulic design requirements.

Section 1.1, page 1, following paragraph 2, add:

In this thesis the term wetland is used loosely to refer to a combination of any number of detention basins and/or macrophyte ponds linked together or a solitary basin or pond.

## 1.2 Objectives of Research

The objective of this research is to examine the use of genetic algorithms to aid in the design of multi-functional constructed stormwater wetlands. The wetland functions included in the analysis are flood mitigation, pollutant removal and stormwater harvesting for re-use.

The research aims to produce an optimisation framework that can be used by government authorities and engineering consultants to design more efficient wetlands. Using this framework, it is proposed to determine an optimal wetland design for a given set of functional design requirements. An investigation into the effect on the optimal wetland design, of altering some of the design requirements will also be carried out. Changing some of the design requirements will enable an examination of the trade-offs that occur when designing multi-functional wetlands.

## 1.3 Methodology

The methodology adopted involves the use of a genetic algorithm to optimise the physical design characteristics of a constructed stormwater wetland. The characteristics are optimised based on their ability to satisfy the prescribed functional requirements at minimum cost.

The optimisation framework used involves the integration of a flood simulation model and a long-term simulation model within a simple genetic algorithm. The two simulation models are used to assess the performance of a particular wetland design in satisfying the functional design requirements.

## 1.4 Structure of Thesis

A review of current literature concerning urban stormwater wetland design is presented in Chapter 2. Specifically, literature related to the design of wetlands for flood mitigation, pollutant removal and stormwater harvesting for reuse is reviewed. A brief discussion on groundwater and aquifer storage and recovery is also included.

Chapter 3 contains a brief outline of the operation of a simple genetic algorithm. The various forms of the three important operators, (selection, crossover and mutation) are presented.

The description of the optimal wetland design framework used in this research is contained in Chapter 4. The chapter also includes a review of literature concerning the optimisation of stormwater detention basins.

The South Parklands case study is introduced in Chapter 5. A detailed description of the proposed wetland site, the catchment, and previous studies is given. The selection and derivation of the water quantity, water quality models and cost information required by the framework is discussed.

Chapter 6 presents the results from the optimisation runs conducted for the case study site. A description of the methods used to obtain these results is given. The results are presented in two sections. In the first section, results from the numerous optimisation runs, conducted to examine the multi-functional trade-offs, are presented as contour plots. Three wetland designs are then selected. Results from the flood and long-term simulation models conducted on these designs are presented in the second section.

Conclusions and recommendations from the research are presented in Chapter 7. Results from the case study are summarised and recommendations for changes to existing design practices as well as possible future research paths are outlined.

# Chapter 2 Urban Stormwater Wetland Design

## 2.1 Introduction

The design requirements for the three wetland functions examined in this thesis are different, consequently the procedures used in their design are also different.

A detailed description of technical information about the planning, design, construction and operation of constructed wetlands for a range of applications is contained in DLWC (1998).

This chapter reviews current procedures used for designing wetlands for pollutant removal, flood mitigation and water harvesting for reuse.

## 2.2 Wetland Pollutant Removal

The wetland processes related to the removal of pollutants in stormwater wetlands are complex. Consequently, there is a significant amount of literature on pollutant removal related subject areas.

Many processes affect a wetland's ability to remove pollutants from stormwater inflows. Physical characteristics of the wetland such as the basin shape, basin size and the outlet structure configuration affect the location of flow paths and velocities, which have a major influence on pollutant removal performance. Biological characteristics such as the sizes, types and locations of macrophyte zones affect the efficiency of plant related pollutant removal processes. The quantity and water quality of inflow events will also affect a wetland's pollutant removal efficiency.

The design of stormwater wetlands for pollutant removal involves, firstly, investigating the pollutant and hydrological characteristics of the source water at the proposed site. Design constraints such as available space, location, climate and funding are also taken into account before establishing the wetland's pollutant removal objectives (ie. the pollutant types and removal efficiency). Once all the objectives and constraints have been identified the appropriate wetland design can be determined using a suitable design procedure.

This section contains some background information on stormwater pollutants and pollutant removal processes and procedures identified in the literature.

### 2.2.1 Major Stormwater Pollutants

2.2.1 a) Suspended Solids

Section 2.2.1 a), page 5, sentence 3, change:

Many contaminants, including heavy metals, nutrients and pathogenic microorganisms, adhere to suspended sediments.

To:

Many contaminants, including heavy metals, nutrients and pathogenic microorganisms, are adsorbed onto fine particles that are transported in suspension.

2.2.1 b) Nutrients

The nutrients that are of major concern to water quality that are usually present in high concentrations in stormwater are nitrogen and phosphorus. High nutrient concentrations in receiving waters can cause excessive algal growth, which has an adverse impact on the aquatic ecosystem and human health. Algal blooms reduce light transmission and cause unpleasant odours. Some species of marine algae contain toxins that can be harmful to humans ingesting affected fish. The die-off and decay of large amounts of algae can cause depletion of dissolved oxygen in the water column (Kent, 1994).

Nitrogen occurs in stormwater in a variety of different forms, including nitrate, nitrite, ammonia and organic nitrogen. It can readily change from one form to another depending on the physical and biochemical conditions present. Soluble forms of nitrogen typically constitute 50% of the total nitrogen (BCHF, 1992).

Phosphorus is usually present in runoff in two forms, organic phosphorus and orthophosphate ($PO_4^{-3}$) (Mays, 1996). Typically between 30% and 60% of the total phosphorus present in stormwater is soluble (BCHF, 1992).

Sources of nutrients in urban catchments include organic matter, fertilisers, sewer overflows/septic tank leaks, animal/bird faeces, detergents, and spillage/illegal discharge (NSW EPA, 1997a).

### 2.2.1 c) Heavy Metals

A variety of trace metals are carried in stormwater, both in soluble and insoluble forms. The metals that are of most concern to plant and animal health and typically present in high concentrations are chromium, copper, lead and zinc. These metals can be toxic to aquatic organisms and can poison humans who eat any affected fish or shellfish (Kent, 1994). Sources of trace metals include vehicle wear, sewer overflows/septic tank leaks, weathering of buildings/structures and spillage/illegal discharge (NSW EPA, 1997a).

### 2.2.1 d) Oxygen Demanding Organics

Oxygen demanding organics carried in stormwater include naturally occurring material such as soil and plant detritus and man-made materials such as oil and greases. These pollutants are referred to as oxygen demanding organics and contribute to the biological oxygen demand (BOD) because they promote the growth of bacteria that consume oxygen present in the water column. Sources of oxygen demanding organics include the decay of organic matter, animal bird faeces, sewer overflows/septic tank leaks, leaks from vehicles, car washing and spillage/illegal discharges (NSW EPA, 1997a).

### 2.2.1 e) Toxic Organics

Toxic organics comprise synthetic compounds such as industrial chemicals, pesticides, plasticizers and hydrocarbons (Mays, 1996). These compounds can be harmful to aquatic ecosystems and once they enter the food chain, they can affect other animals. Sources of toxic organics include pesticides, herbicides, spillage/illegal discharge and sewer overflows/septic tank leaks (NSW EPA, 1997a).

2.2.1 f) Micro-Organisms

The principal microorganisms of concern in stormwater include bacteria, algae, protozoa and viruses. Many of these pathogenic microorganisms carry diseases that can be harmful to humans (Mays, 1996). Sources of microorganisms include animal/bird faeces, sewer overflows/septic tank leaks and organic matter decay (NSW EPA, 1997a).

### 2.2.2 Typical Pollutant Concentrations

The concentrations of pollutants in stormwater runoff are highly variable, from catchment to catchment and from storm event to storm event. Due to the high degree of variability it is very difficult to accurately estimate stormwater quality. Many summary studies have been conducted into stormwater quality the results from a few of them are given in Table (2-1).

**Table 2-1 Urban Stormwater Quality**

| Pollutant | NSW EPA dry weather - instantaneous concentrations | NSW EPA wet weather – event mean concentrations | SPCC (1987) | Metcalf & Eddy (1991) |
|---|---|---|---|---|
| **Suspended solids (mg/L)** | 1 – 350 | 20 – 1000 | 150 – 650 | 141 - 224 |
| **Nutrients (mg/L)** | | | | |
| Total Phosphorus | 0.001 – 2.2 | 0.12 – 1.6 | 0.1 – 1.5 | 0.37 - 0.47 |
| Total Nitrogen | 0.1 – 11.6 | 0.6 – 8.6 | 0.5 – 3.0 | 3 - 24- |
| Oxidised Nitrogen | - | 0.07 – 2.8 | | - |
| Ammonia | - | 0.01 – 9.8 | | - |
| **Faecal coliforms (cfu/100mL)** | 40 – 40,000 | 4,000 – 200,000 | 1,000 – 1,000,000 | 1,000 – 21,000 |
| **BOD (mg/l)** | - | - | 10 - 60 | 10 –13 |
| **Trace metals (mg/L)** | | | | |
| Cadmium | - | 0.01 – 0.09 | 0.006 | - |
| Chromium | - | 0.006 – 0.025 | 0.17 | - |
| Copper | - | 0.027 – 0.094 | 0.04 | - |
| Lead | - | 0.19 – 0.53 | 0.2 | 0.161 - 0.204 |
| Nickel | - | 0.014 – 0.025 | - | - |
| Zinc | - | 0.27 – 1.10 | 0.2 | - |

Duncan (1997) conducted a statistical overview of urban stormwater quality using data from many water quality investigations. He discovered that urban zoning (residential, industrial, etc.) has little effect on runoff quality. The greatest influence was found to be actual land use (road, roof, etc.).

There are many models in the literature that attempt to predict the washoff of pollutants from urban catchments (Huber & Dickinson, 1988; Sivakumar & Boroumand-Nasab, 1995; XP

**Table 2-2 Settling Velocity Distribution of Urban Sediment (Source Based on NURP Studies; USEPA 1986)**

| Mean settling velocity (m/hr) | Equivalent diameter (μm) * |
|---|---|
| 0.009 | 2 |
| 0.091 | 5 |
| 0.46 | 12 |
| 2.1 | 35 |
| 20 | 82 |

* Calculated from settling velocity, assuming spherical particles with specific gravity 2.68 and water at 20°C.

Sedimentation of very fine particles (diameter < 5 μm), referred to as colloids, is dependent on their coagulation behaviour. Settling rates for particles of this size are influenced by slight movements in the water column. Walker (1995) found those suspended particles with particle diameters less than 15 μm required horizontal flow velocities less than 0.00013 m/s in order to settle out of the water column. The coagulation of suspended particles is influenced by their surface charge, the density of particles, the amount of water agitation and the ionic composition of the water (Lawrence & Breen, 1998).

Physical re-suspension of settled particles may occur if a wetland is subject to high horizontal velocities. The velocity required to tear a particle loose from the sediment bed is dependent on the particle's size and the characteristics of the wetland. Wetland plants stabilize wetland soils and sediments reducing the amount of re-suspension. There are three other re-suspension mechanisms that occur in wetlands: wind-driven turbulence, bioturbation, and gas lift (Kadlec & Knight, 1996).

### 2.2.3 b) Filtration 过滤

Filtration involves the interception of suspended particles by wetland vegetation. The extent of the role that filtration plays has not been quantified, however several researchers have observed particle coatings on plant surfaces. Wetland characteristics that increase the mass of particles removed from the water column through filtration include, uniform flow velocities and the presence of dense, uniformly distributed vegetation in the flow path (Lawrence & Breen, 1998).

2.2.3 c) Adsorption

Dissolved nutrients and metals can be adsorbed onto sediment particles by various physical and chemical processes. Once adsorbed the previously dissolved pollutants can be removed from the water column via sedimentation and filtration.

2.2.3 d) Plant Uptake

Wetland plants play an important role in removing dissolved and colloidal nutrients from the water column. Colloidal nutrients are adsorbed by the benthic biofilm and transferred to the sediments. The benthic biofilm is a gelatinous sheath of algae and polysaccharides located on the bed or substratum of the pond. Dissolved nutrients are taken up by algae attached to both the pond bed and plants (Lawrence & Breen, 1998).

The benthic and epiphytic biofilm can be flushed out of the pond if the pond is subject to flow velocities greater than 0.05 m/s (Lawrence & Breen, 1998). Algae flushed out of the pond can cause a significant increase in BOD and a potential source of nutrients, downstream.

### 2.2.4 Wetland Water Quality Models

There has been a considerable amount of research conducted into wetland water quality processes. Much of this research has focused on the efficiency of wetlands in removing one or more key pollutants from the inflow water. Pollutant removal efficiency has been adopted by several authors as the most important indicator to the ability of a wetland to perform effectively as a pollutant removal device. Pollutant removal efficiency for a particular pollutant is defined as

$$\text{Pollutant Removal Efficiency} = \left(1 - \frac{\text{average output concentration}}{\text{average input concentration}}\right) \times 100 \qquad (2\text{-}2)$$

The pollutant removal efficiency of a wetland is generally determined through water quality monitoring of inflow and outflow concentrations for one or more storm events. Results from water quality monitoring indicate that pollutant removal efficiency can vary significantly between different wetlands and between storm events for an individual wetland (Murphy et al, 1998; Wu et al, 1996; Pettersson, 1998; Martin, 1988).

Many factors influence wetland pollutant removal efficiency including pollutant inflow concentration, wetland size and shape, inlet and outlet configuration, climate, and plant and soil types. The water quality sampling technique, used during monitoring, will also affect the accuracy of the estimate. Since these characteristics are different for each wetland and some are hard to quantify, it is difficult to formulate a general model for pollutant removal that could be applied to all sites. Several authors have attempted to derive generic pollutant removal curves based on results from sampling conducted on existing wetlands (Lawrence, 1986; Tomlinson, 1993; DLWC, 1998). These curves related wetland retention time (days) to percentage pollutant removal for several key pollutants. The most recent of these, DLWC (1998), takes into account the inherent variability of pollutant removal in wetlands by relating retention time to a range of removal percentages. These curves were formulated for suspended solids (Figure 2-1), phosphorus (Figure 2-2) and nitrogen (Figure 2-3).

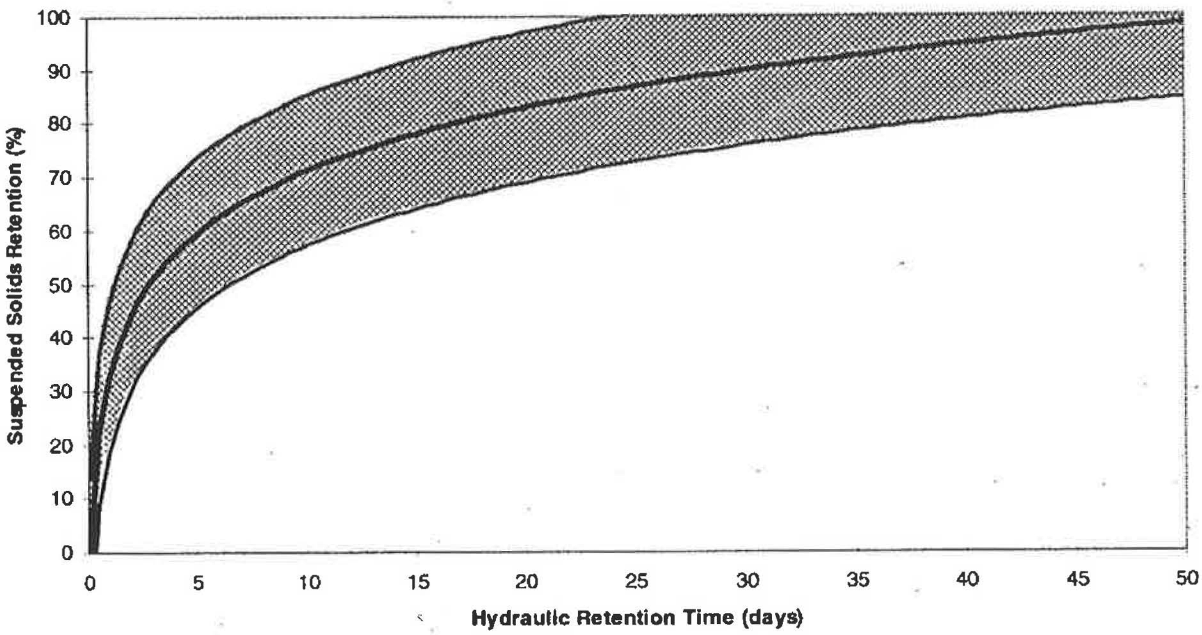


**Figure 2-1 Suspended Solids Generic Pollutant Removal Curve (Source: DLWC, 1998)**

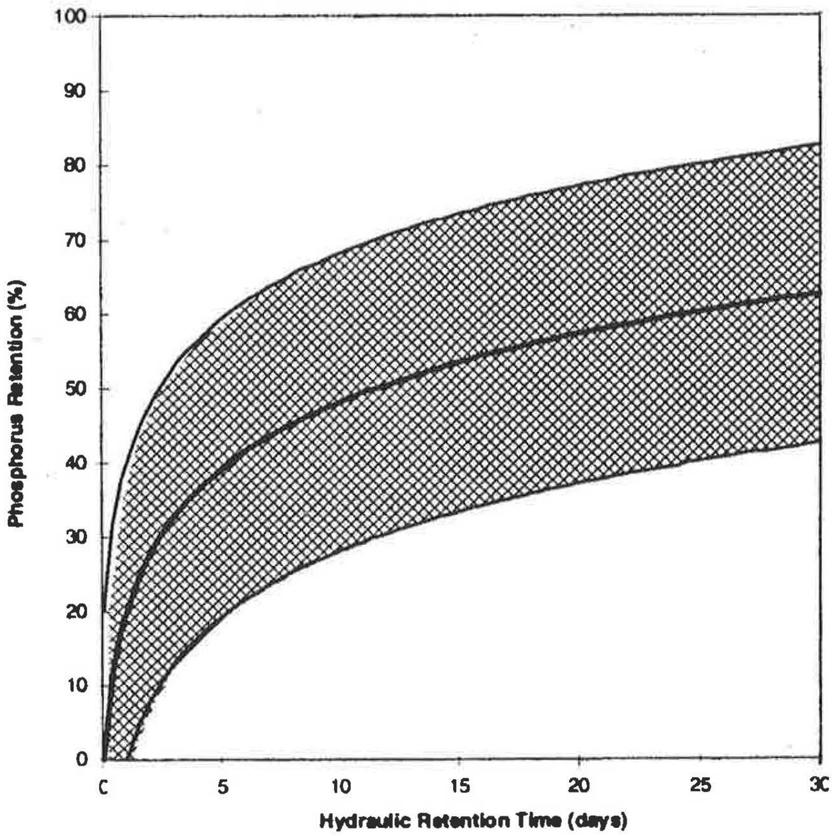


Figure 2-2 Phosphorus Generic Pollutant Removal Curve (Source: DLWC, 1998)

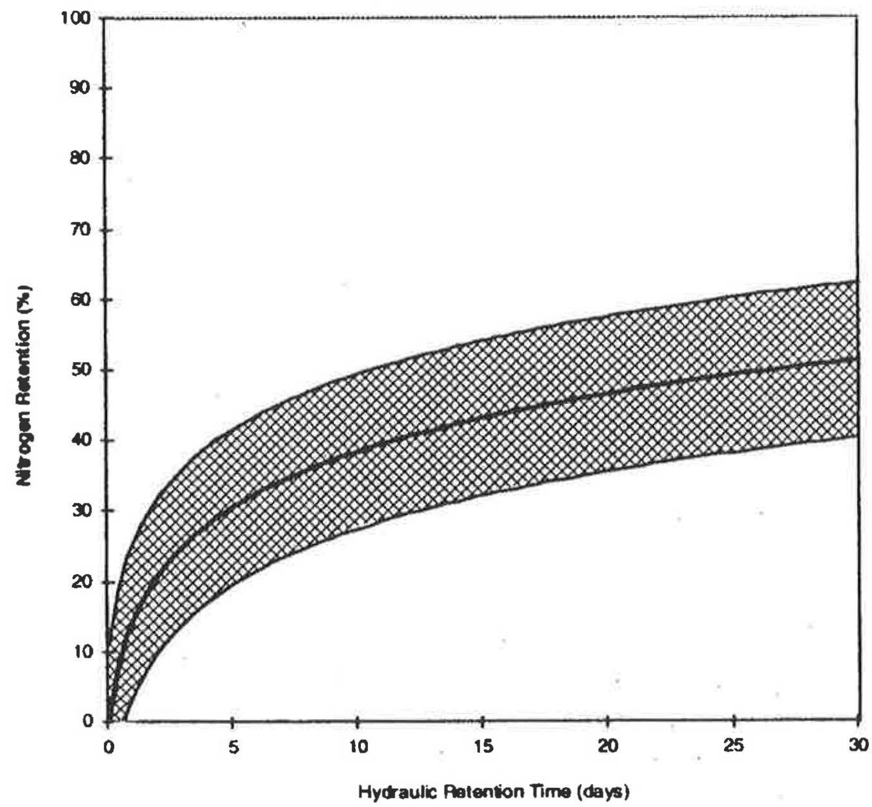


Figure 2-3 Nitrogen Generic Pollutant Removal Curve (Source: DLWC, 1998)

Models applied to the simulation of pollutant processes in stormwater wetlands can be grouped into three categories; sediment-settling models, empirically derived pollutant removal models and detailed water quality simulation models.

Sediment settling models assume that the dominant pollutant removal process is sedimentation. Pollutant removal is based solely on sediment particles settling out of the water column. The rate of removal is dependent on the settling velocities of the particles present.

An example of a sediment settling model is the US EPA design procedure (USEPA, 1986). The US EPA procedure includes models for sedimentation under dynamic and quiescent conditions. Dynamic removal is calculated by

$$R_d = 1.0 - \left(1.0 + \frac{1}{n}.\frac{V_s}{\frac{Q}{A}}\right)^{-n} \tag{2-3}$$

| | | | |
|---|---|---|---|
| Where | $R_d$ | = | fraction of solids removed under short term dynamic conditions |
| | $V_s$ | = | settling velocity |
| | $Q$ | = | mean flow rate |
| | $A$ | = | surface area of detention pond |
| | $n$ | = | turbulence constant |

The procedure performs separate calculations for five different particle sizes at the mean flow rate. The settling velocities for each particle size were calculated from experiments (see Table 2-2). The turbulence constant (n) is a measure of the hydraulic efficiency of the pond, ranging from one to infinity. The higher the value of n the better the ponds hydraulic efficiency.

Quiescent removal operates on the volume of water remaining in the pond following outflow, and is calculated by

$$R_q = V_s.A \tag{2-4}$$

| | | | |
|---|---|---|---|
| Where | $R_q$ | = | fraction of solids removed under quiescent conditions |

As with dynamic removal this equation is applied to the five representative particle sizes.

Sediment settling models have been applied primarily to detention basins, where the assumption that pollutant removal only occurs via sedimentation is more valid. The US EPA procedure, however has been applied successfully to the Happy Valley wetland in South Australia (Walker, 1994).

There is a substantial amount of data available on the pollutant removal efficiency of wastewater wetlands. A number of authors have developed simple empirical pollutant removal models for a range of pollutants, using the available data (Reed et al, 1985; Kadlec & Knight, 1996). These relationships assume that the flow rate is constant and relate the pollutant removal percentage to the hydraulic retention time of the wetland. These models work well when applied to wastewater wetlands due to the steady flow conditions, however their applicability is limited for stormwater wetlands. Flow through stormwater wetlands is intermittent and it is inaccurate to assume a constant flow rate.

Kadlec & Knight (1996) developed a two parameter model used for predicting pollutant removal in wastewater wetlands known as the 'k-C*' model.

$$\frac{C_o - C^*}{C_i - C^*} = e^{-k/q} \qquad (2\text{-}5)$$

| | | | |
|---|---|---|---|
| Where | $C_o$ | = | outflow concentration |
| | $C_i$ | = | inflow concentration |
| | q | = | hydraulic loading rate (m/yr) |
| | C* | = | background concentration |
| | k | = | rate constant |

The parameters k and C*, determined empirically, differ depending on the pollutant being modelled. Wong & Geiger (1997) adapted this model for stormwater wetlands by substituting q in Equation (2-5) with an equivalent steady flow rate ($Q_{esf}$). It was suggested that the equivalent steady flow rate should be calculated by continuous simulation or as the ratio of the volume of the wetland to the pollutant detention period.

Water quality simulation models attempt to simulate both the removal of pollutants from the water column and pollutant transfers between the major components of the wetland. There are a number of water quality simulation models in the literature, differing both in their derivation and application (Shih-Long Liao et al, 1998; Nnadi & Addasi, 1999; Lawrence & Breen , 1998). These models often require a substantial amount of water quality, water quantity and climate data to function and consequently their application to some wetland systems may be limited.

The model developed at the CRC for Freshwater Ecology models a number of water quality related wetland processes, including adsorption, sedimentation, sediment reduction and oxidation, algal growth and oxygen transfer. The model assumes that the wetland is a continuously stirred tank reactor (CSTR) when calculating mass inflows and outflows from the wetland. CSTR assumes that complete mixing occurs between water flowing into and water stored in the wetland. The model comprises algorithms describing the water quality processes within the wetland. These algorithms were calibrated using observed water quality data from Canberra wetlands and validated utilising data sets from ponds located in different catchment soil, climate and loading conditions.

The VASWETS model developed at the University of Virginia in the USA is a theoretical based model (Shih-Long Liao, 1998). The model determines pollutant concentration changes by modeling five transport mechanisms, diffusion, settling/sedimentation, sorption/filtration (to both plant species and substratum) and plant species uptake. Experiments were conducted on four 15 L plastic bucket wetlands filled with gravel and different wetland plants. Water quality parameters were monitored both in the water column and substratum over a 21 day period and the results were compared with the theoretical results obtained from the model. Results from the model compared favourably with data collected from the experiments.

### 2.2.5 Retention Time Calculation

Residence time is defined as "the time period for a particle to flow from the pond inlet to the pond outlet" (Martin, 1988). It is a very important factor in water quality improvement as pollutant removal generally improves as residence time increases. The residence time of a wetland is influenced by internal hydrodynamic factors, including basin bathymetry, spatial variability of vegetation and inlet and outlet conditions, and external hydrodynamic factors such as wind shear and hydraulic loading rate (Somes et al, 1997).

The ideal residence pattern in most treatment wetlands is plug flow. Plug flow is often assumed in simulation models of wastewater wetlands and has been used in simulation of stormwater wetlands (Walker & Murphy, 1997). Plug flow occurs when there is no mixing between flow volumes, and flows passing through the pond displace water deposited by earlier events. Plug flow is a theoretical concept; the actual residence patterns or residence time distributions of constructed wetlands are not ideal. Another technique often used in wetland simulation for residence time calculation is complete mixing or the CSTR approach. Average retention times calculated using the CSTR approach are lower than those calculated using plug flow (Tye & Dandy, 1998). Lawrence & Breen (1998) stated that, based on field validation, only in a small number of cases is there any deviation from the CSTR assumption.

Mean residence time is often used in determining the pollutant removal effectiveness of constructed wetlands. It is defined as follows:

$$T = \frac{V}{q} \tag{2-6}$$

| | | | |
|---|---|---|---|
| Where | T | = | mean residence time |
| | V | = | basin volume |
| | q | = | flowrate |

The flow rate used could be the annual total flow volume ($m^3$/year) or a mean event flowrate ($m^3$/s). Mean residence time is a good indicator to the actual residence time for wastewater treatment wetlands where flowrates are relatively constant. Flowrates in stormwater wetlands however are highly variable and the mean residence time is not a good indicator of actual residence time (Walker, 1996a). Walker suggested a better statistic might be $T_{50}$ or $T_{90}$, the residence time achieved for 50% and 90% respectively, of the flows passing through the wetland in a particular period.

Another method often used to examine the residence time of outflows from a wetland is to examine the residence time distribution (RTD). The residence time of all outflows during a single flood event or over a series of flood events, are plotted on a histogram. Using this method allows the full distribution of residence times to be investigated.

### 2.2.6 Treatment Train Selection

The selection of the appropriate treatment train is usually the first step in designing a water treatment or wastewater treatment plant. It should also be considered when designing a stormwater wetland. The hydrological characteristics of the site, the inflow pollutant levels, the intended functions of the wetland and the available space, have to be considered in the selection. Figure (2-4) shows a number of possible arrangements of gross pollutant traps, detention ponds and wetlands. Gross pollutant traps are essential in urban areas to prevent organic material and litter from impairing the effectiveness of ponds and wetlands. Detention ponds are used for peak flow reduction of high flows and water quality improvement through sedimentation. Detention ponds, contain little vegetation, and hence can cope with high storm discharges. They are usually constructed on-stream. Wetlands however are generally well vegetated and need to be protected from large flows. They perform pollutant removal through sedimentation, adsorption, filtration and plant uptake.

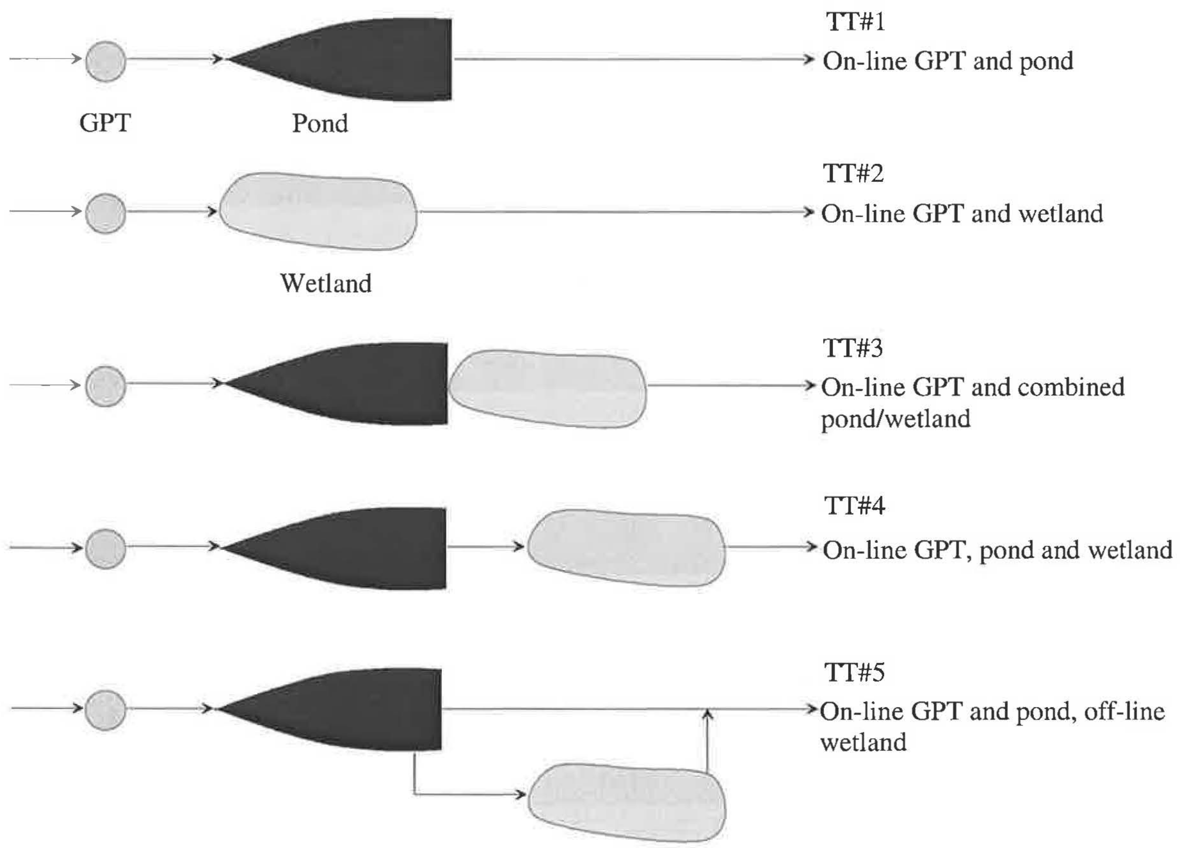


**Figure 2-4 Treatment Train Options (Source: Lawrence & Breen, 1998)**

(TT# = Treatment train number; GPT = Gross pollutant trap)

The first treatment train in Figure (2-4) is suitable for sites that experience high storm discharges that are high in suspended solids. The functional objectives for this system would be flood mitigation and pollutant removal via sedimentation. Treatment train two would be chosen for low flow conditions where the majority of pollutants are in dissolved or colloidal form. Pollutant removal via adsorption, filtration, coagulation and plant uptake would be the primary function of the wetland. Treatment trains three and four are suitable for moderate to high flow conditions where pollutants occur in all forms and it is desirable to treat all stormwater flows. For these cases, construction of a wetland on-stream is feasible as it is protected from high discharges. Treatment train three incorporates the pond and wetland into a single facility. Treatment train five should be selected if it is not possible to reduce flows into the wetland sufficiently or it is not necessary to treat all flows with the wetland. A diversion structure diverts some flows from the detention pond to the off-stream wetland.

### 2.2.7 Pond Sizing

The methods used for sizing stormwater wetlands are generally quite simple. DLWC (1998) presents a summary of these methods.

The catchment area method involves sizing the surface area of the wetland to be 2% of the total catchment area. The 2% figure was found to be a critical figure in an analysis conducted in the USA. Wetlands with surface areas greater than 2% of the catchment area were found to perform significantly better than wetlands with surface areas less than 2%.

Somes and Wong (1998b) conducted continuous simulations of wetland storage behaviour of variously sized storages in seven Australian capital cities. Statistically derived hourly rainfall data were used in the simulations. Results from the simulations were used to establish a relationship between hydrologic effectiveness and storage volume as a percentage of the mean annual runoff volume (MARV), for the seven cities (Figure 2-5). The curves were derived based on a 72-hour detention period. Hydrologic effectiveness is defined as

$$\Psi = \frac{V_t}{V_r} \qquad (2\text{-}7)$$

Where $\Psi$ = hydrologic effectiveness (%)
$V_t$ = total volume treated ($m^3$)
$V_r$ = total runoff volume ($m^3$)

For Figure (2-5), $V_t$ equals the volume of runoff that has at least a 72-hour detention period in the wetland.

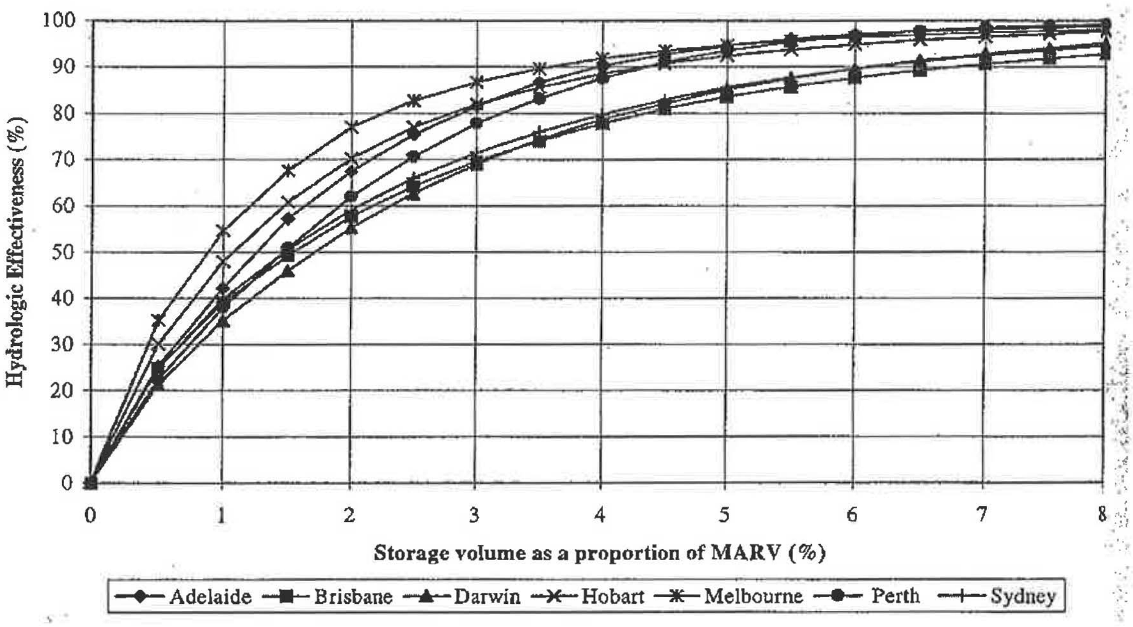


**Figure 2-5 Hydrologic Effectiveness of Seven Australian Capital Cities (Source: Somes & Wong, 1998b)**

Another method commonly used in sizing ponds is the generic curve method (DLWC, 1998). It involves determining the hydraulic residence time required for the removal of the target pollutants from generic pollutant removal curves similar to those in Figures (2-1), (2-2) and (2-3). The basin size required for the design hydraulic residence time (HRT) is then determined by a simple average daily runoff calculation ($R_{da}$). The $R_{da}$ is found by dividing the average annual catchment runoff by 365. The wetland volume is then calculated by

$$\text{Wetland Volume (m}^3\text{)} = R_{da}(\text{m}^3/\text{day}) \times \text{HRT(day)} \qquad (2\text{-}8)$$

Nix et al (1988a) used long term simulation modelling to predict long-term pollutant removal in a detention basin. A particle settling routine was used to predict suspended solids removal under a variety of conditions. The storage capacity and diameter of a circular outflow orifice were varied and graphical relationships were found between these two design parameters and percent suspended solids removal. These relationships could be used to select the appropriate basin size and orifice diameter for a given pollutant removal percentage. Sear et al (1992) used a similar technique to derive relationships between pond volume and pond capture efficiency for a range of catchment types.

### 2.2.8 Outlet Structures

There are four general types of outlet structures used in constructed stormwater wetlands; weirs, orifices, drop inlets and pipes.

Weirs are often used as primary overflow control devices. Weirs can come in many different shapes and sizes. The three most commonly used are rectangular weirs (Figure 2-6), V-notch weirs (Figure 2-7), and trapezoidal weirs (Figure 2-8).

The discharges for these weirs are given by

Rectangular weir:
$$Q = C_r L h^{3/2} \tag{2-9}$$

V-notch (triangular) weir:
$$Q = C_v h^{5/2} \tan\left(\frac{\theta}{2}\right) \tag{2-10}$$

Trapezoidal weir:
$$Q = C_z \sqrt{2g}\left(\frac{2}{3} L h^{3/2} + \frac{8}{15} Z h^{5/2}\right) \tag{2-11}$$

Where
$Q$ = discharge rate ($m^3/s$)
$C_r$ = discharge coefficient for a rectangular weir
$C_v$ = discharge coefficient for a v-notch weir
$C_z$ = discharge coefficient for a trapezoidal weir
$h$ = head (m)

(Urbonas & Stahre, 1993; BCHF, 1992)

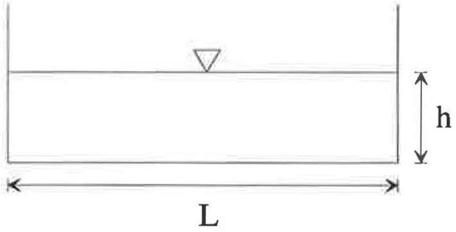


**Figure 2-6 Rectangular Weir**

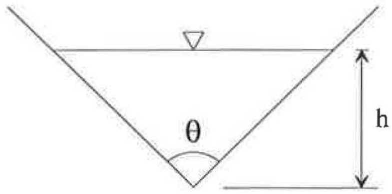


**Figure 2-7 V-notch Weir**

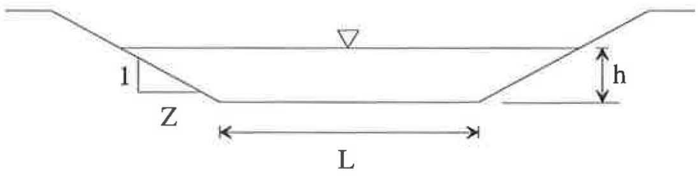


**Figure 2-8 Trapezoidal Weir**

Rectangular weirs can be either sharp-crested or broad crested. The discharge coefficient for a sharp-crested rectangular weir is typically about 1.8 and for a broad-crested rectangular weir the discharge coefficient is around 1.7. These values were determined experimentally. V-notch weirs are generally sharp crested; the value of $C_v$ depends on the angle θ (typically between 1.4 for a 45° notch and 1.5 for a 90° notch). Trapezoidal weirs are usually only found in the broad-crested variety and the value of $C_Z$ is usually around 0.6.

Orifices are openings cut into the side of a drop inlet, weir or the side of the storage basin. The flow through a vertically aligned orifice is governed by

$$Q = C_o A \sqrt{2gh'} \qquad (2\text{-}12)$$

| | | | |
|---|---|---|---|
| Where | $A$ | = | area of the orifice |
| | $C_o$ | = | coefficient of discharge |
| | $h'$ | = | height of the water surface above the centre line of the orifice |

Co is typically 0.62 for a sharp edged orifice.

Drop inlets consist of a vertical riser pipe, located inside the pond, leading to an outlet culvert or pipe (see Figure 2-9).

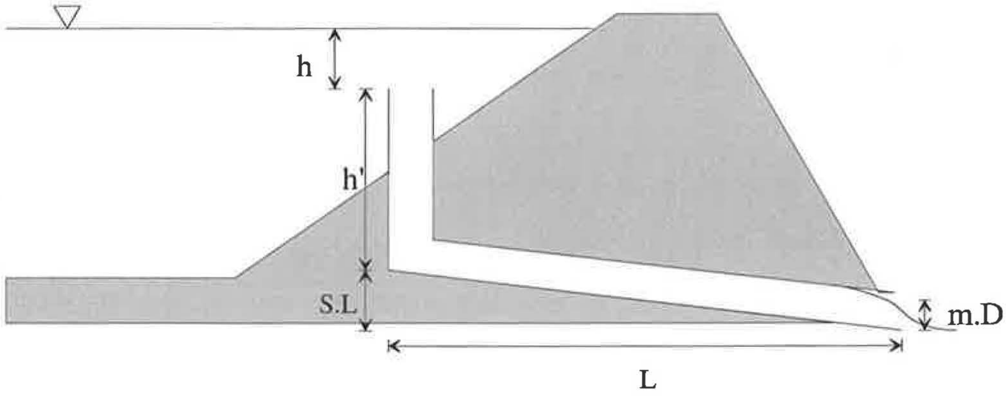


**Figure 2-9 Drop Inlet (Source: Urbonas and Stahre, 1992)**

The flow through a circular drop structure is governed by three equations, representing the different flow regimes.

I $$Q = C_1 2\pi R h^{3/2} \quad (2\text{-}13)$$

II $$Q = C_2 \pi R^2 \sqrt{2gh} \quad (2\text{-}14)$$

III $$Q = A\sqrt{\frac{2gDH}{f\,L'}} \quad (2\text{-}15)$$

Where $H = (h + h' + SL - mD)$ and $L' = h' + L$

| | | |
|---|---|---|
| R | = | radius of drop inlet (m) |
| A | = | area of outlet pipe ($m^2$) |
| S | = | slope of outlet pipe |
| L | = | length of outlet pipe (m) |
| h | = | height of water above drop inlet |
| h' | = | height of drop inlet above pipe's invert |
| D | = | diameter of pipe (m) |
| m | = | ratio of water depth to pipe diameter at the outlet end of the pipe |
| f | = | friction factor |
| H | = | head lost in friction |

(Daugherty & Franzini, 1977)

The first flow regime (I) occurs when the drop inlet is not submerged and it behaves as a circular weir. When the drop inlet is fully submerged but the pipe is not flowing full the flow is governed by orifice flow (Equation 2-14). The final flow regime occurs when the pipe is flowing full (Equation 2-15). The lowest flow given by Equations (2-13), (2-14) or (2-15) is used in each particular case.

Pipes are generally placed towards the bottom of wetland ponds and are small in diameter to release water slowly so as to maximise water quality improvement. The flow through a pipe is governed by Equation (2-15).

The functions performed by outlet structures in constructed stormwater wetlands can be categorised into 6 groups (BCHF, 1992)

1. Emergency outlet – used to pass extreme flood events. It usually consists of a spillway section cut into the pond embankment.
2. Service outlet – used to pass small flood flows and regular storm events. It usually consists of a weir or drop structure.
3. Extended detention outlet – used to release the live storage slowly for water quality improvement. The types of structures used for this purpose are typically pipes, or orifices cut into a drop structure or weir section.
4. Drainage pipe and outlet – used to drain the pond completely for maintenance.
5. Vegetation establishment outlet – used to control pond water levels during the initial establishment of vegetation. Usually a pump or drainage outlet is used for this purpose.
6. Flood control outlet – used to attenuate flows from large storms in conjunction with flood control storage. The device used for this purpose might be a weir, vertical slot, narrow v-notch weir, orifice or other device

Only the methods used in the design of pollutant removal outlets will be discussed in this section. The methods used for the selection and sizing of outlet structures for flood mitigation are given in Section 2.3.1.

BCHF (1992) presents a procedure for sizing extended detention outlets for pollutant removal. The size of the outlet is chosen based on 75% removal of suspended solids for a selected stormwater quality design storm. The water quality volume ($V_d$), defined as the pond volume to the base of the flood control outlet, is set equal to the total runoff for the design storm. The report suggests emptying times ($t_d$) to achieve 75% suspended solids removal for various outlet devices (Table 2-3). These emptying times were based on results from experiments. Once the type of outlet is selected, the water quality volume flow rate is determined by

$$Q = \frac{V_d}{t_d} \tag{2-16}$$

The outlet size is found using Q, the available head (h) and the appropriate flow equation.

**Table 2-3 Emptying Times (Hours) for the Water Quality Volume for Extended Detention (Source: BCHF, 1992)**

| Type of outlet | Minimum required | Preferred |
|---|---|---|
| Weir | 2.5 | 5 |
| Orifice | 8 | 16 |
| Constant flow device | 12.5 | 25 |

Pazwash (1992) presented a similar design approach, however the design criteria used was that no more than 90% of the water quality volume was released during an 18-hour period following the peak flow.

Beatley and Wigfield (1993) suggested that the water quality volume detained in the pond be equal to the first 12.5 mm of runoff multiplied by the catchment area, and that the detained volume be released over a 30-hour period.

Somes and Wong (1998c) used a long-term simulation model to examine the effect that changing outlet type had on the detention time distribution. It was concluded that weir outlets have the most widespread distribution followed by drop inlets and orifices. The authors also investigated the effect of raising the height of a drop inlet, hence increasing the size of the permanent pond in the wetland, on detention times. They found that increasing the permanent pond increased mean detention times.

### 2.2.9 Basin Geometry

The geometry of a wetland basin influences its ability to perform its intended functions. Design characteristics such as depth, shape, shoreline profile and the location of inlet and outlet structures, vegetation zones and islands affect the basin's hydraulics and pollutant removal efficiency.

Short-circuiting is the most important factor to consider when designing stormwater wetlands. Short-circuiting occurs when 'dead' zones exist in the basin, resulting in non-uniform flow paths in the basin (Figure 2-10). A pond with short-circuiting will not perform pollutant removal as efficiently as a pond with a uniform flow velocity distribution, because some of the inflow will remain in the basin only for a short time.

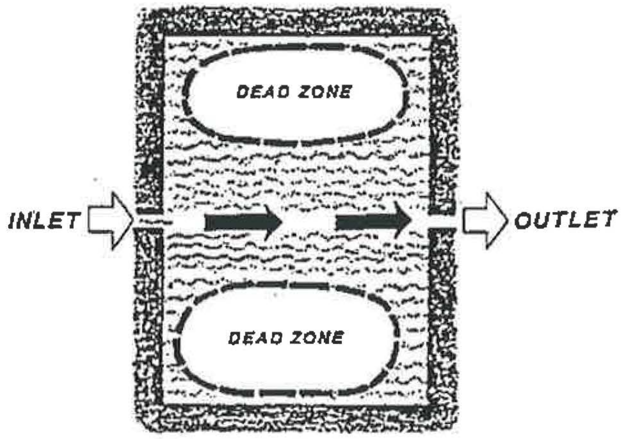


**Figure 2-10 Example of Pond Short-Circuiting (Source BCHF, 1992)**

The basin design guidelines used by wetland designers have been derived from data collected through monitoring existing wetlands and computer simulations. The focus of research conducted to date has been on how the various basin design aspects affect the key performance indicators of pollutant removal efficiency, hydraulic efficiency and the flow velocity distribution.

Hydraulic efficiency is defined as

$$\text{hydraulic efficiency} = \bar{t}/T \tag{2-17}$$

Where $\bar{t}$ = actual mean residence time
$T$ = theoretical, volumetric residence time

Hydraulic efficiency is measured either by dye-tracer experiments or by computer modelling. Wetlands with hydraulic efficiencies close to 1 will closely approximate the ideal plug flow conditions.

The most discussed aspect of basin geometry is the length to width ratio. It has substantial influence on a basin's hydraulic and pollutant removal efficiency. Generally, length to width ratios of between 3 and 5 ensure efficient distribution of flow (Lawrence & Breen, 1998).

Thackston et al (1988) analysed results from a study into the hydraulic efficiency of wastewater treatment wetlands. The length to width ratio was found to have the strongest influence on hydraulic efficiency. Data from 38 wetlands ~~was~~ were used to derive a relationship between hydraulic efficiency and length to width ratio (Figure 2-11).

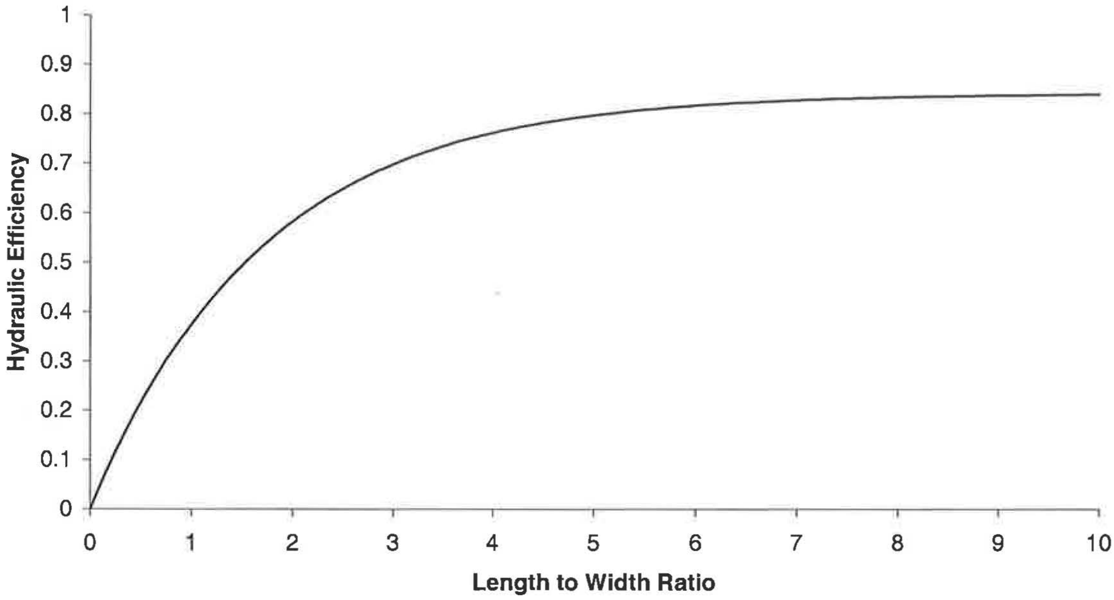


**Figure 2-11 Hydraulic Efficiency as a Function of Length to Width Ratio**

Walker (1996b) conducted numerical modelling of stormwater flow through rectangular and triangular shaped basins with length to width ratios of 0.5. 1.0, 2.0, 4.0 and 8.0. He found that hydraulic efficiency improved as the length to width ratio increased; however, the improvement between ratios of 4 and 8 was small. The results obtained compared favourably with the relationship derived by Thackston et al (1988).

The type of treatment process proposed usually determines the basin depth. For wetlands where the proposed pollutant removal process is the adsorption of dissolved pollutants by macrophytes and associated epiphytes, depth must generally be kept below 0.6 m (Lawrence and Breen, 1996). For wetlands where the proposed pollutant removal process is sedimentation, average depths should be between 1 m and 2 m (BCHF, 1992). Depths should not exceed 2.5 m to 3 m, because of the increased risk of thermal stratification (Lawrence & Breen, 1998).

Side slope gradients should range between 1:6 to 1:8 (vertical:horizontal) to prevent erosion by wave action, to encourage edge vegetation, to minimise mosquito problems and for safety reasons (Lawrence & Breen, 1998).

Islands placed in the shortest flow path improve the flow velocity distribution in the basin and increase the surface area available for the establishment of macrophytes (BCHF, 1992). Persson (1998) conducted two-dimensional hydraulic simulations on a variety of basin configurations. The placement of a small island in the shortest flow path improved the hydraulic efficiency of the basin significantly.

A large distance between the pond inlet and outlet/s improves the hydraulic efficiency of basins (Persson, 1998).

A well functioning wetland should consist of a series of vegetated and open water zones located perpendicular to the direction of flow (Figure 2-12). Wetland plants assist in the even distribution and slowing of flows, they transfer oxygen to the sediments and they provide a substrate for algal and biofilm biomass (Lawrence and Breen, 1998). Open water areas help reduce short-circuiting and they aid in the mixing of flows (NSW EPA, 1997b).

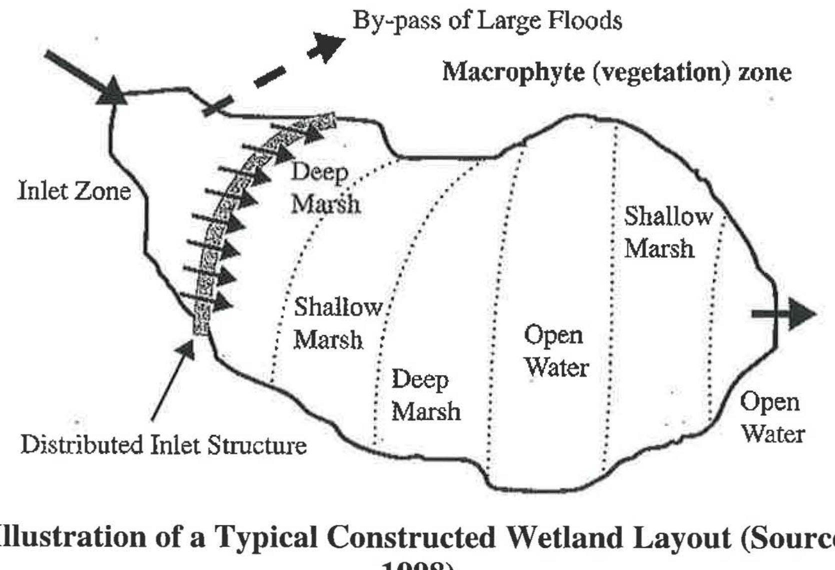


**Figure 2-12 Illustration of a Typical Constructed Wetland Layout (Source: Wong et al, 1998)**

### 2.2.10 Wetland Simulation Modelling

#### 2.2.10 a) Event-based Techniques

Event-based analysis involves sizing the wetland pond and outlet structures so that the required pollutant removal is achieved for a particular design storm. Typically a 1 in 1 year or 1 in 2 year ARI storm event is used as the design storm event (DLWC, 1998). The single event is used to test whether a basin design satisfies a detention or drawdown time requirement. The disadvantage in using event-based analysis is that it ignores the cumulative effect of closely spaced storms, hence the design estimates may not be accurate.

Fabian (1998) developed an event-based model for evaluating the performance of a wetland in terms of water quality improvement. An inflow hydrograph and pollutograph (pollutant concentration versus time) were the inputs into the model. An outflow hydrograph, determined by hydraulic calculations, was used to calculate the average potential detention time of each water or pollutant particle entering the wetland. The quantity of retained pollutant was calculated based on a empirical pollutant retention versus detention time function, similar to those in Figures (2-1), (2-2) and (2-3). The calculated inflow mass and the retained pollutant mass were used to determine the wetland's pollutant removal efficiency.

#### 2.2.10 b) Long-Term Simulation Methods

Time series analysis is used to assess the long-term performance of a wetland. This method involves conducting a computer simulation of the wetland over a long time period. The basis for most time series simulations is a mass balance with a daily or hourly time step. After each simulation the pollutant removal performance of the wetland is compared with the desired performance criteria. A number of simulations are usually conducted before a suitable design is found.

A typical model used for conducting long-term simulations of wetlands and detention basins is the Storm Water Management Model (SWMM) (Huber et al, 1988). SWMM has the capability to model a wide range of basin geometries and outlet structures over an arbitrary number of time steps. Pollutant removal is modelled either by a user supplied removal equation or by a built in discrete particle settling routine. Retention times of outflow can be calculated using either a CSTR or plug flow assumption. SWMM has been used by a number of authors to examine design considerations for stormwater wetlands (Nix et al, 1998a; Sear et al, 1992).

Section 2.3, page 29, sentence 2, change:

The basin's storage volume and the outlet structure configuration are the only design parameters considered in flood mitigation design procedures.

To:

The wetland's active storage volume and the outlet structure configuration are the only design parameters considered in flood mitigation design procedures.

---

Section 2.3.1, page 29, paragraph 1, change:

Procedures used for the preliminary sizing of stormwater detention ponds involve routing a chosen design inflow hydrograph through a storage to achieve a specified reduction in peak flow. The inflow hydrographs used are either derived from actual flow data, or artificial approximations, usually triangles or trapezoidal shapes.

To:

Procedures used for the preliminary sizing of a stormwater detention storage involves routing a chosen design inflow hydrograph through a storage to achieve a specified reduction in peak flow. In Australia, the inflow hydrographs are almost always estimated using a hydrological (rainfall/runoff) model of the upstream catchment and design rainfall data from Australian Rainfall & Runoff, 2000.

The two flood mitigation design parameters often quoted in the literature are the design storage capacity and the permissible site discharge (PSD). The design storage capacity is the storage

Section 2.3.1, page 29, paragraph 3, change:

A very simple method for estimating the storage volume required for peak flow attenuation is based on the assumption that both the inflow and outflow hydrographs are triangular in shape (see Figure 2-13). The storage volume ($V_s$) is shown as the shaded region in the graph. Estimates of the peak inflow and peak outflow rates are required for this method. The required storage volume is calculated by

To:

A very simple method for estimating the storage volume required for peak flow attenuation is based on the assumption that both the inflow and outflow hydrographs are triangular in shape (see Figure 2-13). The storage volume ($V_s$) is shown as the shaded region in the graph. Estimates of the peak inflow and peak outflow rates are required for this method. The required storage volume is calculated by Equation 2-18. Although this method was derived principally for the design of small on-site detention storages it was felt necessary to include because the framework presented here is intended to help in the design of wetland systems of all sizes from small localised systems to large catchment scale multi-pond systems.

---

$$V_s = 0.5t_i(Q_i - Q_o) \tag{2-18}$$

Where $V_s$ = storage volume estimate ($m^3$)
$Q_i$ = peak inflow rate ($m^3/s$)
$Q_o$ = peak outflow rate ($m^3/s$)
$t_i$ = duration of storage facility inflow (s)

(IEA, 1987; Debo, 1995)

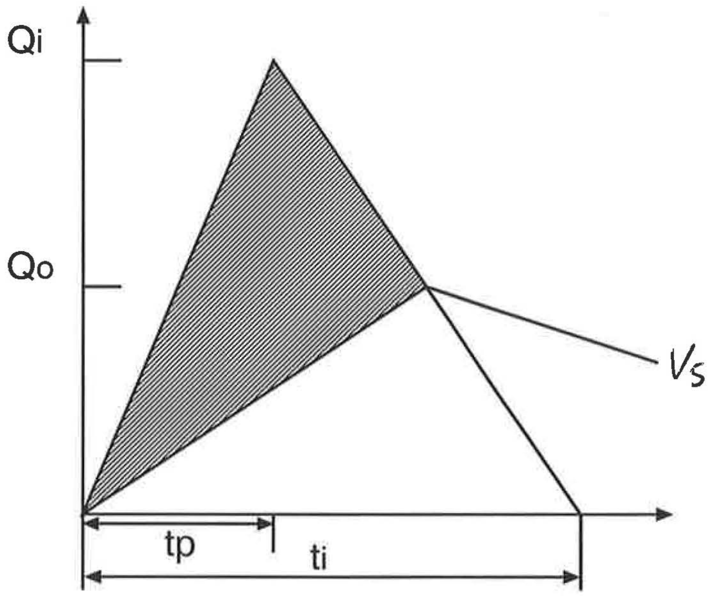


**Figure 2-13 Triangular Shaped Hydrographs**

Reservoir routing is the most commonly used sizing procedure. It involves the repeated application of the discretised continuity equation

$$\frac{S_{i+1} - S_i}{\Delta t} = \frac{I_i + I_{i+1}}{2} - \frac{O_i + O_{i+1}}{2} \tag{2-19}$$

Where $S_i$ = storage volume at time step i
$S_{i+1}$ = storage volume at time step i+1
$I_i$ = inflow at time step i
$I_{i+1}$ = inflow at time step i+1
$O_i$ = outflow at time step i
$O_{i+1}$ = outflow at time step i+1

An initial estimate of the surface elevation $h_{i+1}$ at the end of the time interval $\Delta t$ is used to calculate $S_{i+1}$ and $O_{i+1}$ from storage-elevation and outflow-elevation relationships respectively. These two values and the known values, $I_i$, $I_{i+1}$, $S_i$ and $O_i$ are then substituted into Equation (2-

19), and the difference between the left and right hand sides is calculated. If this difference is not small than another estimate of $h_{i+1}$ is given and the above process repeated for the same time step, otherwise the time step is advance by one. Trial and error is used to determine the appropriate storage volume and PSD required (Mays, 1996).

Australian Rainfall and Runoff (IEA, 1987) contains a number of numerical and graphical methods for sizing reservoirs, detention basins and dams. These methods are based on simple relationships similar to those given in Equations (2-18) and (2-19).

A simplified procedure that avoids the repeated application of reservoir routing is dimensionless routing (McEnroe, 1992; Phillips, 1992).

McEnroe (1992) derived equations relating storage to time and outflow rate to storage that were converted to dimensionless form. The resulting equations were used to derive curves for (required flood storage:flood volume) versus (peak outflow rate:peak inflow rate) for differing outflow, storage and inflow hydrograph shape parameter values. These curves could be used to select an appropriate storage volume and outlet structure size.

Phillips (1992) used a similar technique to derive expressions for the PSD, design storage capacity and period of storage operation for the design of small onsite stormwater detention storages. Triangular shaped inflow hydrographs were used in deriving these expressions.

## 2.4 Stormwater Harvesting for Re-use

Stormwater harvesting for re-use entails the removal of treated stormwater from the wetland for direct or indirect application. Indirect application involves the storage of water in a surface storage or underground through aquifer storage and recovery (ASR).

There are no procedures aimed specifically at the design of wetlands for water harvesting, since the design requirements are similar to those for pollutant removal. The basin is designed so that the quality of the water stored in the basin is suitable for the intended application and that there is a sufficient volume of water available for drawdown to meet quantity requirements.

This section contains a review of water reuse guidelines and some background information on ASR.

### 2.4.1 Water Quality Guidelines for Stormwater re-use

There are two water quality guidelines used for reuse of stormwater in South Australia, the S.A. reclaimed water guidelines (EPA, 1998) and the Australian water quality guidelines for fresh and marine waters (ANZECC, 1992).

The S.A. reclaimed water guidelines were written primarily for reclaimed wastewater, however

Section 2.4.1, page 32, paragraph 2, sentence 2, change:

The guidelines categorise water quality requirements into five classes.

To:

The guidelines categorise water quality requirements into four classes.

The ANZECC water quality guidelines (ANZECC, 1992) provide a more extensive breakdown of the minimum water quality requirements for various uses and receiving ecosystems. These guidelines are summarised briefly in Table (2-5). Water quality requirements are given for a range of pollutants, including faecal coliforms, suspended solids, total dissolved solids, and inorganic and organic compounds. The guidelines are more comprehensive than the values contained in Table (2-4). They also provide detailed water quality criteria for watering of various livestock and for the irrigation of many different agricultural crops. Criteria for a range of pesticides, monocyclic aromatics, organochlorines, organophosphates and polyaromatic hydrocarbons for drinking water and water discharged to aquatic ecosystems, are also given. These compounds are rarely monitored and are excluded from the summary table.

**Table 2-4 S.A Reclaimed Water Guidelines**

| Class | Uses | Microbiological criteria – thermotolerant coliforms / 100 ml (median) | Chemical / physical criteria (mean) |
|---|---|---|---|
| A | • Primary contact recreation<br>Residential<br>- gardens<br>- toilet flushing<br>- washdown water<br>• Municipal irrigation with public access/adjoining premises<br>• Dust suppression with unrestricted access<br>• Unrestricted crop irrigation | < 10<br><br>Removal of viruses, protozoa and helminths to be considered | Turbidity<br>≤ 2 NTU<br><br>BOD < 20 mg/L<br><br>Chemical content to match use |
| B | • Secondary contact irrigation<br>• Ornamental ponds with public access<br>• Municipal irrigation with restricted access<br>• Dust suppression with restricted access<br>• Restricted crop irrigation<br>• Irrigation for pasture or fodder for grazing animals<br>• Fire fighting<br>• Washdown and stockwater | < 100<br><br>Removal of viruses, protozoa and helminths to be considered | BOD < 20 mg/L<br><br>SS < 30 mg/L<br><br>Chemical content to match use |
| C | • Passive recreation<br>• Municipal irrigation with restricted access<br>• Restricted crop irrigation<br>• Irrigation for pasture and fodder for grazing animals | < 1,000<br><br>Removal of viruses, protozoa and helminths to be considered | BOD < 20 mg/L<br><br>SS < 30 mg/L<br><br>Chemical content to match use |
| D | • Restricted crop irrigation<br>• Irrigation for turf production<br>• Silviculture<br>• Non food chain aquaculture | < 10,000<br><br>Helminths need to be considered for pasture and fodder | Chemical content to match use |

**Table 2-5 ANZECC Water Quality Guidelines**

| Parameter | Recreation primary contact | Freshwater ecosystems | Marine waters ecosystems | Drinking water | Raw water | Irrigation |
|---|---|---|---|---|---|---|
| **MICROBIOLOGICAL CHARACTERISTICS** | | | | | | |
| **Faecal coliforms / 100ml** | 150 median | - | - | 0 | < 10 | 1000 |
| **PHYSICAL CHARACTERISTICS** | | | | | | |
| **Dissolved Oxygen (%)** | - | - | > 6 mg/L (80 – 90% saturation) | 6.5 - 8.5 | 6.5 - 8.5 | 9 max |
| **Hardness (as $CaCO_3$) (mg/L)** | - | - | - | 200* | 500 | - |
| **PH** | 5 – 9 | 9 max | 6.5 – 8.5 (< 0.2 pH unit change) | 6.5 - 8.5 | 6.5 - 8.5 | 9 max |
| **TDS (mg/L)** | N/A | < 1000 | N/A | 500* | 500 - 1000 | - |
| **True colour (HU)** | 100 Pt-Co | - | - | 15 | 15 Pt-Co | - |
| **Turbidity (NTU)** | Unobjectionable | - | < 10% change seasonal mean concentration | 5 | 5 | - |
| **METALS (mg/L)** | | | | | | |
| Aluminium (Al) total | - | - | - | - | 0.2 | 5.0 |
| Aluminium pH <= 6.5 | - | 0.005 | N/A | 0.2* | - | - |
| Aluminium pH > 6.5 | - | 0.1 | N/A | - | - | - |
| Arsenic (As) | 0.05 | 0.05 | 0.05 | 0.007 | 0.05 | 0.1 |
| Cadmium (Cd) | 0.005 | 0.0002 - 0.002 (a) | 0.002 | 0.002 | 0.005 | 0.01 |
| Chromium (Cr) | 0.05 | 0.002 | 0.002 | 0.05 | 0.05 | 1 |
| Copper (Cu) | - | 0.002 - 0.005 (a) | 0.005 | 2.0 | 1.0 | 0.2 |
| Cyanide | 0.1 | 0.005 | 0.005 | 0.08 | 0.1 | - |
| Lead (Pb) | 0.05 | 0.001 - 0.005 (a) | 0.005 | 0.01 | 0.05 | 0.2 |
| Mercury (Hg) | 0.001 | 0.0001 | 0.0001 | 0.001 | 0.001 | 0.002 |
| Nickel (Ni) | 0.1 | 0.015 - 0.15 (a) | 0.015 | 0.02 | 0.1 | 0.2 |
| Selenium (Se) | 0.05 | 0.005 | 0.07 | 0.01 | 0.01 | 0.02 |
| Silver (Ag) | 0.05 | 0.0001 | 0.001 | 0.1 | 0.05 | - |
| Zinc (Zn) | - | 0.005 - 0.05 (a) | 0.05 | 3* | 5.0 | 2 |
| **NON-METALS (mg/L)** | | | | | | |
| Ammonia-N | - | 0.02 - 0.03 (h) | < 0.005 (c) | 0.5* | 0.01 | - |
| Nitrate-N | 10 | - | 0.1 - 0.01 (c) | 50 | 10 | - |
| Nitrite-N | 1 | - | - | 3 | 1 | - |
| Nitrogen total-N | - | 0.04 - 0.06 (d) | 0.005 - 0.01 (e) | - | - | - |
| Phosphorus (P) | - | 0.1 - 0.2 (b) | 0.005 - 0.015 (c) | - | - | - |
| Sulphide | - | 0.002 | 0.002 | - | 0.05 | - |
| Sulphate | - | - | - | 500 | 400 | - |

(a) - Depends on the hardness of the water
(b) - Lakes and reservoirs
(c) - Estuaries
(d) - Rivers
(e) - Coastal
* - Aesthetic guideline (taste/odour/colour etc.)

### 2.4.2 Aquifer Storage and Recovery (ASR)

2.4.2 a) What is ASR?

Aquifer Storage and Recovery (ASR) is the deliberate transfer of surface water into the groundwater system and its subsequent extraction at a later stage. It provides a cost-effective water management tool whereby stormwater runoff and reclaimed wastewater can be used to supplement existing water supply sources. ASR is particularly attractive in arid, semi-arid and monsoonal areas, where there is a water surplus for part of the year followed by a long dry period (Dillon et al, 1997). Aquifer recharge usually takes place during the wet season when water demand is low, and extraction, for reuse, occurs during the dry season when water demand is high.

There are a number of advantages in using ASR instead of surface storage:

- Less land is required
- No water is lost due to evaporation
- The volume of outflow to the marine and freshwater environment is reduced
- The injected water is filtered by porous media as it passes through the aquifer

ASR systems attached to constructed wetlands in urban areas generally consist of an offtake pump located in the pond leading to an injection well or series of injection wells, via a pipe. The depth of the injection wells and the recharge and extraction rates are all dependent on the physical and chemical characteristics of the aquifers underlying the site (Pyne, 1996).

### 2.4.2 b) Groundwater Hydrology

Groundwater hydrology is defined as "the science of the occurrence, distribution, and movement of water below the surface of the earth" (Todd, 1980). Groundwater enters the earth via infiltration through the ground surface or through seepage from streams and lakes. Water-bearing formations of the earths crust, known as aquifers, act as conduits for transmission and as reservoirs for storage of groundwater (Todd, 1980).

Aquifers can be classified into three general groups depending on the properties of the surrounding rock layers. A confined aquifer has confining or impermeable layers above and below. The water in a confined aquifer is under pressure and when the aquifer is penetrated by a well the water level will rise to a point above the bottom of the overlying confining bed. The height to which the water rises is referred to as the piezometric head (USDIWPRS, 1981). Natural recharge to a confined aquifer can occur either in a recharge area, where the aquifer crops out on the ground surface, or by slow downward leakage through a leaky confining layer (Fetter, 1994). Aquifers without an impermeable layer above are referred to as unconfined aquifers or water-table aquifers. The movement of the water in a confined or an unconfined aquifer is governed by gravity (USDIWPRS, 1981). Natural recharge to an unconfined aquifer occurs from either downward seepage through the unsaturated zone or by upward seepage from underlying aquifers (Fetter, 1994). A semi-confined aquifer or leaky aquifer is overlain or underlain by a semi-permeable layer. This type of aquifer represents a combination of the previous two types (Todd, 1980).

When water is pumped from an aquifer, water flows from the aquifer into the well and the height of the water table or the piezometric surface at the well and in the surrounding aquifer will be lowered. The reduction in height or drawdown becomes smaller with increasing distance from the well (see Figure 2-14). Drawdown is also affected by the properties of the aquifer, the effect of other pumping wells, the pumping rate, and the aquifer boundaries (Harlan et al, 1989). When water is injected into an aquifer, through artificial recharge, the height of the piezometric surface or the water table is increased and a mound forms around the injection well (see Figure 2-15). The shape of the curve is the reverse of the drawdown curve resulting from extraction (Todd, 1980).

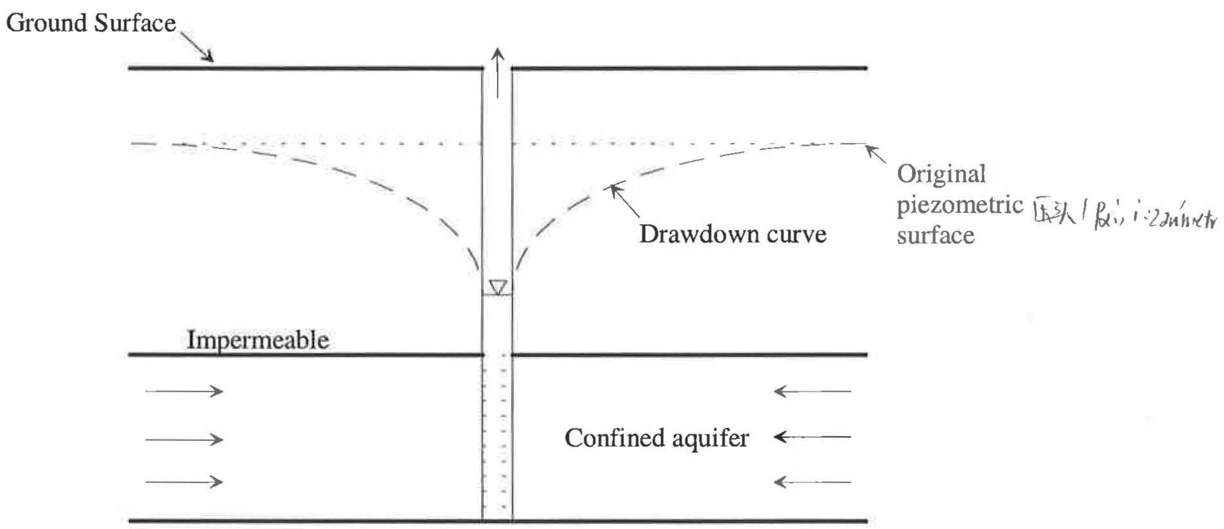


**Figure 2-14 Flow to a Well Penetrating a Confined Aquifer During Pump Extraction**

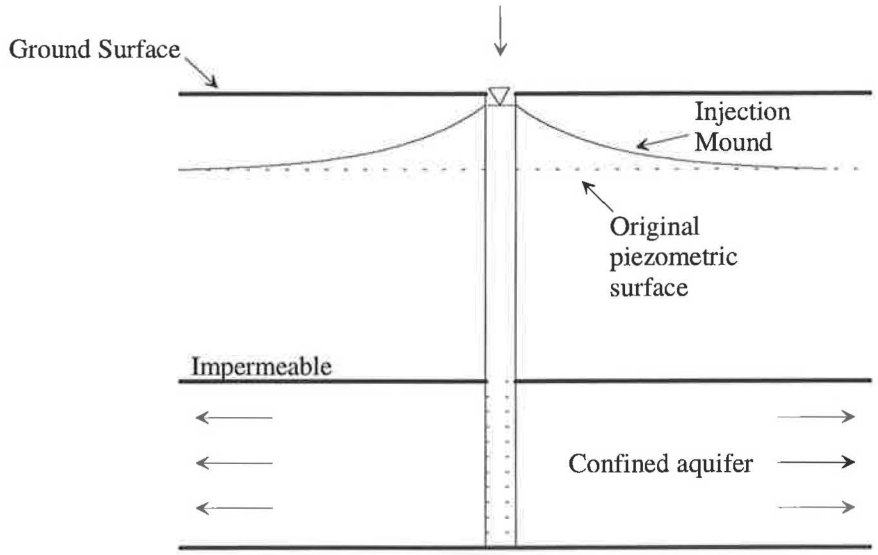


**Figure 2-15 Flow from a Well Penetrating a Confined Aquifer During Artificial Recharge**

2.4.2 c) ASR Well Clogging Processes

Artificial recharge of groundwater usually results in an increasing resistance to flow, which is called “clogging”. Clogging primarily occurs in the gravel pack, the borehole wall, and the formation immediately surrounding the borehole wall. Clogging during recharge, under a constant head, can result in a continually decreasing rate of recharge (Pyne, 1995).

The major physical, chemical and biological processes contributing to clogging are filtration of suspended solids, microbial growth, chemical precipitation, clay swelling and dispersion, air entrapment and gaseous binding, particulate rearrangement and mobilisation of aquifer fines (Dillon et al, 1996). A review of the occurrences and forms of clogging found at 40 different sites was presented by Dillon et al (1996). He reported that clogging occurred at 80% of the sites reviewed, with the most common form being filtration of suspended solids, which occurred in 50% of cases.

The risk of clogging can be reduced by; (a) minimising the concentrations of pollutants in the injected water, particularly suspended solids and organic matter, (b) maintaining positive pressure in the well to prevent air bubbles entering the aquifer, and (c) ensuring that the chemical properties of recharge waters are compatible with the groundwater (Dillon et al, 1996).

Well redevelopment is the process of returning well performance to its prior state by restoring the hydraulic properties of the aquifer. There are a number of mechanical and chemical redevelopment methods. Mechanical methods include pumping from the well (typically at a higher rate than recharge), jetting with compressed air or water, or sectional flush pumping (pumping from access tubes located in the gravel pack). Chemical methods include the addition of acids, flocculants (eg. calcium chloride), disinfection and/or oxidising agents (eg. chlorine). The frequency of redevelopment is dependent on the speed at which recharge rates decline, with some sites being redeveloped on a daily basis (Dillon et al, 1996).

#### 2.4.2 d) Water Quality Guidelines for Aquifer Injection

In South Australia there are no government regulations concerning the quality of water that can be injected into aquifers. Dillon et al (1996) published a set of guidelines for the injection of stormwater and treated wastewater for storage and reuse based on experience and guidelines from overseas. The water quality requirements contained in these guidelines were based on three objectives; the management of clogging, the protection or improvement of groundwater quality and ensuring that the quality of recovered water is fit for its intended use.

The Dillon et al (1996) guidelines were intended to be used in conjunction with the ANZECC Australian water quality guidelines for fresh and marine waters (ANZECC, 1992) and Australian drinking water guidelines (ANZECC, 1994). General recommendations were provided only for those water quality parameters that were regarded as relevant to ASR, ie. suspended sediments, total dissolved solids (TDS), faecal coliforms and nitrogen.

Suspended solids concentrations of below 30 mg/L were recommended. The acceptable concentration could differ depending on the grain size in the aquifer.

A maximum TDS concentration of 500 mg/L for potable reuse, and 1000 mg/L for non-potable reuse was recommended. It was suggested in the report that higher concentrations could be used where acceptable, provided the concentration did not exceed the TDS of the ambient groundwater.

A maximum of 10,000 colony-forming units per 100 mL was recommended. The survival of pathogenic microorganisms in groundwater diminishes with residence time, because the introduced microorganisms are filtered, adsorbed, die-off or are antagonised by native microorganisms. Typical times for the removal of 90% of bacteria is 3 to 6 days and 5 to 30 days for viruses. The time required to remove protozoa is unknown. A minimum residence time of 50 days was recommended for undisinfected injectant to provide an acceptable degree of health protection when recovered water is used for recreation or irrigation.

Nitrogen levels of below 10 mg/L, subject to ammonia concentrations being less than 0.5 mg/L, were recommended for potable reuse. The suggested maximum nitrogen concentration for irrigation reuse was also 10 mg/L.

#### 2.4.2 e) Groundwater Mixing

When water is injected into an aquifer via artificial recharge a lens of injected water forms around the well. Over time the native groundwater mixes with the injected water through diffusion. Where groundwaters are highly saline, mixing can reduce the quality of the injectant to below the required concentration for reuse. Increasing the time lag between injection and extraction increases the amount of mixing that will occur. Figure (2-16) shows the effect that increasing lag has on the recovery efficiency from the bore. The recovery efficiency is also influenced by the physical properties of the aquifer, such as grain size and distribution, transmissivity and storativity (Dillon et al, 1996).

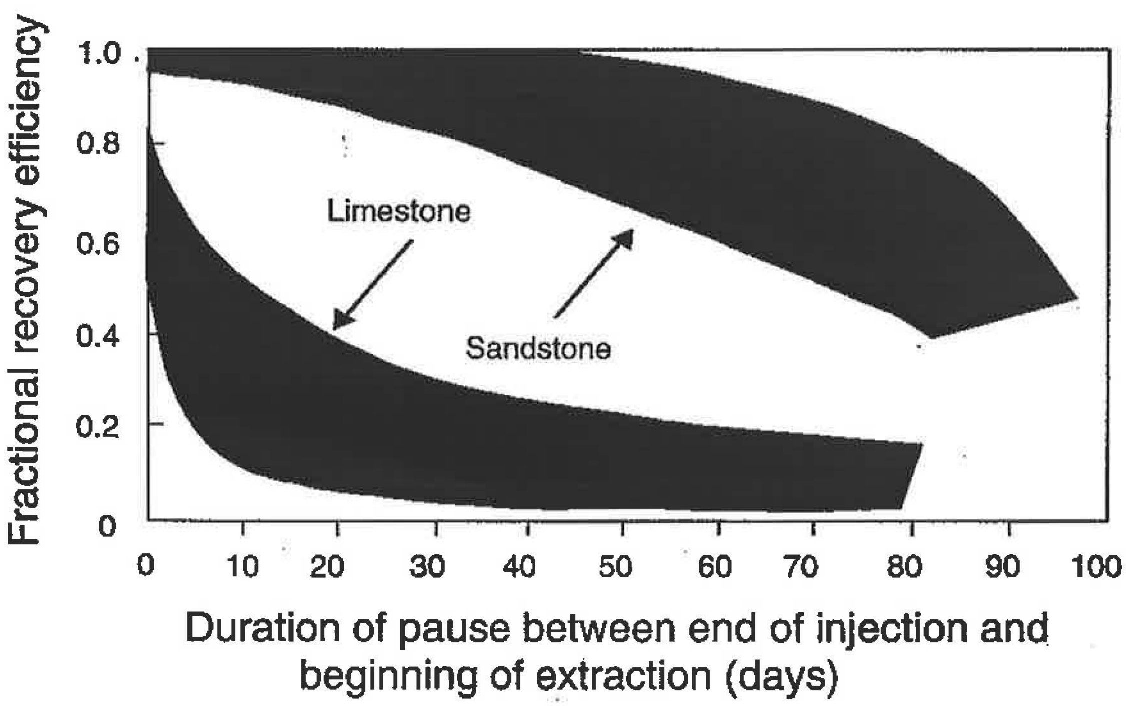


**Figure 2-16 Effects of the Pause Between Injection and Extraction , on Recovery Efficiencies in Limestone and Sandstone Aquifers (Source: Harpaz, 1971)**

2.4.2 f) Local Experience

Adelaide's Mediterranean climate is ideal for ASR; there are four constructed stormwater wetlands in and around the city providing water for ASR schemes (see Table 2-6). The Andrews Farm wetland / ASR scheme is a pilot site established for research purposes. Investigations are being conducted there into the fate of various contaminants, the movement of injected waters, and the clogging and redevelopment of the injection well (Dillon & Pavelic, 1997). The Paddocks and Northfield wetlands are fully operational ASR schemes. The recovered water is used in the irrigation of nearby grassed areas. Operation of the ASR system is very similar for both developments. Recovery from the aquifer occurs when the water level is above the height of the ASR intake pipe and the water quality meets the injection water quality guidelines (ANZECC, 1992; SA EPA, 1998). Turbidity probes in the Northfield and Paddocks wetlands are used for continuous water quality monitoring, and at the Paddocks wetland an electrical conductivity probe is also used. The turbidity probe located near the intake pump is used to monitor the concentration of suspended sediment in the stored water and the electrical conductivity probe measures the concentration of total dissolved solids. All probes were calibrated against water quality data obtained from grab samples (Smith, 1999; PAEC, 1999). Recharge only occurs when the quality of the water meets the guidelines.

**Table 2-6 Summary of ASR Operations in Adelaide (Adapted from Dillon et al, 1997)**

| Site (start year) | Aquifer type | Injection regime | End use | Bore recharge rate (L/s) | Annual recharge volume (ML) |
|---|---|---|---|---|---|
| Andrews Farm (1993) | Tertiary Limestone Confined | Gravity or Pressure | Test site | 15 – 20 | 60 |
| Greenfields (1995) | Tertiary Limestone Confined | Gravity | Irrigation | 10 – 15 | 100 |
| The Paddocks (1995) | Tertiary Limestone Confined | Pressure | Irrigation | 8 | 80 |
| Northfield (1993) | Fractured Rock | Gravity | Irrigation | 10 – 15 | 40 |

## 2.5 Summary

Procedures used for designing multi-function stormwater wetlands for pollutant removal, flood mitigation and harvesting stormwater for re-use were presented.

The procedures used in designing wetlands for pollutant removal consist of a collection of guidelines describing desirable values for various aspects of basin geometry to maximise a wetland's pollutant removal efficiency. These are formulated predominantly from monitoring results. It is generally accepted that, in order to perform efficiently, a wetland requires the following:

(a) A basin length to width ratio of greater than 4:1;

(b) Depths of less than 0.6 m for a well vegetated wetland and between 1 m and 2 m for a detention pond;

(c) Side slope gradients of between 1:6 and 1:8;

(d) Macrophytes planted perpendicular to the flow direction and islands placed in the shortest flow path.

The two wetland design aspects for which general methods are available are basin volume or surface area and the configuration of outlet structures. Methods used to determine the appropriate basin volume or surface area differ in their complexity, from simple calculations based on the area of the contributing catchment to detailed long-term simulation models incorporating pollutant removal algorithms. Long-term simulation models have also been used to design the wetland's outlet structures. Other methods used to calculate an appropriate outlet structure configuration are based on achieving a specified emptying time for a selected design storm event.

The design procedures used for flood mitigation, determine the basin volume and outlet structure configuration required to attain the required reduction in peak flow for a given inflow hydrograph. The inflow hydrograph is either taken from recorded flow data or derived artificially. All flood mitigation procedures presented contained a reservoir routing procedure.

There are no published wetland design procedures aimed specifically at stormwater harvesting for reuse. A summary of the factors that have to be taken into account when designing a wetland for this purpose were presented. These included:

- Ensuring the volume of drawdown water (harvestable water) is sufficient for water quantity requirements
- The quality of the water must be suitable for the intended application
- If ASR is to be used, the injectant must not impact adversely on the quality of water resident in the aquifer

A brief introduction to groundwater hydrology and the physical, chemical and biological aspects of ASR was also presented.

# Chapter 3 Genetic Algorithms

## 3.1 Introduction

Genetic algorithms (GA) are a type of evolutionary optimisation technique based on Darwin's 'Theory of Natural Selection'. GA theory was first introduced by Holland (1975) and is comprehensively explained by Goldberg (1989), that will serve as the major reference text for this chapter.

## 3.2 Simple Genetic Algorithm

The analogy with nature is established by the creation of a set of solutions called a population. Each individual in a population is represented by a series of parameters that completely describe a solution. These parameters (referred to as genes) are encoded into chromosomes, analogous to chromosomes found in DNA. The standard GA uses a binary alphabet (0's or 1's) to form chromosomes, however integer and real numbers may also be used. Only the application of GA's using integers will be discussed in this chapter, as they were the only variable types used in this research.

The initial population of solutions, usually chosen at random, is allowed to evolve over a number of generations, until a near optimal solution is found. During each generation each chromosome is assigned a fitness value. This is a measure of how well it performs with respect to an objective function. The fitter solutions are selected for mating, then crossover and mutation operators act on the remaining solutions to generate a new population for the next generation (see Figure 3-1).

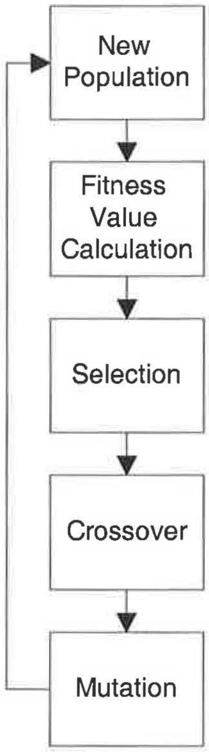


**Figure 3-1 The Simple GA**

### 3.2.1 Operators

3.2.1 a) Selection

Selection is used to ensure that the chromosomes with the highest fitness values are present in increasing numbers in future generations. This operator is an artificial version of natural selection. The higher the fitness of an individual, the greater chance it has of passing its genetic material to the next generation.

There are two main methods by which selection is implemented; tournament and proportionate selection.

3.2.1 a) i. Tournament

In tournament selection two or more parent chromosomes are selected at random and their fitness values are compared with each other. The most fit chromosomes are added to the mating pool and the less fit chromosomes are discarded. This process is repeated until there are the required number of chromosomes in the mating pool, while ensuring that the entire parent population has been involved at least once.

**Table 3-1 Sample Problem Chromosomes and Fitness Values**

| No. | Chromosome | | | | | | | | Fitness |
|---|---|---|---|---|---|---|---|---|---|
| 1 | 123 | 35 | 64 | 72 | 37 | 38 | 91 | 12 | 1,237 |
| 2 | 37 | 52 | 85 | 67 | 127 | 458 | 29 | 93 | 4,873 |
| 3 | 38 | 27 | 428 | 84 | 10 | 63 | 49 | 367 | 3,962 |
| 4 | 83 | 48 | 36 | 79 | 263 | 102 | 37 | 45 | 6,382 |
| 5 | 205 | 32 | 17 | 53 | 59 | 43 | 83 | 26 | 1,002 |
| 6 | 8 | 384 | 284 | 46 | 25 | 48 | 18 | 25 | 8,329 |
| 7 | 93 | 48 | 36 | 37 | 41 | 50 | 392 | 95 | 5,925 |

The example population of chromosomes shown in Table (3-1) will be used to illustrate the functioning of a simple genetic algorithm.

A simple example of how tournament selection operates is shown in Figure (3-2). In this case two chromosomes from the parent population, selected at random, are compared and the chromosome with the higher fitness value is passed into the mating pool.

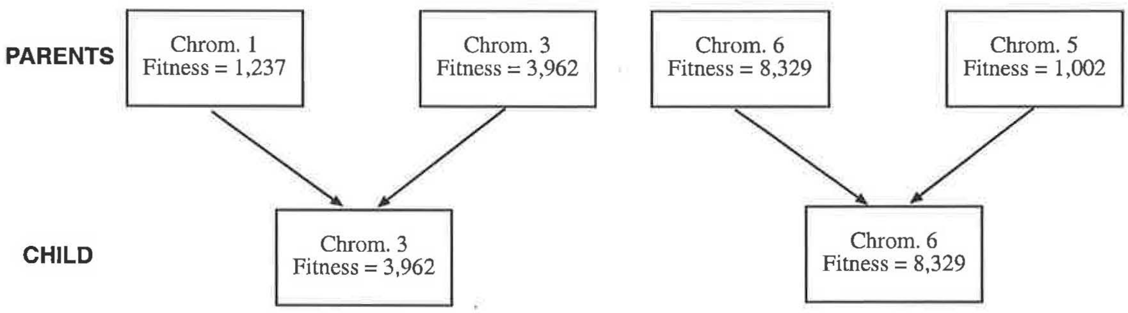


**Figure 3-2 Tournament Selection Example**

### 3.2.1 a) ii. Proportionate

Proportionate selection is a process that works on the same principles as a roulette wheel in a casino. Each individual in a population is allocated a portion of the wheel in proportion to its fitness. Individuals with higher fitness values are allocated a larger portion of the wheel and are therefore more likely to be selected. Every time the wheel is spun, a chromosome from the parent generation is selected and passed into the mating pool. The wheel is spun a number of times until the child mating pool is of the required size.

Figure (3-3) shows how the chromosomes given in Table (3-1) would be allocated space on the proportionate selection 'roulette wheel'.

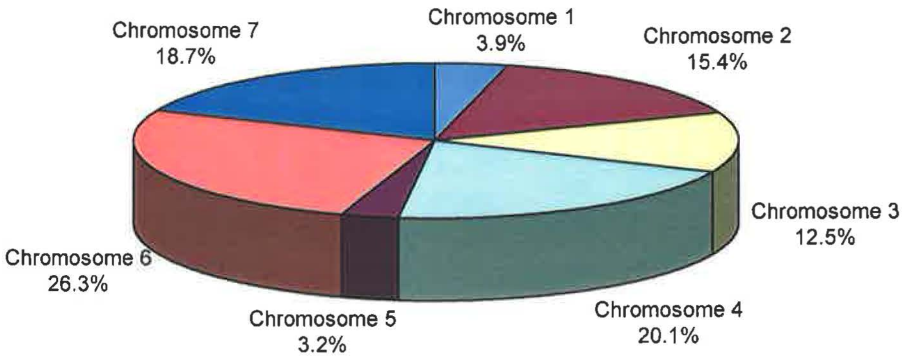


**Figure 3-3 Proportionate Selection ('Roulette Wheel') Example**

### 3.2.1 b) Crossover

A crossover operator is used to simulate gene recombination during reproduction amongst the mating population. The entire mating pool is passed through the crossover operator in randomly selected pairs.

There are three types of crossover operator that are commonly used, single point, multi-point and uniform crossover.

#### 3.2.1 b) i. Single Point

Single point crossover is the simplest type of crossover implementation. It involves selecting a single position at random in the two chosen chromosomes. All gene values before this point are swapped (see Figure 3-4). The two chromosomes produced have a mixture of the genetic material from both of the initial chromosomes.

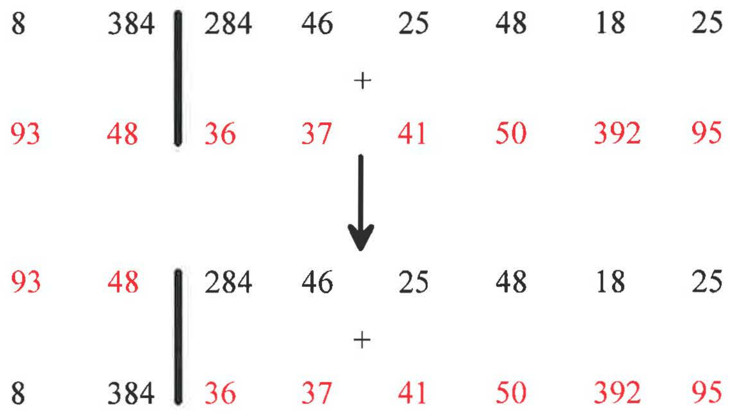


**Figure 3-4 Single Point Crossover**

### 3.2.1 b) ii. Multi-point

Multi-point crossover works in a similar manner to single point crossover, except more than one crossover point is selected. The number of crossover points can be set or chosen at random. The genes located between alternate pairs of selected points are swapped (see Figure 3-5).

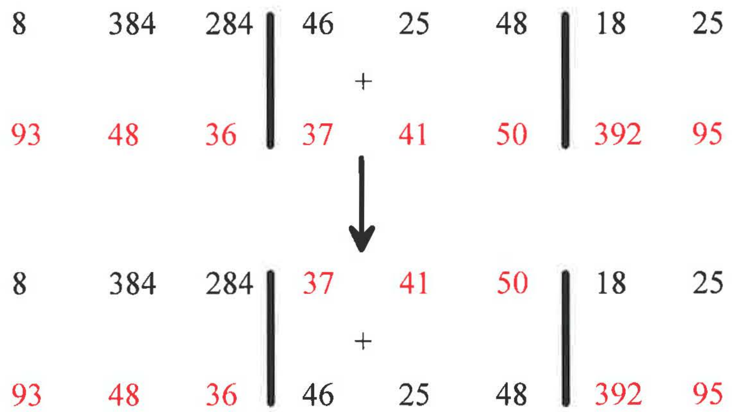


**Figure 3-5 Multi-Point Crossover**

Multi point crossover leads to a more diverse population than single point crossover, as a greater mixture of genetic material occurs.

3.2.1 b) iii. Uniform

Uniform crossover involves examining every gene individually and generating a random number to determine whether swapping occurs. If the random number is below a set crossover probability value, then the genes are swapped (see Figure 3-6).

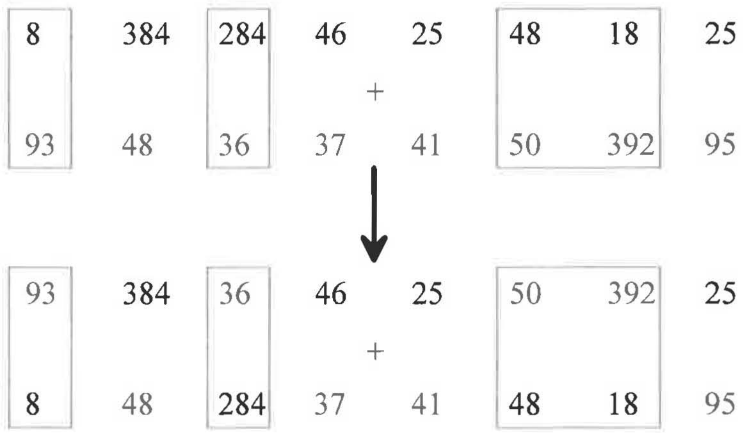


**Figure 3-6 Uniform Crossover**

This method allows a more rapid examination of the available search space then the other methods, as a greater diversity of genetic material is encouraged.

3.2.1 c) Mutation

Mutation is the occasional random alteration (with small probability) of the value of a gene position. A random number is generated for every gene and if this number is below the specified probability of mutation then mutation occurs. The probability is usually kept low (< 0.01) because if it is set too high, the search degenerates into a random process. The mutation operator is introduced to maintain gene diversity in the population.

There are two main types of mutation operator commonly applied to integer-valued chromosomes, creep and random mutation. Creep mutation alters a gene value by stepping it either up or down one unit. Random mutation involves the replacement of a gene value with a randomly generated integer (whose value lies within the permissible range for that particular gene).

### 3.2.2 Convergence

The GA is run for a large number of generations until convergence occurs as indicated by no improvement in the fittest solution for a specified number of generations.

## 3.3 Applications

The robustness of GA's, in comparison with traditional optimisation methods, has made them a popular problem solving technique. Their advantages over other optimisation methods are; they work with the coding of a parameter set, not the parameters themselves; they search the solution space from a population of points, not a single point, they use payoff (objective function) information, not derivatives or other auxiliary knowledge; and they use probabilistic transition rules not deterministic rules (Goldberg, 1989). The main disadvantage with using GA's is that due to their stochastic nature there is no guarantee that the global optimum will be found. Repetitive applications of the algorithms are often needed to ensure that the best solution found is near the global optimum.

GA's have been successfully applied to a number of problems in civil engineering, including pipe network optimisation (Simpson et al, 1994; Savic & Walters, 1997), water resources planning (Connarty, 1995), leak detection in pipe networks (Vítokovský et al, 1999) and detention basin optimisation (Dorn et al, 1995).

## 3.4 Summary

GA's are an evolutionary algorithm based on Darwin's 'Theory of Natural Selection'. They are robust optimisation techniques that have been successfully applied to a wide range of different problems. The workings of a simple GA involve the operations of selection, crossover and mutation.

# Chapter 4 The Optimal Wetland Design Framework

## 4.1 Introduction

The optimal design framework, developed in this thesis, consists of an event-based flood simulation model and a long-term simulation model contained within a standard GA (Figure 4-1). Each chromosome generated by the GA depicts a wetland system design and the genes that they are composed of represent the design parameters of the wetland system.

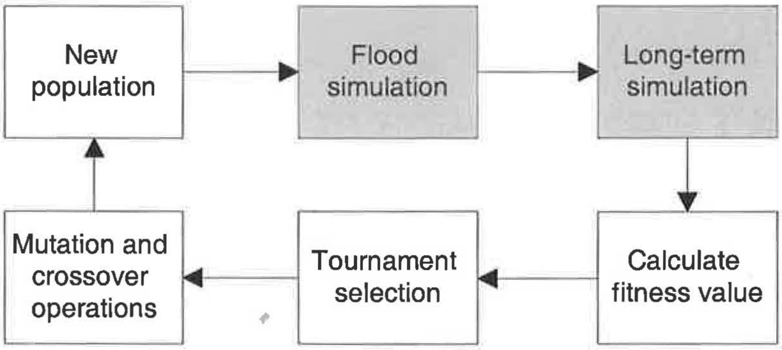


**Figure 4-1 The Optimal Design Framework**

During every generation of the GA the population of chromosomes are put through both simulation models and a fitness value is assigned to them based on cost and their ability to satisfy the wetland system design objectives. The fitter solutions are retained and then mutation and crossover operators are used to generate a new population of chromosomes. This procedure is repeated until a near optimal wetland design is found.

## 4.2 Literature Review of Optimal Wetland Design

An extensive search of the literature did not discover any research conducted to date on the application of optimisation to the design of stormwater wetlands, however optimisation has been used in designing stormwater detention basins. Nix and Heaney (1988) used a graphical procedure to find the optimal basin release rate and storage volume, such that capital and operational costs were minimised for a specified pollutant removal performance level. Segarra (1995) used marginal analysis to solve a similar problem. The optimal condition occurred at the point where the marginal product for a dollar spent on competing resources was equal for all resources.

Several authors have used optimisation to design regional stormwater detention basin networks (Cheng-Kang Taur et al, 1987; Chao-Hsien Yeh & Labadie, 1997; Dorn et al, 1995).

Cheng-Kang Taur et al (1987) used dynamic programming to determine the optimal layout and dimensions for a detention basin network near Austin, USA. The objective was to minimise the total cost for the entire detention basin network, while satisfying peak flow and land availability constraints. The storage volume and peak discharge rate for each detention basin were decision variables in the procedure.

Chao-Hsien Yeh & Labadie (1997) applied successive reaching dynamic programming to a similar problem to that of Cheng-Kang Taur et al (1987), for a catchment in southern Taiwan.

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Section 4.2, page 51, paragraph 4, sentence 4, change:

The GEOHEC-1 modelling software was used to rout the design storm through the detention basin network.

To:

The GEOHEC-1 modelling software was used to route the design storm through the detention basin network.

Dorn et al (1995) used a genetic algorithm to develop trade-offs between cost and overall sediment removal for a system of detention ponds. Each potential pond in the detention system was represented by three decision variables (genes) in a pond system configuration (chromosome). The first variable was a logical variable; 1 signifying the pond was included in the particular solution, and a 0 indicating that the pond was not in use. The other two variables were real numbers and represented, the depth of the normal pool of each pond as a ratio of

Section 4.3, page 52, paragraph 2, change:

Design objectives for flood mitigation could be to reduce peak outflows from the wetland below a specified flow rate and/or to prevent flooding of areas surrounding the wetland. Both of these objectives are assessed using results from a flood simulation. The objective when designing a wetland for pollutant removal is either to reduce the pollutant concentrations of all outflows or to reduce the mean outflow concentration, below a certain level. Flood or long-term simulation could be used if the objective is to reduce all outflows below a certain concentration. The flood simulation approach involves calculating the maximum outflow pollutant concentration for a selected stormwater quality storm. The water harvesting design objective may be to maximise the volume of harvestable water or to achieve a set volume of harvestable water depending on the water requirements and the site design constraints. Both objectives require long-term simulation, as changes in the basin storage volume and water quality over a long period of time (usually $\geq$ 1 year) will determine the harvestable volume.

To:

Design objectives for flood mitigation could be to reduce peak outflows from the wetland below a specified flow rate and/or to prevent flooding of areas surrounding the wetland. The term basin spillage refers to water that spills over the wetlands levee banks, where as basin outflow refers to water that spills over the outflow weir. Both of these objectives are assessed using results from a flood simulation. The objective when designing a wetland for pollutant removal is either to reduce the pollutant concentrations of all outflows or to reduce the mean outflow concentration, below a certain level. Flood or long-term simulation could be used if the objective is to reduce all outflows below a certain concentration. The flood simulation approach involves calculating the maximum outflow pollutant concentration for a selected stormwater quality storm. The water harvesting design objective may be to maximise the volume of harvestable water or to achieve a set volume of harvestable water depending on the water requirements and the site design constraints. Both objectives require long-term simulation, as changes in the basin storage volume and water quality over a long period of time (usually $\geq$ 1 year) will determine the harvestable volume. Some other design objectives that don't fall under the three functions examined in this thesis are the maintenance of a minimum pond volume to preserve wetland habitat and the maintenance of an environmental flow for downstream habitats for which the appropriate output statistic would be a minimum outflow.

to reduce the mean outflow concentration, below a certain level. Flood or long-term simulation could be used if the objective is to reduce all outflows below a certain concentration. The flood simulation approach involves calculating the maximum outflow pollutant concentration for a selected stormwater quality storm. The water harvesting design objective may be to maximise the volume of harvestable water or to achieve a set volume of harvestable water depending on the water requirements and the site design constraints. Both objectives require long-term simulation, as changes in the basin storage volume and water quality over a long period of time (usually $\geq$ 1 year) will determine the harvestable volume.

## 4.4 G.A. Formulation

### 4.4.1 Decision Variable Selection

After determining the treatment train, number of basins, basin types, outlet structure types and water reuse infrastructure required the GA decision variables can be selected. Wetland design parameters that can be represented as decision variables include the basin sizes, outlet heights and sizes, the water harvesting off-take height, size of off-take pump and the number of ASR bores. Outlet height and off-take height refers to the height of the structure from the basin floor. Each decision variable is represented by an integer gene in a chromosome. The permissible range that each of these variables can take is determined by the wetland system constraints.

To illustrate decision variable selection an example problem is presented. The treatment train selected for this problem is a two-basin configuration consisting of an on-line detention pond and an off-line wetland pond (see Figure 4-2). The detention pond has a trapezoidal weir discharging to the receiving channel plus a drop inlet structure and pipe leading to the wetland pond. The outlet structures in the wetland pond are a broad-crested weir that discharges to the stream and a pump off-take feeding the ASR bores.

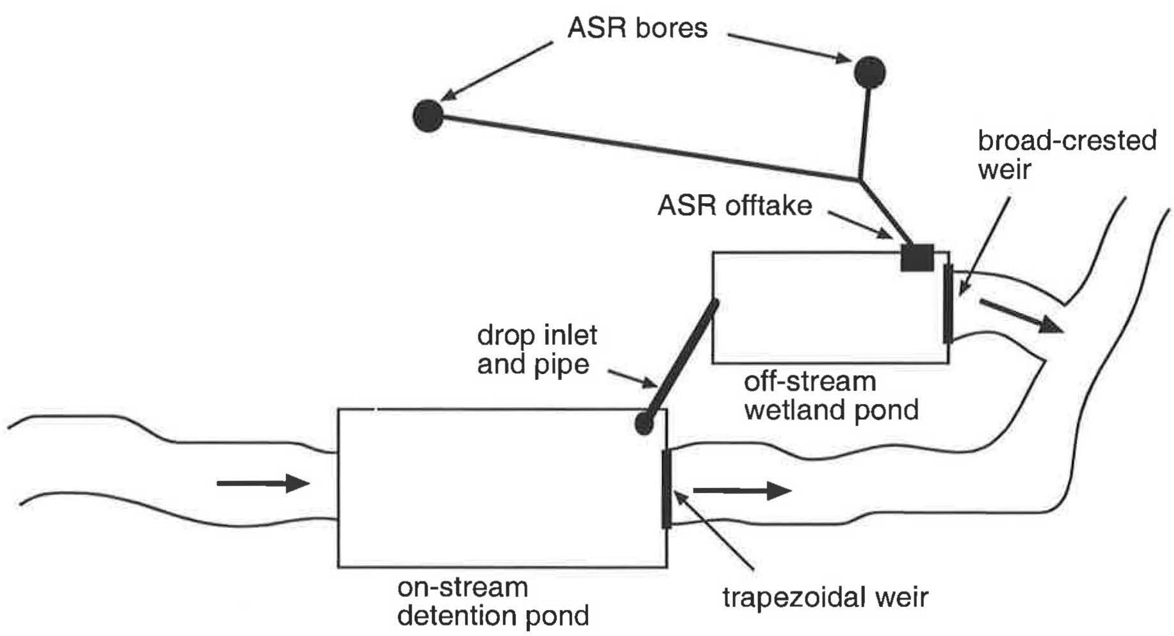



**Figure 4-2 Example Wetland System Configuration**

The decision variables for this problem are the sizes of both basins, the heights and sizes of the weirs and drop inlet, the height of the ASR off-take and the number of ASR bores. These are shown in Figure (4-3).

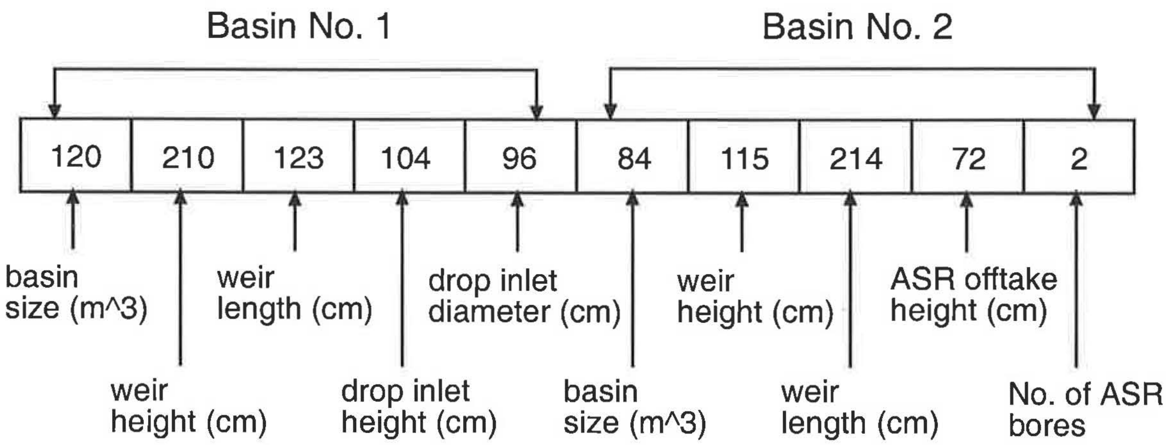



**Figure 4-3 Example GA Chromosome**

### 4.4.2 Mutation and Crossover Operators

As discussed in Section 3.2 the mutation and crossover operators are the means by which the GA generates new solutions to a problem. There are a number of different methods by which mutation and crossover can be implemented. In the design framework presented here, uniform crossover and random mutation are used. These methods were chosen because they are considered better suited to chromosomes with small numbers of integer genes.

### 4.4.3 Fitness Value Calculation

Every chromosome generated by the GA is assigned a fitness value that is composed of construction, operating and maintenance costs and penalty costs. Construction, operating and maintenance costs are the real costs associated the wetland system proposed by the design parameters contained in a chromosome. Penalty costs are artificial costs placed on wetland systems that do not meet the design objectives.

4.4.3 a) Construction, Operating and Maintenance Costs

Only costs that depend on the decision variables were included in the optimisation procedure. These costs include, wetland basin excavation and maintenance, outlet structure construction, and construction and maintenance costs associated with the ASR structures. Other costs that were not affected by changes in any of the decision variables had little or no influence on results. The only directly quantifiable benefit, related to the three functions discussed in this thesis, is the cost saved by reusing harvested water.

The total fitness cost of each wetland configuration generated by the genetic algorithm is calculated by

$$C_t = C_e + C_o + C_{whc} + C_{who} + C_{whm} + C_{wm} - B_{wr} + C_p \quad (4\text{-}2)$$

Where

| | | |
|---|---|---|
| $C_t$ | = | total net cost |
| $C_e$ | = | excavation and formation cost of basins |
| $C_o$ | = | outlet structures construction cost |
| $C_{whc}$ | = | water harvesting infrastructure construction cost |
| $C_{wm}$ | = | wetland maintenance cost |
| $C_{who}$ | = | water harvesting operation cost |

Section 4.4.3, page 55, paragraph 4, change:

The Net Present Value (NPV) of the annual operation and maintenance costs ($C_{wm}$, $B_{wr}$, $C_{who}$, $C_{whm}$) are used in Equation 4-2.

To:

The Net Present Value (NPV) of the annual operation and maintenance costs and water reuse benefit ($C_{wm}$, $B_{wr}$, $C_{who}$, $C_{whm}$) are used in Equation 4-2.

It is extremely difficult to generalise these costs and benefits, as they depend on the local conditions.

4.4.3 b) Penalty Costs

A penalty cost is imposed on any chromosomes that do not meet the design objectives specified for the wetland system. The appropriate output statistic is compared with the corresponding value requirement (see Table 4-1). If the output statistic falls short of the required value, then a penalty is imposed. The penalty cost can be either a set value or it can be proportional to the shortfall. A set penalty cost is constant regardless of the difference between the relevant simulated and required statistic. Set penalties are generally quite high to effectively eliminate chromosomes that do not satisfy the design objective from the GA evolution. Proportional penalty costs are applied to non-critical design objectives, as they don't completely discourage chromosomes that represent infeasible solutions from taking part in the GA evolution. The proportional penalty cost is calculated by multiplying the difference between the required and simulated values by a unit penalty cost.

## 4.5 Flood Simulation

The basis of the flood simulation model is a simple mass balance

$$S_{i+1} = S_i + I_{i+1} - O_{i+1} - Sp_{i+1} \tag{4-3}$$

Where
$S_i$ = storage volume at time i
$I_{i+1}$ = inflow at time i+1
$O_{i+1}$ = outflow at time i+1
$Sp_{i+1}$ = basin spillage at time i+1

The inflow volume for each time step is given in the derived flood hydrograph. The basin's stage/storage relationship is used to calculate the water depth for outflow and basin spillage calculations. Outflow from the wetlands outlet structures is calculated from the discharge equations given in Section 2.2.8. Basin spillage occurs when the water depth exceeds the height of the wetland banks. A simulation time step (around 15 to 60 minutes) is usually suitable for this simulation.

### 4.5.1 Inflow Hydrographs

Section 4.5.1, page 57, sentence 4, change:

If there is no data available than the probabilistic rational method (IEA, 1997) may be used to derive an artificial flood hydrograph, with coefficients estimated based on regional values.

To:

If there is no data available than the probabilistic rational method may be used to derive an artificial flood hydrograph (Phillips, 1992), with coefficients estimated based on regional values.

### 4.5.2 Stage/Storage Relationship

The intended functions and the local conditions determine the shape that constructed wetland basins take. Since these design requirements and constraints are different for each site the shape of constructed wetlands is variable and consequently the selection of a generic basin shape is difficult. In this analysis, wetland basins simulated in both the long-term and flood simulation models were assumed to be rectangular basins with sloped sides (see Figure 4-4). The basin was assumed to have side slopes of 1 in 5 and a basin length to width ratio of 4 to 1. The maximum depth of the basin varied depending on the wetland type, 3 m for a detention pond and 1.5 m for a heavily vegetated wetland. These values are the depths when the basin is full, which is rarely reached. This basin shape was chosen because it simplified the calculation of volume/depth relationships for different basin volumes, and it followed many of the design guidelines described in Chapter 2.

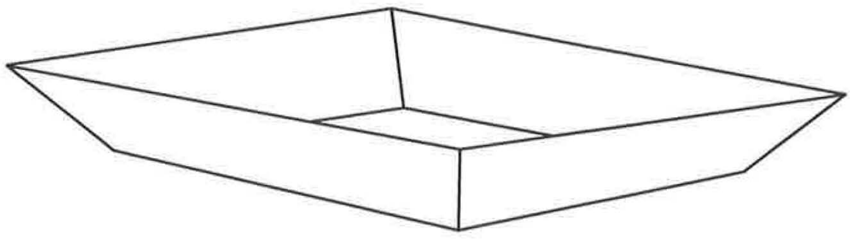


**Figure 4-4 Rectangular Shaped Basin**

During the flood and long-term simulation, volume to depth and depth to volume conversions are made frequently. These conversions are necessary to calculate outflow rates and storage volumes for each time step.

The depth to volume relationship for the rectangular shaped basin was determined using simple integration

$$V = \frac{4}{3}s^2d^3 + sd^2w(1+f) + fw^2d \tag{4-4}$$

Where,
- $V$ = volume
- $d$ = depth
- $s$ = side slope
- $f$ = basin length to width dimension ratio
- $w$ = width of the basin floor

Depth to volume conversions were trivial and took up little computation time. However, the reverse process, calculating the water depth for a given volume of water, required the use of a numerical root finding technique. The application of such techniques can be time consuming, in order to speed up the conversion process, Equation (4-4) was simplified. Five representative basin volumes were chosen, for both 3 m and 1.5 m deep wetlands, and quadratic equations fitted to each depth versus volume curve, using regression. The representative volumes selected represented a range of basin volumes (see Table 4-2).

**Table 4-2 Representative Basin Volumes**

| Group basin volume range ($m^3$) | Representative basin volume ($m^3$) | Quadratic equation |
|---|---|---|
| 0 – 5 000 | 3 500 | $V_w = V_f(0.0958d^2 + 0.0402d)$ |
| 5 000 – 10 000 | 7 500 | $V_w = V_f(0.0707d^2 + 0.1185d)$ |
| 10 000 – 20 000 | 15 000 | $V_w = V_f(0.0525d^2 + 0.1745d)$ |
| 20 000 – 50 000 | 35 000 | $V_w = V_f(0.0358d^2 + 0.2254d)$ |
| 50 000 + | 75 000 | $V_w = V_f(0.0250d^2 + 0.2580d)$ |

Where
- $V_w$ = water volume ($m^3$)
- $V_f$ = full basin volume ($m^3$)
- $d$ = water depth (m)

The depth to volume curves for each of the representative volumes (see Figure 4-5) could be scaled to each volume within the group range. Five representative volumes were chosen because it provided an adequate coverage of the possible depth/volume curve shapes.

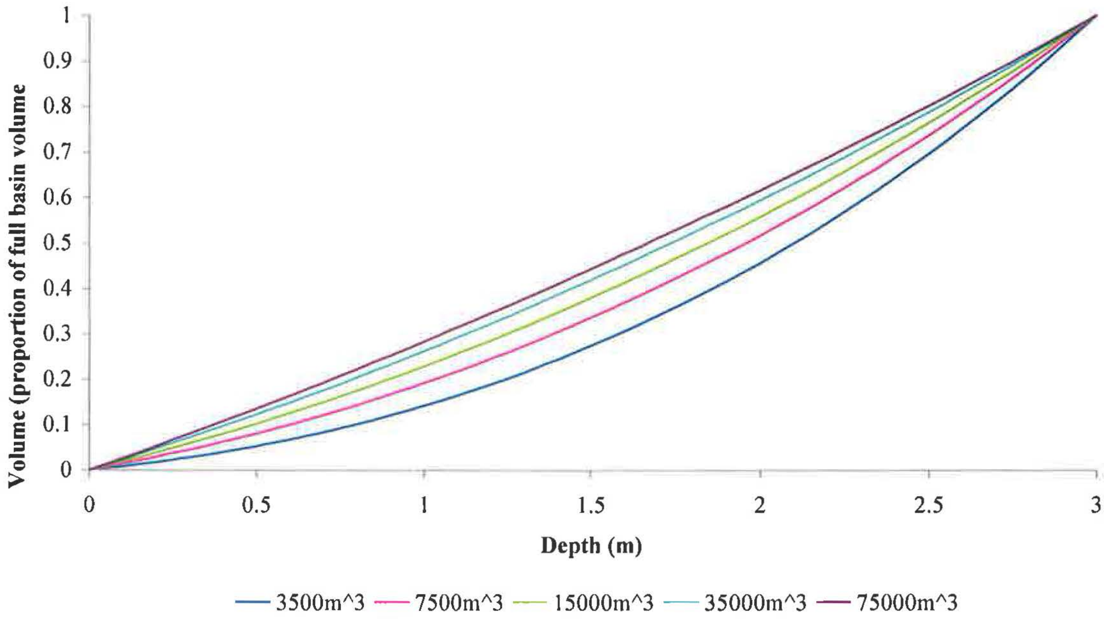



**Figure 4-5 Depth to Volume Curves of the 5 Representative Basin Volumes for a 3m Deep Wetland**

## 4.6 Long Term Simulation

The basis of the long-term simulation model is a simple mass balance

$$S_{i+1} = S_i + I_{i+1} - O_{i+1} - E_{i+1} - Sp_{i+1} - WH_{i+1} + WP_{i+1} \tag{4-5}$$

| | | | |
|---|---|---|---|
| Where | $S_i$ | = | storage volume at time i |
| | $I_{i+1}$ | = | inflow during time step i to i+1 |
| | $O_{i+1}$ | = | outflow during time step i to i+1 |
| | $E_{i+1}$ | = | evaporation during time step i to i+1 |
| | $Sp_{i+1}$ | = | basin spillage during time step i to i+1 |
| | $WH_{i+1}$ | = | water harvested from wetland during time step i to i+1 |
| | $WP_{i+1}$ | = | volume pumped into wetland during time step i to i+1 |

The time step used in the model changes depending on the water depth and the inflow volume. If the water depth is above any of the wetlands outflow structures and/or inflow occurs, on a particular day, then the simulation is conducted using an hourly time step, otherwise a daily time step is used. The stage storage relationships discussed in Section 4.5 are used to calculate the water volume and depth during the simulation.

### 4.6.1 Inflow Characteristics

The long-term simulation model integrated into the design framework requires hourly inflow volumes and pollutant concentrations. These inflow characteristics could be taken from actual data collected from monitoring stations or they could be artificially derived using a model. There are many rainfall runoff and pollutant wash-off models discussed in the literature. Selection of the appropriate model will be based on the characteristics of the catchment and the amount and quality of data available.

4.6.1 a) Water Quantity

Unless there is sufficient observed data available at or near the proposed wetland site a model must be used to estimate catchment runoff volumes. The simplest method used in long-term simulations is a runoff coefficient model

$$F = CAR \tag{4-6}$$

Where
$F$ = flow ($m^3$)
$C$ = runoff coefficient
$A$ = catchment area ($m^2$)
$R$ = rainfall depth (m)

The runoff coefficient can be calculated from observed data or estimated from a similar catchment. More detailed models such as XP-Aqualm (Xp-Software, 1995), XP-Rafts (XP-Software, 1997), HSPF (Johansson et al, 1984) or SWMM (Huber & Dickinson, 1988) could also be used for sites with adequate data. These models use rainfall and evaporation data, combined with some representation of the rainfall losses and soil moisture store to simulate a time series of runoff volumes.

The models contained in this section are designed primarily for daily time step simulations. If adapted to produce hourly output, the runoff may need to be routed through the streams and drains in the catchment

#### 4.6.1 b) Water Quality

The methods used to simulate pollutant export from catchments range in complexity from simple event mean concentration estimates to complex process based mathematical models. The choice of model is influenced by the data availability and the data input requirements of the wetland water quality model incorporated into the wetland mass balance. The form of the output from the pollutant export model must match the data input requirements of the wetland water quality model. If there is very little or no data available for a catchment, a simple estimate of the event mean concentration may be the only suitable method. This estimate could be calculated from sampling data collected from the catchment to be modelled or a similar catchment. If there is sufficient data available then a pollutant wash-off model such as those described in Section 2.2.2. could be used.

#### 4.6.1 c) Basin Outflow and Spillage

Outflow from the basin's outlet structures is calculated using the discharge equations discussed in Section 2.2.8. Basin spillage occurs when the water height is above the wetland banks. The model assumes that the entire water volume, above the banks, spills during a single daily time step.

#### 4.6.1 d) Evaporation

Evaporation occurs from the surface of the wetland basin. The Bureau of Meteorology (BOM) is officially responsible for the collection of rainfall data in Australia. They have adopted the class A pan as the standard measurement of evaporation. The depth of actual evaporation from a large open water body, such as a reservoir or lake, is generally between 70% and 80% of the corresponding class A pan evaporation depth (Chow et al, 1988). A constructed wetland is a much smaller water body and hence their evaporation depths will be higher. The model assumes that the actual evaporation depth from a wetland will be 90% of the class A pan evaporation depth.

Daily evaporation data is used in the mass balance, as this is the smallest time increment collected by the BOM.

4.6.1 e) Water harvesting

Water harvesting from a wetland is usually carried out by pumping. The pumping rate and the time of day when pumping can occur is determined by the requirements and logistics of the water reuse scheme. Usually water is harvested for a restricted period during each year, typically during the wet season if ASR is being performed. Water harvesting takes place when the quality of the water satisfies the appropriate reuse guideline and the water level is above the pumping height. These two requirements are checked every time step during the simulation to see if water harvesting can occur. In some wetlands, it may be desirable to keep the water level above a certain height, for aesthetic or recreational purposes. If this is the case, it may be necessary to top the wetland up, during the dry season, with mains or reclaimed water. If topping up of the wetland is required then the water level is checked during every time step. At any stage when the level is below the set minimum water height, topping up will occur.

### 4.6.2 Water Quality Modelling

The modelling of water quality in and out of the wetland is a critical part of long-term simulation modelling. The quality of water in the wetland during the simulation determines the outflow water quality and the times when water harvesting occurs.

As with the other models incorporated into the design framework the selection of a water quality model is largely based on data availability and the types of pollutant removal processes active in the wetland. Section 2.2.4 contained a description of a number of models used for water quality simulation in stormwater wetlands.

If a single event mean concentration estimate is used to model water quality inflows and a variety of pollutant removal processes are present, than a simple water quality simulation model such as the generic pollutant removal curve method should be used. Sediment settling models are appropriate only for wetlands where the only pollutant removal present will be sedimentation (ie. detention basins). The use of more detailed models is only feasible if there is a great deal of data available for the proposed wetland.

## 4.7 Summary

The optimal wetland design framework makes use of a flood simulation model and a long-term simulation model contained within a simple GA. The selection of the rainfall-runoff and water quality models required by the simulation models will be based on data availability, the treatment train chosen and the catchment and site characteristics. The construction, operating and maintenance and penalty costs used in the framework will be case specific.

# Chapter 5 Case Study: Adelaide's South Parklands Wetland System

## 5.1 Introduction

The wetland system proposed for Adelaide's South Parklands was planned to perform all three of the wetland functions discussed in this thesis; flood mitigation, pollutant removal and water harvesting for reuse. Since this enabled the trade offs between the respective functions to be examined, it was chosen as a case study.

## 5.2 Adelaide's South Parklands

### 5.2.1 Site Description

The South Parklands occupy the southern portion of a ring of parklands surrounding the City of Adelaide. They are bounded by South Terrace and Greenhill Road, to the north and south, and Anzac Highway and Fullarton Road, to the west and east. They are crossed by four major arterial roads and three smaller roads, splitting the South Parklands into eight separate areas, referred to as parks 16, 17, 18, 19, 20, 21, 21w and 22 (see Figure 5-1). Combined, these areas occupy 150 hectares of gently sloping, low-lying terrain. The vegetation cover is sparse, with the exception of a moderately dense region of trees in park 17. A number of bicycle and pedestrian paths cross the South Parklands, and stormwater channels also flow through the area.

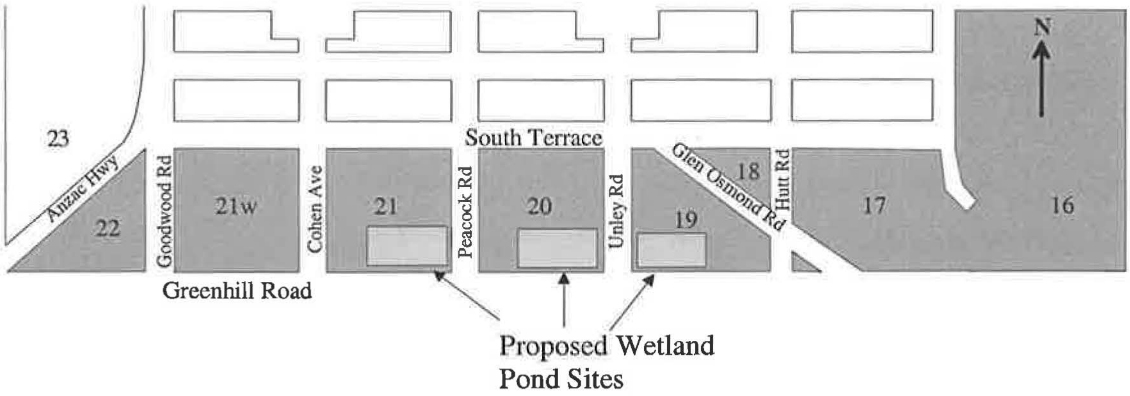



**Figure 5-1 Adelaide's South Parklands**

5.2.1 a) Park Usage

The South Parklands are a popular area for recreation. They contain facilities for a range of sports, as well as gardens and open space for more passive pursuits. The three parks that were identified as possible sites for the wetland were parks 19, 20, 21 and 21W.

*Park 19* contains the Himeji Gardens in the northeastern corner, a few run-down tennis courts near Glen Osmond Road, a children's playground and picnic area east of the tennis courts and numerous cricket pitches and soccer fields scattered in other areas.

*Park 20* is well used by a number of sporting groups. Pulteney Grammar School maintains several cricket and football ovals in the northern portion of this park. There are also tennis courts, soccer fields, a petanque club, and a BMX track. The park is also popular for walking and jogging and is used by the Adelaide Harriers Athletics Club.

*Parks 21 and 21W* contains Veale Gardens, tennis courts and numerous soccer fields and cricket ovals. The park is also used by the South Australian Athletics Club and the Southern Soaring League, a model aeroplane club.

### 5.2.2 Drainage

Parklands Creek is the major tributary flowing through the South Parklands. It enters the region at the southeast corner of Park 16 and winds its way through parks 17, 18, 19, and park 20 where it leaves, passing under Greenhill Road between Unley Road and Peacock Road. It then flows through Unley and Goodwood, where it becomes Keswick Creek, and eventually flows into the Patawalonga River. The portion of Parklands Creek contained within the South Parklands is an unlined open channel. There are also some small drains located in parks 20 and 21, collecting runoff from the City, which flow into Parklands Creek. These drains are also unlined open channels.

### 5.2.3 Geology

The soil profile of the South Parklands area is characterised by a thin layer of calcareous mantle that extends to a depth of 4 m. Underneath this is a hard highly plastic clay, known as Hindmarsh Clay. This layer varies in depth from 25 m, in the East, to 2 m near Peacock Road (see Figure 5-2) (Selby & Lindsay, 1982).

There are five known aquifer formations beneath the South Parklands. The shallowest of these, Carisbrook Sand (aquifer 1A) is approximately 10 m thick and is located beneath the Hindmarsh Clay layer. This layer is confined and composed predominantly of fine to medium grained sands. Aquifer 1 is fractured calcareous sandstone located 13 m below the surface, in the Western part of the South Parklands only. This aquifer is also confined. Aquifers 2 and 3 are very similar in their composition, both confined by silt or siltstone and composed of fine to medium grained sand. Aquifer 2 is 10 m thick and is located approximately 45 - 55 m below the surface. Aquifer 3 is smaller, only 5 m thick, and is located at a depth of around 60 - 65 m. The deepest known aquifer is South Maslin Sand, confined by lignitic clay below and a mixed layer on top made up of clayey sand and limestone. The aquifer is composed of fine sands and the water quality is poor (Selby & Lindsay, 1982).

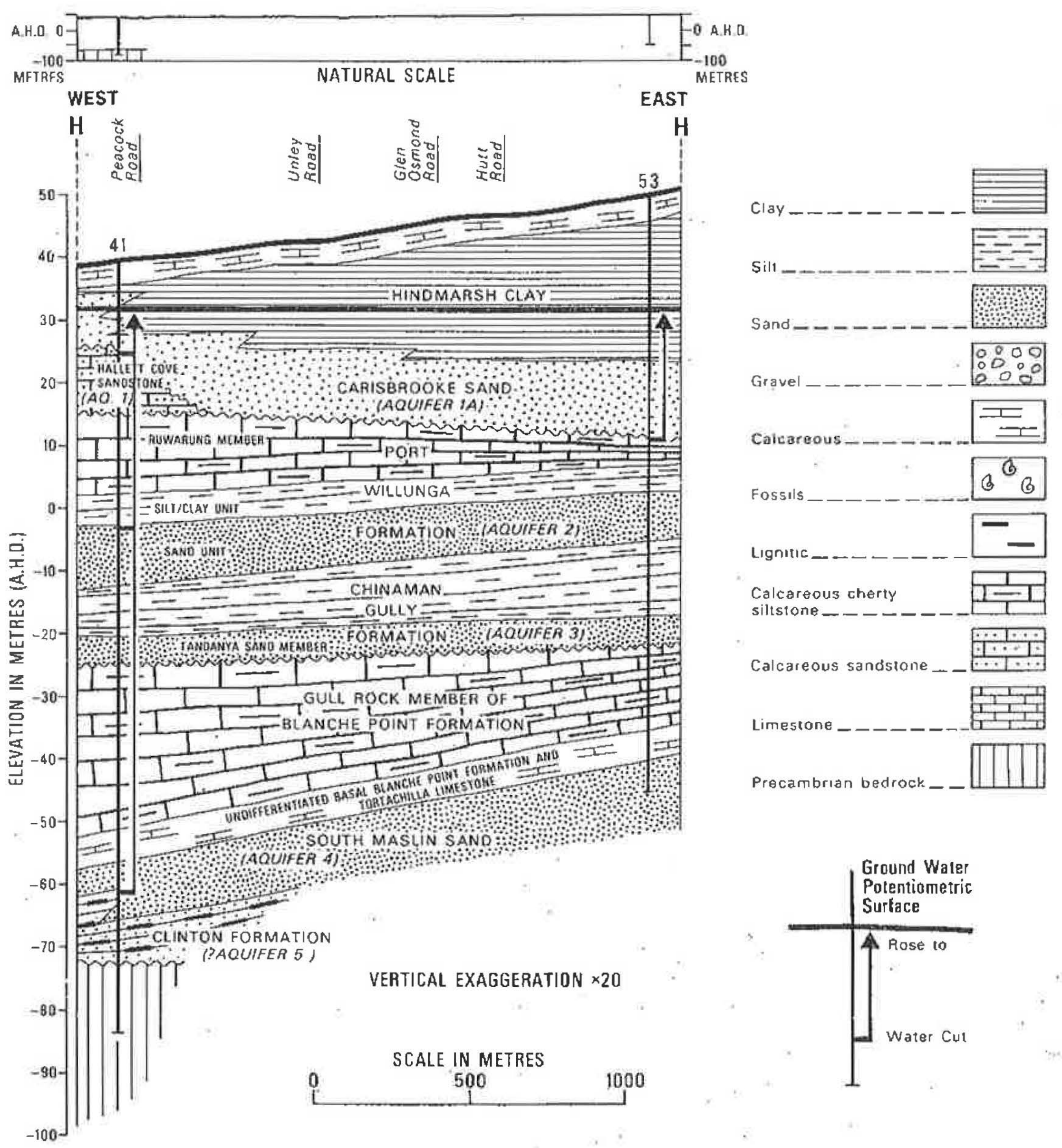



**Figure 5-2 Geology of the South Parklands (Source: Selby and Lindsay (1982))**

### 5.2.4 Irrigation

The City of Adelaide irrigates 115 ha of the Parklands with automatic sprinkler systems, using approximately 19.5 ML of mains water per week. The irrigation season runs from the middle of October until the end of March each year. No irrigation takes place for a month over Christmas and New Year. The sprinklers operate on average 3 nights a week between 10pm and 6.30am. There are a number of manually operated sprinkling systems in the Parklands. The total area irrigated and the volume of water used by these sprinklers varies significantly from year to year. Currently the City of Adelaide is not charged for mains water (Shaw, 1999).

## 5.3 Parklands Creek Catchment

The Parklands Creek Catchment occupies an area of 9.1 $km^2$. It encompasses some foothills of the Mount Lofty Ranges, the residential areas located south of Greenhill Road and east of Glen Osmond Road, and most of the city centre's south-west corner (see Figure 5-3). The majority of the catchment is urban land, except for the eastern-most portion, located in the foothills, which is rural. The land generally slopes from east to west, with a gentle slope, except for the region in the foothills of the Mount Lofty Ranges, which is characterised by steep slopes. There are small pockets of light commercial areas throughout the catchment and some light industry located in the city centre, but the predominant land use in the urbanised portion of the catchment is residential.

**Figure 5-3 Locations of Water Quality and Hydrological Monitoring Sites**


Key

| | | | |
|---|---|---|---|
| A | - | Roberts Street flow gauge (BM023119) | 1 - PCWMB water quality sampling station AW504583 |
| B | - | Victoria Park flow gauge (AW504907) | 2 - PCWMB water quality sampling station AW504575 |
| C | - | Glenside rain gauge (AW504906) | 3,4,5 & 6 Waterwatch water quality sampling stations |
| D | - | Beaumont rain gauge (BM023114) | (green line) - South Parklands wetland catchment boundary |
| E | - | Kent Town rain gauge (BM023090) | (blue line) - Parklands Creek (major drainage paths) |

## 5.4 Previous Studies

B.C. Tonkin and Associates conducted an investigation into the potential sites for retardation basins on Parklands Creek, downstream of Fullarton Road. They proposed the construction of a 70 ML basin between Fullarton Road and Beaumont Street. This basin was designed to attenuate the 1 in 50 year flood event (B.C.Tonkin, 1974).

Consultants, Wood, Bromley, Carruthers & Mitchell Pty. Ltd. conducted a drainage study of Brownhill, Glen Osmond, Parklands and Keswick Creeks in 1984 (WBCM, 1984). The study involved performing a number of detailed hydraulic simulations, using the RORB model (Laurence & Mein, 1983), to analyse the performance of the catchment that existed at that time. Based on results from these simulations they proposed several measures to help alleviate flooding problems. One of the measures proposed was the construction of a 150 ML detention basin located upstream of Greenhill Road. Construction of this basin would reduce peak flows out of the South Parklands for a 1 in 100 year flood event from 12.1 $m^3/s$ to 4.8 $m^3/s$ and was estimated to cost $700,000.

B.C. Tonkin & Associates conducted a feasibility study for the construction of a wetland system for the South Parklands in 1995 (B.C. Tonkin, 1995). The proposed wetland system located in parks 21 and 21W consisting of two treatment wetlands and a recreational lake. The full supply volume of the wetland system was 112 ML with an additional 102 ML available for flood storage. A diversion weir was located on Parklands Creek between Unley Road and Greenhill Road to limit flows to Parklands Creek to 4.8 m/s, and direct all other flows to the wetland. An ASR system was connected to the wetland, with the water being used to irrigate grassed areas in the parklands. The wetland was designed to limit the 100 year average recurrence interval (ARI) peak flow to 4.8 $m^3/s$. The estimated cost of the wetland and ASR system was $3.8 million.

The 1995 study (B.C. Tonkin, 1995) was revised in 1998 due to community concerns about the loss of sporting and recreational areas in the South Parklands that would have occurred with the three basin wetland system (Begg, 1998). The new study proposed the construction of five treatment wetlands located in parks 16a, 17, 19, 20, and 21W and a recreational lake located in park 21, inter-linked by the existing creek system. The total volume of the five treatment wetlands was 105 ML. The volume of the recreational lake was not specified. The wetlands system could provide significant flood protection benefits but additional works were required to reduce peak flows for a 1 in 100 year flood. The proposal also included the construction of an ASR scheme. The estimated total cost of the system was $4.31 million.

## 5.5 Functional Requirements of the Wetland System

The South Parklands Wetland System was proposed to perform three hydrological functions; flood mitigation, pollutant removal and water reuse through ASR (CAC, 1995).

The Patawalonga Catchment Management Plan suggested a flood mitigation objective standard of a 1 in 50 year event would be adequate for Parklands and Keswick Creeks (B.C. Tonkin, 1997).

Wood, Bromley, Carruthers & Mitchell Pty. Ltd. WBCM (1984) predicted that a storm with an ARI of 50 years in Parklands Creek, would cause $266,000 (June 1980 $) worth of damage, including direct damage, indirect damage and clean up costs. The values for the 1 in 100 and 1 in 200 year flood events were $800,000 and $1,486,000 respectively. The report recommended that the ARI 100 year flood event should be adopted as the required standard for flow along Parklands Creek. This standard was used in all studies conducted for the South Parklands wetland and was adopted for this research.

The Patawalonga Catchment Management Board adopted two water quality standards for creeks and rivers in the catchment, based on the ANZECC water quality guidelines (ANZECC, 1992). The microbiological quality of the water should not exceed the secondary human contact guideline, (EPA, 1998), and the physical and chemical quality of the water should be of sufficient standard to ensure the protection of aquatic ecosystems (B.C. Tonkin, 1997).

An ASR system was to be associated with the wetlands proposed in the 1995 and 1998 studies (Begg, 1998; B.C. Tonkin, 1995), enabling reuse of stormwater harvested from the wetland pond. This water was to be used for irrigation of the parklands surrounding the City of Adelaide, (CAC, 1995). The quality of the water extracted from the aquifer was required to be suitable for primary human contact, (EPA, 1998). This standard was adopted because spraying would be used, the parklands are very accessible, and hence the risk of spray being exposed to people would be quite high. The quantity of water required for aquifer recharge was not specified in the initial brief or in any subsequent reports published on the South Parklands wetland system.

Other non-hydrological functions the wetland was required to perform included increased recreational potential for public use of the South Parklands and the use of the wetland as an environmental educational resource (CAC, 1995). It was very difficult to integrate these functions into the optimisation procedure used in this study, so they were excluded.

### 5.5.1 Flood Simulation

The flood simulation model developed in this study assesses the wetland system's ability to satisfy three performance requirements:

1. Reduction of peak flows for the 1 in 100 year flood event (extreme flood event) to below the carrying capacity of the receiving channel. The current carrying capacity of Parklands Creek in the South Parklands is 5 $m^3/s$. The output statistic relevant to this performance criterion is the peak outflow rate. An ARI of 1 in 100 years has been recommended for extreme flood protection in all feasibility studies conducted for the South Parklands wetland (WBCM, 1984; BC Tonkin, 1995; Begg, 1998).
2. Containment of a 1 in 100 year flood event (extreme flood event) within the levee banks constructed around the wetland system. The output statistic relevant to this performance criterion is basin spillage over the levee banks.
3. Containment of a 1 in 10 year flood event (design flood event) within the wetland ponds. This performance requirement was conceived to protect the wetland banks from frequent overflowing which might cause erosion. The output statistic relevant to this performance criterion is basin spillage from the wetland ponds.

The simulation conducted on each wetland design comprised two parts. In the first part of the simulation a flood hydrograph corresponding to the 1 in 100 year flood event was passed through the wetland to assess performance requirements 1 and 2, and in the second part a flood hydrograph corresponding to the 1 in 10 year flood event was used to assess performance requirement 3.

### 5.5.2 Long-Term Simulation

The long-term simulation model integrated into the optimal design framework is used to assess the following performance requirements

1. Reduction in mean outflow pollutant concentrations from the wetland to below a specified level. The water quality requirement for downstream discharge will be discussed in Section 5.8.2.
2. To maximise the volume of water harvested from the wetland basin during the simulation period. This requirement will be discussed further in Section 5.6.3.
3. To maintain a permanent pond, all year round, for recreational use.

## 5.6 Wetland System Configuration

A simple one-basin on-stream wetland system was chosen for the case study. The primary outlet from the basin is a trapezoidal weir. An ASR pump off-take is located at the downstream end of the wetland, providing water for 5 ASR bores. A levee bank is located 5 m from the edge of the wetland basin (see Figure 5-4). The levee bank will be constructed for extreme flood mitigation and will be 1 m high. The geometry of the basin is as described in Section 4.5.2. Although it may not be practical to construct a one-basin system given the available land area in the South Parklands, this simple wetland system configuration was chosen to provide a demonstration of the optimisation procedure, and to enable a detailed analysis of the trade-offs between different wetland functions to be undertaken.

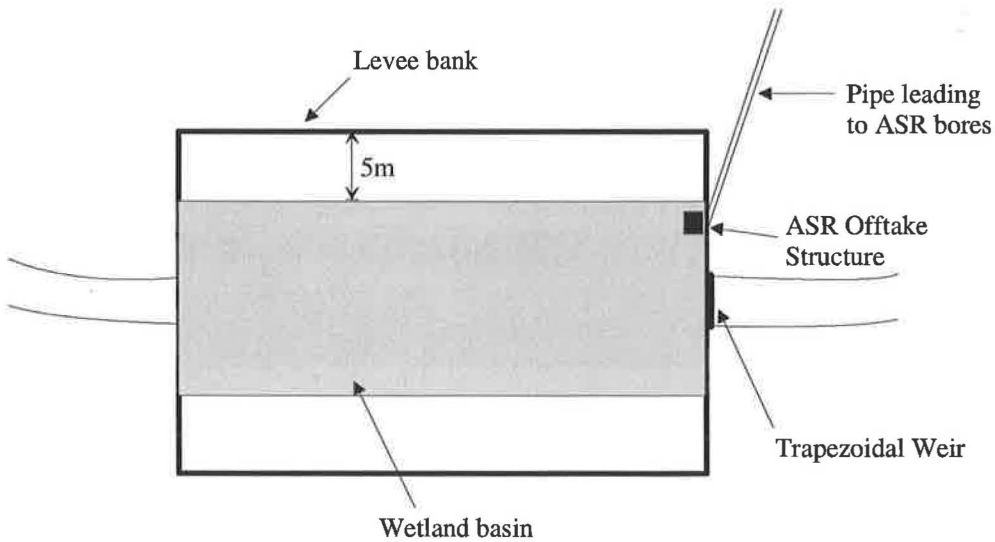
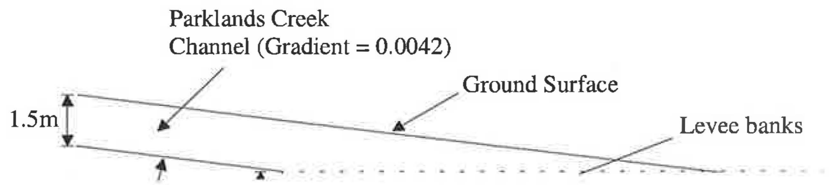



**Figure 5-4 Wetland Basin Top-View**

A cross-section of the basin is shown in Figure (5-5). The height of the levee banks above the ground surface, at the downstream end of the basin, will be different for every wetland volume.

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Section 5.6, page 74, figure 5-5, change:

Figure title: Wetland Basin Cross-Section

To:

Figure title: Conceptual Wetland Basin Longitudinal Section

-

Section 5.6, page74, paragraph 1, change:

A cross-section of the basin is shown in Figure 5-5. The height of the levee banks above the ground surface, at the downstream end of the basin, will be different for every wetland volume.

To:

A longitudinal section of the basin is shown in Figure 5-5. The height of the levee banks above the ground surface, at the downstream end of the basin, will be different for every wetland volume.

aquifer storage and recovery were promising (Gerges & Howles, 1995). Average yield rates of 10 L/s were achieved from the first tertiary aquifer (Hallet Cove Sandstone). The average salinity of this aquifer was relatively low, between 1,000 and 2,500 mg/L (Gerges & Howles, 1995). Subsequent investigations, conducted by Mines and Energy, South Australia, found the maximum injection rate possible was 5 L/s and extraction rates should be restricted to 2.5 – 3 L/s (AACM, 1999; Thomas, 1998). These low injection (5 L/s) and extraction (2.5 L/s) rates make ASR marginal for this site, but were used in order to give realistic results.

Water extracted from the aquifer will be used to water sections of the Adelaide parklands. Using the parklands irrigation data presented in Section 5.2.5, the average water requirement per hectare was 3.4 ML/year. The area of parkland irrigated was dependent on the total volume of water available for extraction. This volume was assumed to equal 90% of the total volume injected. It was assumed that 10% of the injected volume would be unsuitable for irrigation due to mixing with the saline groundwater (Dillon & Pavelic, 1996).

Injection of water into the aquifer took place when the retention time of water stored in the wetland basin was greater than 10 days. The time of year that injection took place was limited to the months when irrigation of the Parklands did not occur. A period of 50 days before the irrigation season, during which no aquifer injection or extraction took place, was also allowed for to ensure the die-off of any bacteria present in the injected water (Dillon & Pavelic, 1996). Based on these restrictions, injection could only occur between April and August, inclusive, of every year.

## 5.7 Rainfall Runoff Modelling

### 5.7.1 Hydrological Monitoring

The Bureau of Meteorology maintains two rain gauges and two flow gauges in the catchment of Parklands Creek (see Figure 5-3). The first rain gauge is located at Beaumont in the upper reaches of the catchment at the base of the Adelaide Hills, and the second is in Glenside, close to the centre of the catchment. A flow gauge is situated in the southwest corner of the South Parklands near Victoria Park racecourse, and the other gauge is located at Roberts St., 100 m south of the South Parklands. All gauges (rainfall and flow) were put in place between 1994 and 1995, to improve the accuracy of flood predictions for Keswick Creek. A rain gauge is located at the Bureau of Meteorology's head office in Kent Town, approximately 1 km north of the catchment and 2 km from the Glenside rain gauge. This is the main meteorological site for Adelaide, and rainfall data has been recorded there since 1977.

Parklands Creek is concrete lined, apart from several sections within grassed areas in the upper reaches of the catchment and in the South Parklands. The length of channel in the South Parklands between Fullarton Road and Greenhill Road has a cross-sectional area of between 3 and 4 $m^2$ and an average bed slope of 0.0042. The bank-full carrying capacity of this channel is 5 $m^3/s$ and the culverts underneath the roads crossing Parklands Creek have carrying capacities of between 7 and 8 $m^3/s$. The culvert running underneath Greenhill Road is capable of carrying flows of up to 8 $m^3/s$. The section of creek upstream of Fullarton Road is steeper and hence flows are carried quite quickly to the South Parklands. Downstream of Greenhill Road, carrying capacities vary from between 5 and 10 $m^3/s$, although flows of 5-7 $m^3/s$ would result in the flooding of a small playground on Young Street (WBCM, 1984).

### 5.7.2 Flood Simulation

The flood hydrographs used in the flood simulation were derived by Kemp (1999) using the Rainfall Runoff Routing (RRR) model that is an adaptation of the RAFTS model (XP Software, 1997). The RRR model separates the channel of the catchment into 10 reaches of equal length with a linear channel storage (Equation 5-1) in each reach. The areas contributing to each of the reaches are assumed equal.

$$S = 3600kQ \tag{5-1}$$

Where
S = storage ($m^3$)
k = lag of the channel reach (s)
Q = reach outflow ($m^3/s$)

Contributions from any number of separate hydrological processes can be added at the downstream end of each reach before routing through the channel storage (Figure 5-6). Each of the hydrological processes is modelled with ten equal sub-catchments each with a storage given by

$$S = 3600k_pQ^m \tag{5-2}$$

Where
S = storage ($m^3$)
$k_p$ = runoff process lag (s)
Q = sub-catchment process outflow ($m^3/s$)
m = non-linearity storage exponent

Each of the hydrological processes has an initial and continuing or proportional loss associated with it.

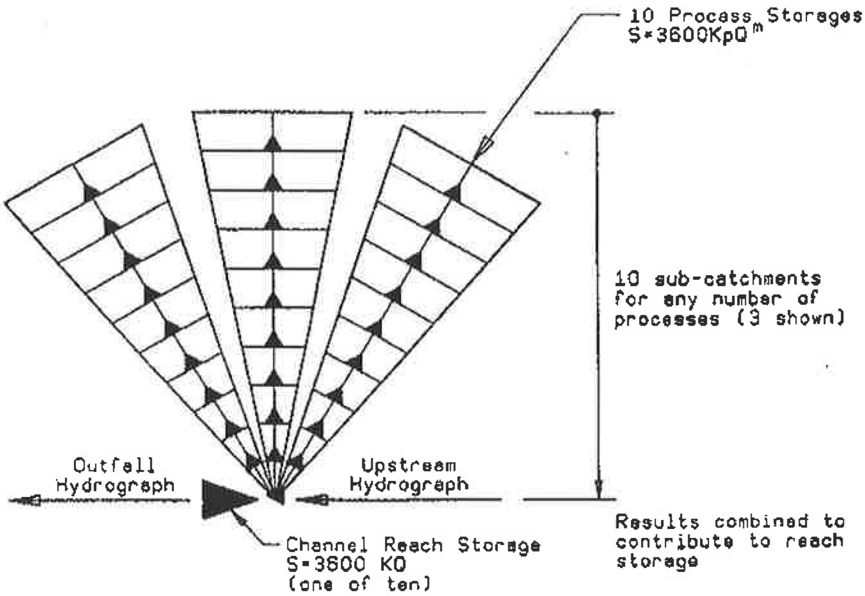


**Figure 5-6 The RRR Model (Source: Kemp, 1999)**

Hydrographs were calculated for storm events corresponding to ARI's of 1 in 1 year, 1 in 2 years, 1 in 5 years, 1 in 10 years, 1 in 20 years, 1 in 50 years and 1 in 100 years for both 3 hour and 6 hour storm events (see Figures 5-7 & 5-8). A storm of six hours duration resulted in the highest peak flow in the South Parklands for ARI's of 1 in 50 years or greater, however for ARI's of 1 in 20 years and below, the peak flow occurred for a three hour duration storm (Kemp, 1999).

Some preliminary investigations were conducted into the outflow behaviour of the wetland

Section 5.7.2, page 77, paragraph 2, replace:

Some preliminary investigations were conducted into the outflow behaviour of the wetland during different duration storms corresponding to a 1 in 20 year ARI. Several combinations of basin sizes, weir heights and weir lengths were studied. It was found that a 6-hour duration storm resulted in the highest peak storage volume for all basin design combinations. Similar findings were discovered for storms corresponding to the 1 in 50 ARI and the 1 in 100 ARI. The 6-hour duration 1 in 20 year ARI and the 1 in 100 year ARI storm events were used in the flood simulations.

With:

The 6-hour duration 1 in 100 year ARI flood event and the 3-hour duration 1 in 10 year ARI flood event were used in the flood simulation.

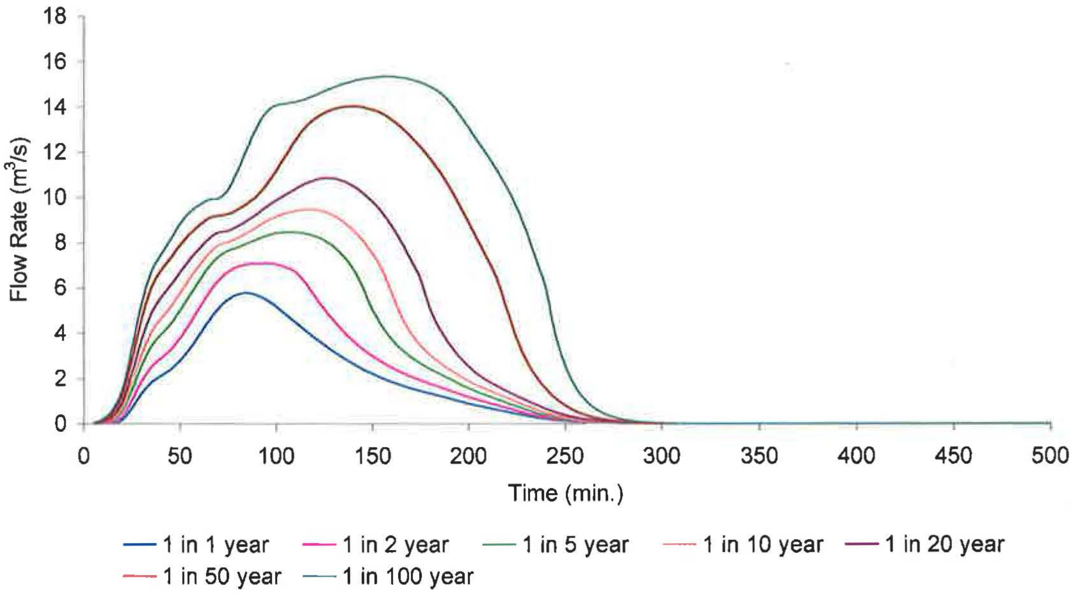


**Figure 5-7 Flood Hydrographs for Parklands Creek, at the Roberts St. Flow Gauge (3-Hour Duration Storm Events) (Kemp, 1999)**

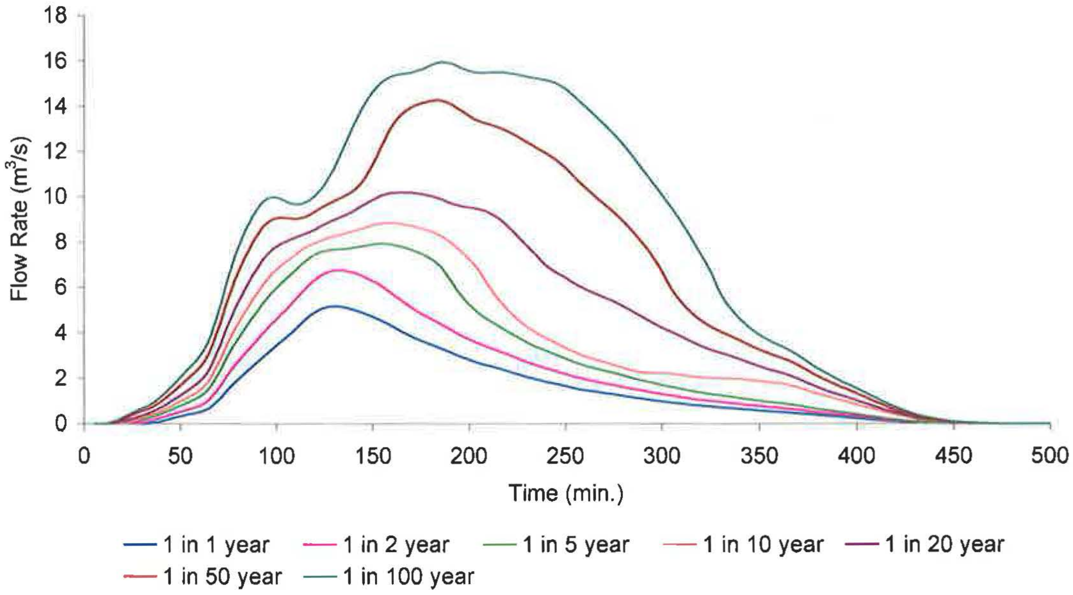


**Figure 5-8 Flood Hydrographs for Parklands Creek, at the Roberts St. Flow Gauge (6-Hour Duration Storm Events) (Kemp, 1999)**

### 5.7.3 Long-Term Simulation

The choice of the rainfall-runoff model used to calculate inflow volumes for the simulation was limited because the length and the quality of flow and rainfall data records available for the Parklands Creek catchment were inadequate (1994 - 1999) to use a detailed model.

An initial loss, continuing loss model was used to calculate inflow volumes for the simulation. The initial loss and continuing loss parameters were determined after comparison between hourly rainfall data from the Glenside site and hourly flow data from the Roberts St. flow gauge. Glenside was chosen because it is the closest rain gauge to the centre of the catchment.

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Section 5.7.3, page 79, paragraph 2, change:

68 storms of varying sizes and durations, between the 22 February 1995 and the 24 October 1998, were analysed. Rainfall depths, measured at Glenside, were assumed to fall uniformly over the entire catchment. Rainfall volumes calculated by multiplying the total rainfall depth by the catchment area were plotted against the recorded flow volume (see Figure 5-9). Twenty storms between the 29 May 1995 and the 24 October 1995 were removed because of inconsistencies with the rest of the data. The flow volumes recorded during this period were much higher than volumes recorded from similar sized storms in other periods. A line was then fitted to the remaining 48 data points using linear regression (see Figure 5-10).

To:

68 storms of varying sizes and durations, between the 22 February 1995 and the 24 October 1998, were analysed. Rainfall depths, measured at Glenside, were assumed to fall uniformly over the entire catchment. Rainfall volumes calculated by multiplying the total rainfall depth by the catchment area were plotted against the recorded flow volume (see Figure 5-9). Twenty storms between the 29 May 1995 and the 24 October 1995 were removed because of inconsistencies with the rest of the data. The flow volumes recorded during this period were much higher than volumes recorded from similar sized storms in other periods. There was also doubt cast on the accuracy of the pressure transducer used during this period (Marshall, 1999). A line was then fitted to the remaining 48 data points using linear regression (see Figure 5-10).

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Section 5.7.3, page 79, paragraph 3; comment:

The initial loss used in the long-term simulation model, 0.15 mm, was incorrect the value should have been 0.94 mm. It was found that this error had a minimal impact on the results.

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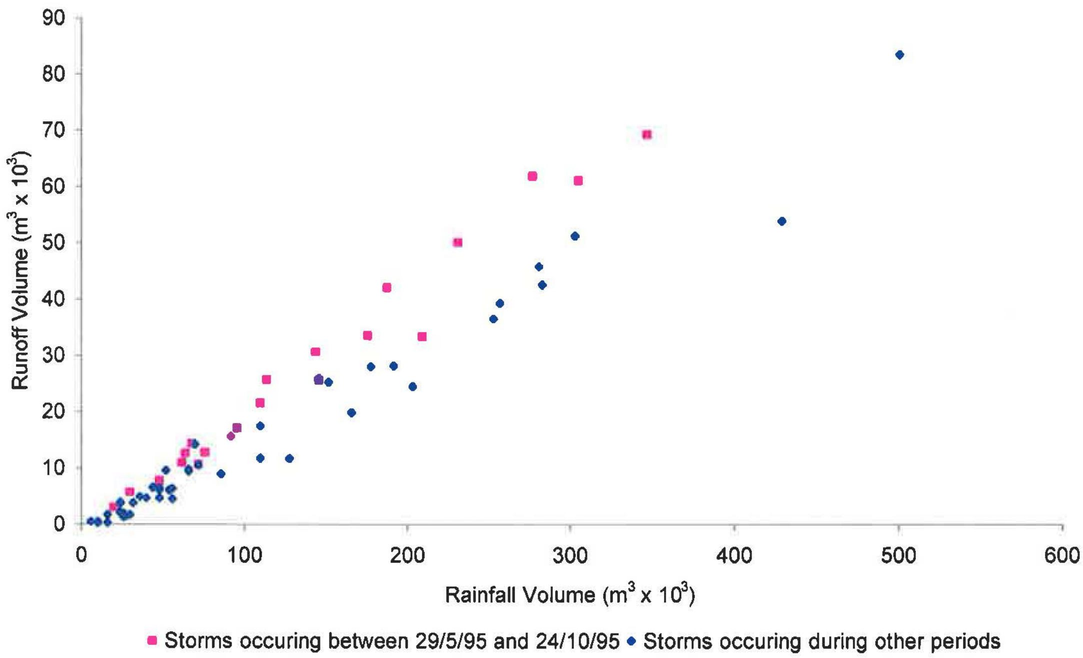


**Figure 5-9: Rainfall Volume Versus Flow Volume for All 68 Storms**

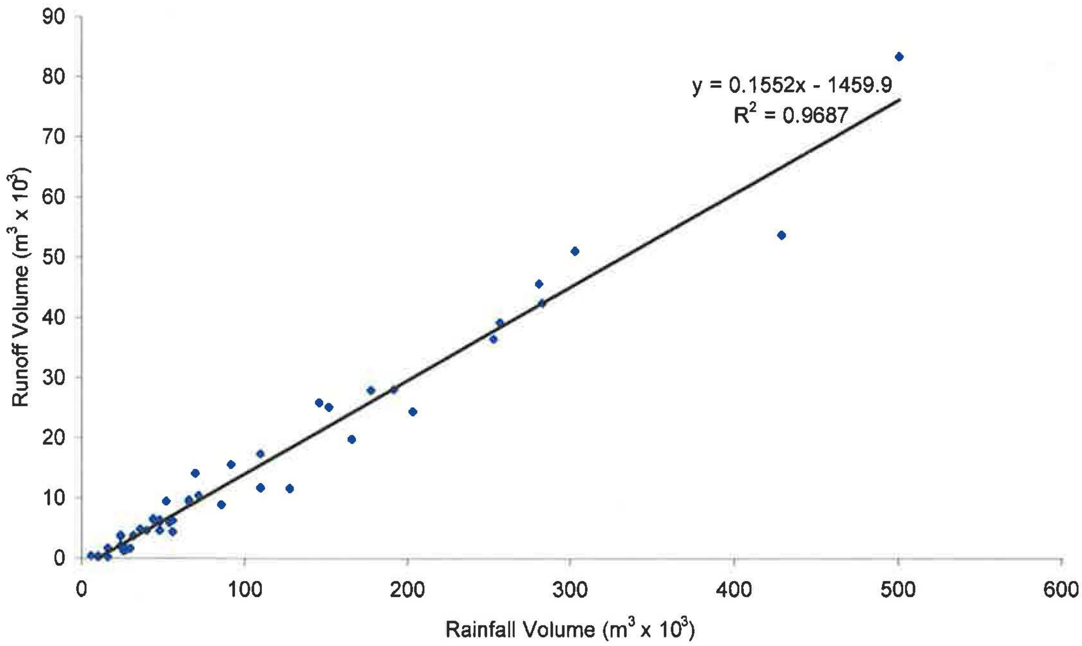


**Figure 5-10 Rainfall Volume Versus Flow Volume for The 48 Storms**

The values for initial and continuing loss are easy to implement for a daily simulation time step, however when using an hourly step, flow routing effects have to be taken into account. The

Parklands Creek catchment is relatively large and rainfall landing in the upper portions will take longer than an hour to arrive at the flow gauge as stormwater flow. In order to simulate this effect, the following proportionate flow model was used

$$F_t = 10 \times c \times A \times \sum_{i=0}^{n} \alpha_i R_{t-i} \tag{5-4}$$

| | | | |
|---|---|---|---|
| Where | $F_t$ | = | flow during hour t ($m^3$/hour) |
| | c | = | runoff coefficient (0.155) |
| | A | = | catchment area (997 ha) |
| | $R_{t-i}$ | = | rainfall falling during the hour (t-i) (mm) |
| | $\alpha_i$ | = | proportion of rainfall flowing past the rain gauge i hours after falling |
| | n | = | number of model parameters |

The $\alpha$ values split the hourly rainfall depths into six packets, the first packet arriving at the flow gauge during the hour that it fell, the second packet arriving at the flow gauge one hour after it fell, the third two hours after, etc. The sum of the $\alpha$ values was set equal to one, to ensure that hourly flow volumes remained consistent with daily flow volumes.

Initial loss was incorporated into this hourly model by subtracting the value from the rainfall depth in the first hour of each storm. A new storm was considered to start after eight consecutive hours of zero rainfall. Eight hours was considered a realistic length of time for the catchment to dry. After analysing the model described in Equation (5-4), it was found that changing the catchment drying time had little effect on the accuracy of the predictions.

Most of the time there is a small volume of base flow flowing down the channel at the Roberts St. flow gauge. The exact source of this flow could not be located, however it appeared to enter the channel after it left the South Parklands. As this baseflow would not flow into the wetland, it was eliminated from the data by reducing all flows below 0.03 $m^3$/s to zero. After removing all flows below this level only flow associated with storm events remained.

The proportionate flow model was calibrated against the transformed Roberts St. hourly flow data, for the period 29/10/1996 to 30/9/1998. This period was the longest continuous data set. The best values for the 3, 4, 5 or 6 parameters were found by minimising the RMS error between predicted and observed runoff values (see Table 5-1).

The 6 parameter model provided the best fit, however the difference in RMS between the 4, 5 and 6 parameter models was not large. The correlation coefficient between predicted and observed values for the 4 parameter model was the largest of these three models (0.757), but the difference between this and the correlation coefficient for the 6 parameter model (0.751), was negligible. The 4 parameter model was selected as the most appropriate model because it

-

Section 5.7.3, page 82, paragraph 1; comment:

Changing the initial loss to 0.94 mm had no impact on the parameters in the proportionate flow model used in the long-term simulation model.

**Table 5-1 RMS and Parameter Values for the Proportionate Flow Model**

| No. of parameters | RMS | $\alpha_0$ | $\alpha_1$ | $\alpha_2$ | $\alpha_3$ | $\alpha_4$ | $\alpha_5$ |
|---|---|---|---|---|---|---|---|
| 3 | 447.16 | 0.26 | 0.5 | 0.24 | - | - | - |
| 4 | 438.54 | 0.23 | 0.48 | 0.19 | 0.1 | - | - |
| 5 | 436.23 | 0.22 | 0.46 | 0.18 | 0.06 | 0.08 | - |
| 6 | 434.15 | 0.21 | 0.46 | 0.17 | 0.05 | 0.06 | 0.05 |

The predicted hourly flows, calculated from the 4 parameter model, were compared to the observed hourly flows for several storms (Figure 5-11, Figure 5-12, Figure 5-13 and Figure 5-14). The storms were chosen from different periods of the year and were typical of storms occurring in the period analysed. The modelled flows compared favourably with observed flows when the rainfall data matched well with the flow data (Figure 5-11) however, there was a tendency for the model to incorrectly predict the magnitude of peak flows (Figure 5-14). This error could have been attributed to using the Glenside rain gauge to represent rainfall over the whole catchment.

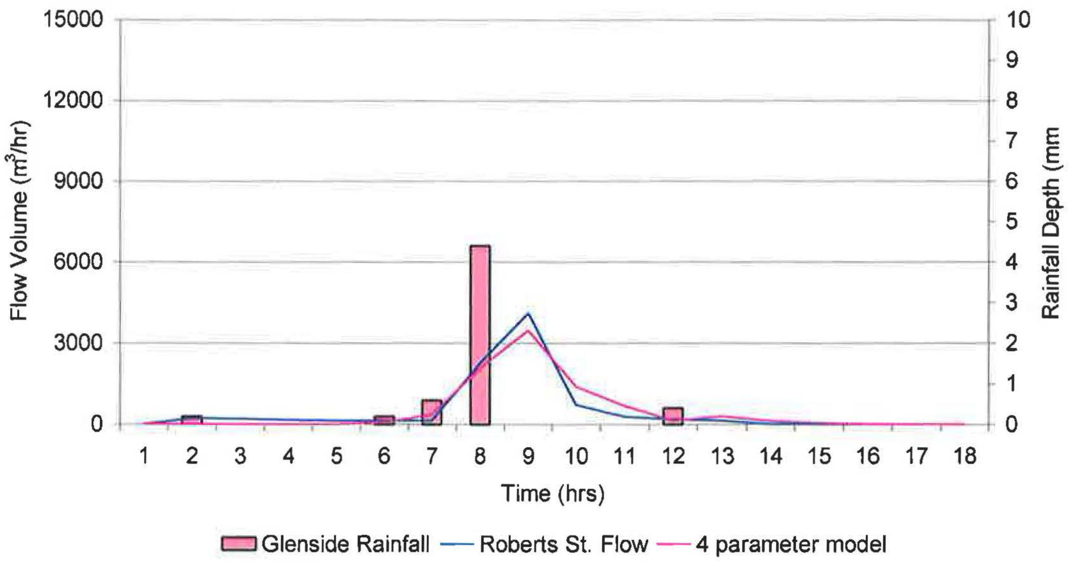


**Figure 5-11 Modelled Versus Observed Flow for Storm on 14/1/97 (Using Glenside Data as Input into Flow Model)**

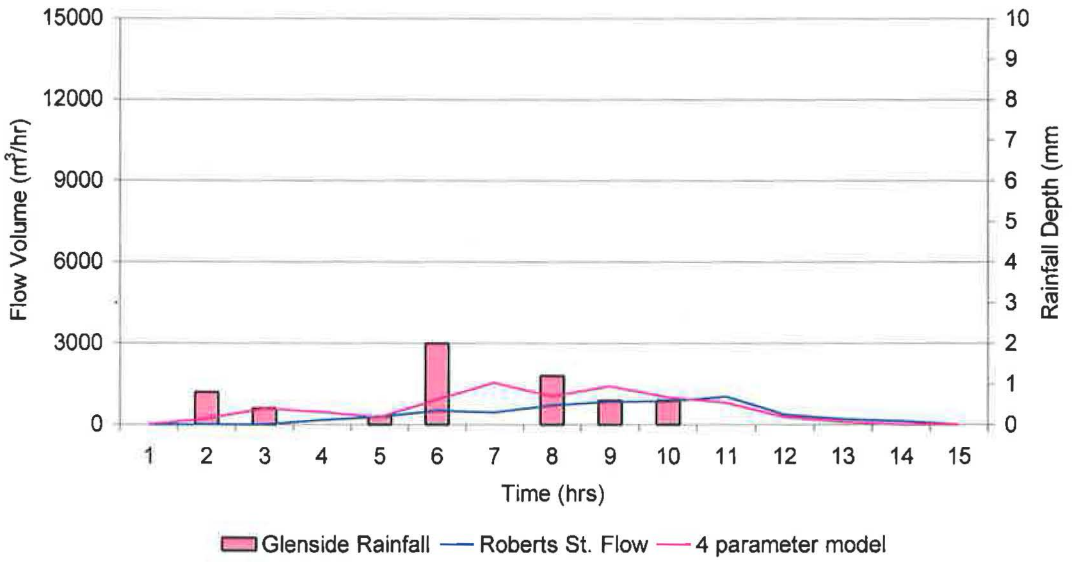


**Figure 5-12 Modelled Versus Observed Flow for Storm on 4/4/97 (Using Glenside Data as Input into Flow Model)**

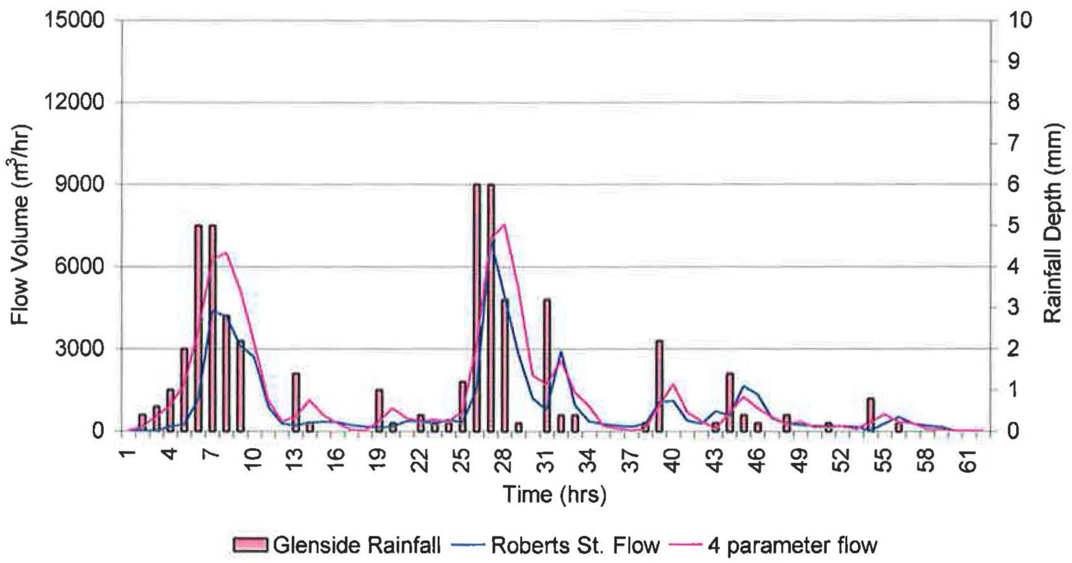


**Figure 5-13 Modelled Versus Observed Flow for Storm on 6/8/97 – 8/8/97 (Using Glenside Data as Input into Flow Model)**

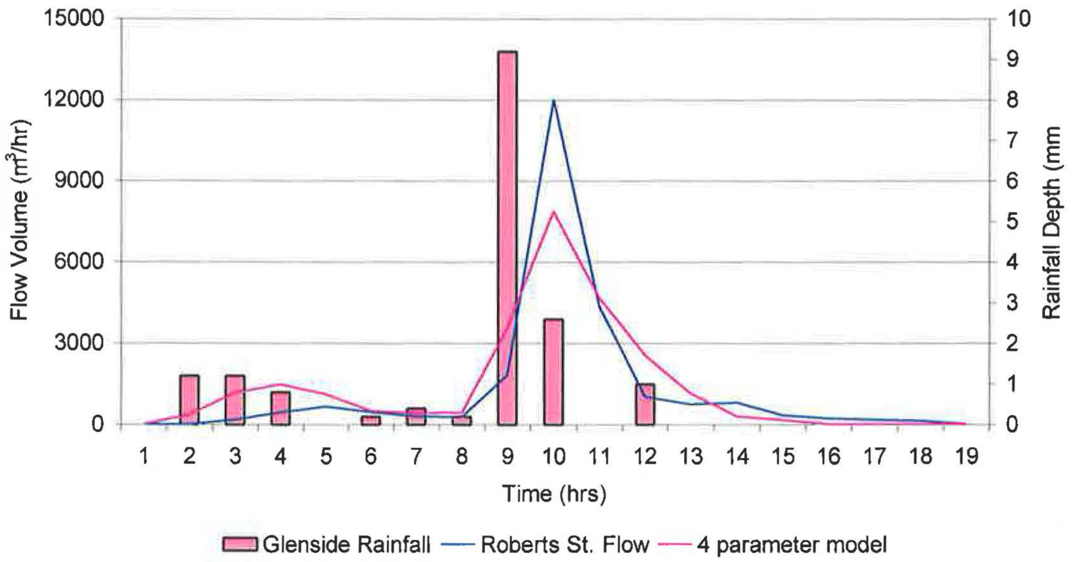


**Figure 5-14 Modelled Versus Observed Flow for Storm on 6/12/97 (Using Glenside Data as Input into Flow Model)**

Rainfall data has only been recorded at the Glenside site since 21/2/95. As a longer data set was required for the wetland simulation, rainfall data from the Kent Town pluviometer site was used. Eleven years of hourly data, from Kent Town, was used as input into the flow model, because the rainfall record prior to 1986 contained considerably more holes than the record after 1986. To fill holes that occurred in the post 1986 data set, rainfall values were cut and pasted from similar sized storms occurring at a similar time of year prior to the eleven year simulation period. The data also contained a small systematic rainfall error after July 1996. A string of 0.01 mm hourly rainfall depths followed most rainfall events. These strings of data noise ran from the hour following the rainfall event to 9am, when the pluviometer was reset. The noise was removed and some rainfall depths adjusted to ensure the resulting daily rainfall totals matched up with the daily data. Complete and unaltered hourly rainfall depths from the Kent Town record were plotted against the corresponding Glenside hourly rainfall depths, for the period 21/2/95 to 30/9/98 (Figure 5-15). Only hourly rainfall depths of greater than 0.2 mm were plotted on the graph. The outlier (depicted as a cross in Figure 5-15) was removed from the analysis because it distorted results. A line was fitted to the graph to determine a relationship between the two rainfall sets. It was found that on average the Glenside data was 6.34% larger than the Kent Town data. The R-squared of the line of best fit was 0.7108.

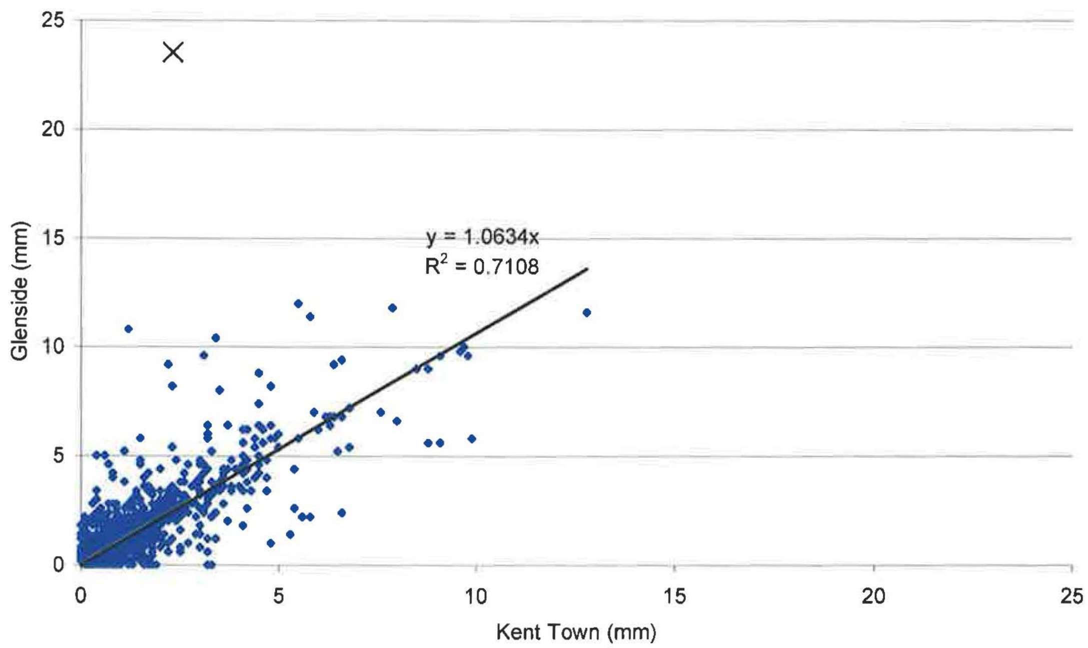


**Figure 5-15 Relationship Between the Hourly Rainfall Data Sets**

The whole eleven years of hourly rainfall depths from Kent Town were multiplied by 1.06 and used as input into the 4 parameter flow model described earlier. The resulting flows were compared to the observed Roberts St. data. The correlation between the resulting modelled flow data and the observed flow data was only slightly worse (0.733) than that found for the Glenside data (0.757). The total observed flow during the data period was 1,580,000 and the total modelled flow (using the modified Kent Town rainfall data) was 1,509,000, which was much closer than the total flow found using the Glenside data. Rainfall and flow data were plotted for the same storms shown earlier. The results shown in Figures (5-17), (5-18) and (5-19) were very similar to the corresponding plots derived using the Glenside data (Figures 5-12, 5-13 and 5-14). However, the fit for the flow data in Figure (5-16) was not as close as the corresponding Figure (5-11). This was due to the considerable difference between the Kent Town and Glenside rainfall data for that particular storm. This may reflect the movement of a storm cell over the area.

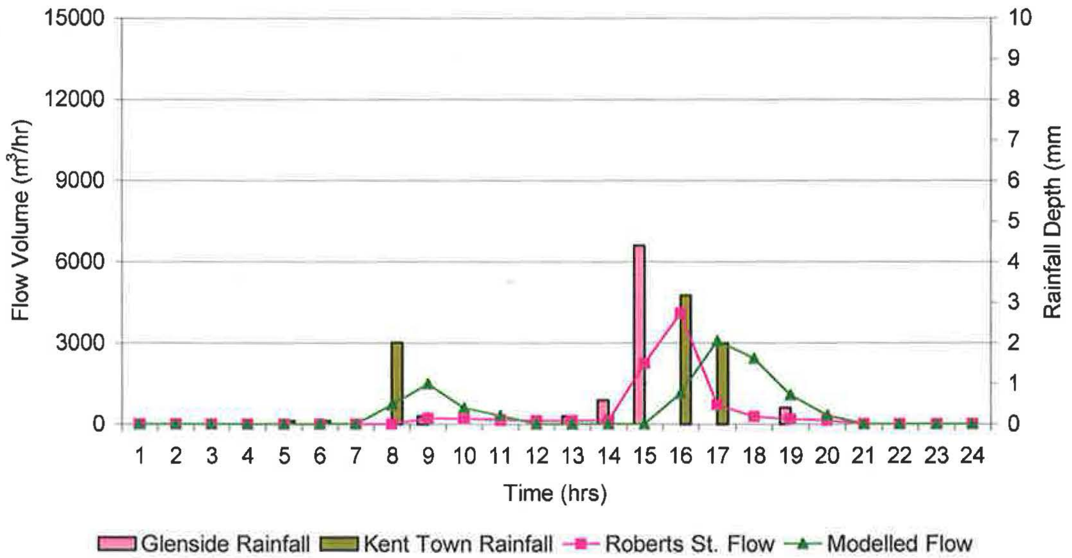


**Figure 5-16 Modelled Versus Observed Flow for Storm on 14/1/97 (Using Adjusted Kent Town Data as Input into Flow Model)**

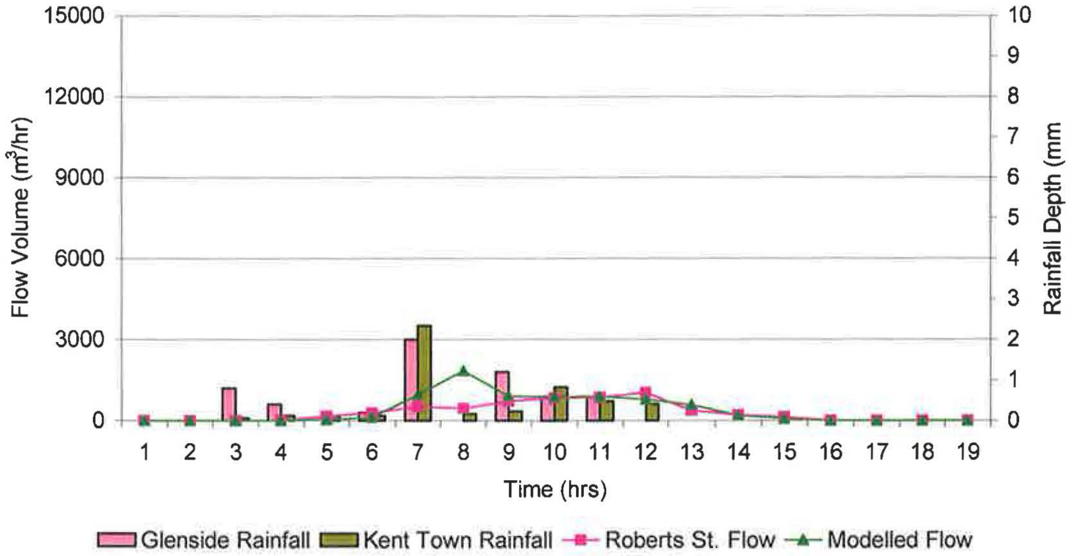


**Figure 5-17 Modelled Versus Observed Flow for Storm on 4/4/97 (Using Adjusted Kent Town Data as Input into Flow Model)**

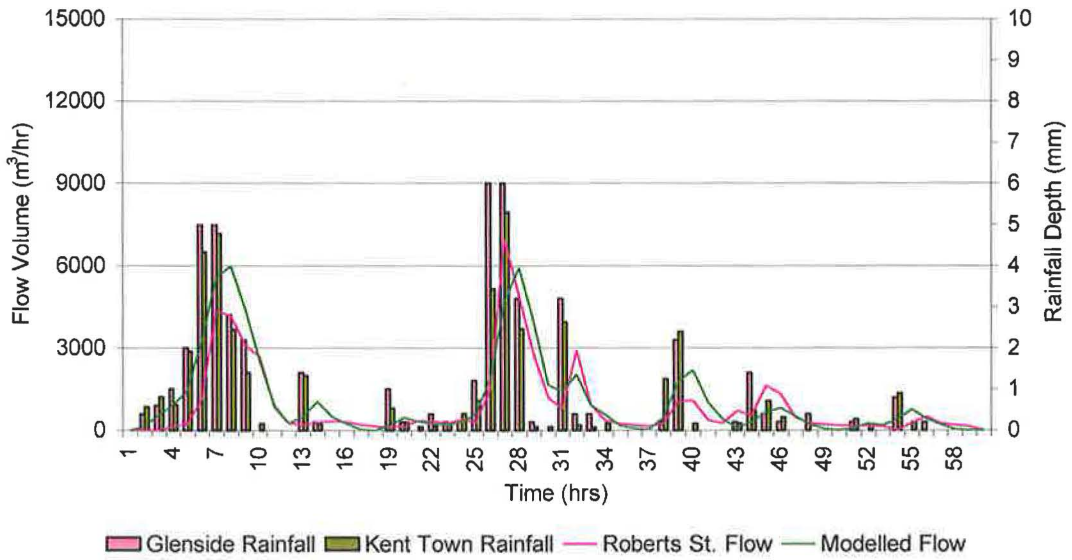


**Figure 5-18 Modelled Versus Observed Flow for Storm on 6/8/97-8/8/97 (Using Adjusted Kent Town Data as Input into Flow Model)**

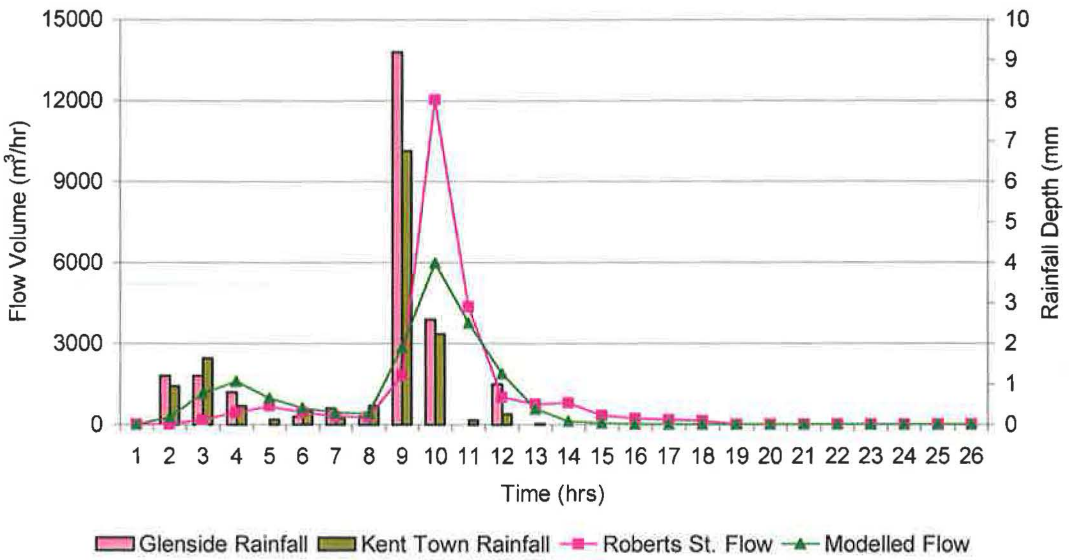


**Figure 5-19 Modelled Versus Observed Flow for Storm on 6/12/96 (Using Adjusted Kent Town Data as Input into Flow Model)**

## 5.8 Water Quality Modelling

### 5.8.1 Water Quality Monitoring

There are a number of composite sampling stations, owned by the Patawalonga Catchment Water Management Board (PCWMB), located within the Patawalonga catchment, however none are located on Parklands Creek (see Figure 5-3). Flow weighted average concentrations for a range of pollutants were calculated for a number of significant storm events. The closest stations to the Parklands Creek catchment were located on Brownhill Creek near Adelaide airport. Station AW504575 located just downstream of the confluence with Keswick Creek was operational from 19/3/96 to 4/3/97 until superseded by station AW504583, which has data from 25/3/97 onwards, and is located approximately 1 km further downstream. Concentrations at station AW504575 were approximately twice the concentrations at AW504583 for many pollutants (see Table 5-2). A drain flowing into Brownhill Creek at Morphett Road, between the two sampling stations, could account for the large drop in concentrations.

**Table 5-2 Water Quality at Brownhill Creek, Near Adelaide Airport**

| | AW504575 | | | | AW504583 | | | |
|---|---|---|---|---|---|---|---|---|
| **Water quality parameter** | **Min** | **Max** | **Average** | **Median** | **Min** | **Max** | **Average** | **Median** |
| TDS (mg/L) | 73 | 1700 | 251.5 | 160 | 52 | 1400 | 308.1 | 220 |
| Turbidity (NTU) | 1.9 | 240 | 61.3 | 54.5 | 3 | 160 | 35.1 | 19 |
| Suspended solids (mg/L) | 6 | 288 | 99.0 | 90 | 2 | 378 | 43.9 | 24 |
| **Inorganic (mg/L)** | | | | | | | | |
| Cadmium | 0.0002 | 0.0011 | 0.0003 | 0.0002 | 0.0002 | 0.0011 | 0.0004 | 0.0003 |
| Chromium | 0.005 | 0.034 | 0.0082 | 0.006 | 0.005 | 0.018 | 0.007 | 0.005 |
| Copper | 0.005 | 0.050 | 0.023 | 0.024 | 0.005 | 0.102 | 0.015 | 0.012 |
| Lead | 0.001 | 0.198 | 0.069 | 0.066 | 0.001 | 0.263 | 0.029 | 0.0215 |
| Zinc | 0.064 | 0.621 | 0.307 | 0.333 | 0.045 | 1.11 | 0.183 | 0.17 |
| **Organic (mg/L)** | | | | | | | | |
| Nitrite | 0.005 | 0.53 | 0.164 | 0.163 | 0.009 | 0.791 | 0.164 | 0.164 |
| Nitrogen total | 0.05 | 5.23 | 1.66 | 1.44 | 0.48 | 3.7 | 1.06 | 0.885 |
| Phosphorus | 0.005 | 0.8 | 0.263 | 0.243 | 0.06 | 0.7 | 0.170 | 0.133 |
| Sulphide | 13.1 | 31.2 | 19.7 | 14.9 | 9 | 48.2 | 18.9 | 12.7 |

The PCWMB, with assistance from the Australian Water Quality Centre has conducted an ambient water quality monitoring program in the Patawalonga catchment (Schultz and Thomas, 1999). Nine sites were sampled, including one located in Parklands Creek on the wetland site. Grab samples were taken only when water was flowing or when significant pools existed. The primary purpose of the sampling program was to assess the number and types of macroinvertebrates present in the catchment's creeks, although testing was also done for some metals, total dissolved solids (TDS), suspended solids and nutrients. Very low numbers of macroinvertebrates were found at the Parklands Creek site, possibly because the channels flowing into the creek were concrete lined for most of their lengths (Schultz and Thomas, 1999). Median pollutant concentrations were low for all water quality parameters, although there was great variability between samples.

Waterwatch have conducted sampling at four locations in the South Parklands. All of these sites were close to the proposed wetland site, one was located on Parklands Creek (close to the PCWMB ambient monitoring site) and the other three on drains coming from the CBD. Grab samples were taken six times a year by students of nearby Pulteney Grammar School. These samples were taken on set dates spread throughout the year. If there was no water flowing in the drains then samples were taken from pools. Results varied considerably between samples, generally nutrient levels were high for all sites. The quality of water samples taken from the Parklands Creek site was similar to samples from the other sites. The only water quality parameters that showed significance difference were salinity, which was much lower, and nitrates, which was three to five times higher.

The Department of Environment and Natural Resources, with the assistance of the National Landcare Programme, has conducted a survey of silts in the subdrains of the Patawalonga catchment. The results of this survey were presented by pH environment (1995). The aims of the study were to ascertain the most polluted fraction of the sediment, to identify the major sources of pollution, to examine the relationship between land use and pollutants and to provide a basis for improving the design and location of silt traps. Six sites were sampled in the Keswick Creek catchment, however only one was located on Parklands Creek. One of the sampling sites was located on the south side of the airport after the junction with Brownhill Creek, very close to the PCWMB composite sampling station. The study found that the majority of pollutants were associated with grain sizes of less than 75 μm. The pollutant loads in sediments taken from the Parklands site were generally low in comparison to loads from other sites. Heavy metal concentrations at the airport sampling station were 60% to 100% higher than concentrations in Parklands Creek and nutrient and organic material levels were between 20% to 30% higher.

### 5.8.2 Long Term Simulation

The results from the water quality monitoring outlined above are summarised in Table (5-3). The table illustrates the great variability in water quality between the sampling sites. Also contained in the table are the water quality guidelines for primary and secondary contact

Section 5.8.2, page 90, paragraph 1, sentence 5, change:

The different sampling techniques used at the three sites could account for this, however there does appear to be a substantial decrease in water quality between Parklands Creek and Brownhill Creek (near the airport) as indicated by results from a sediment study (pH environment, 1995).

To:

A decrease in water quality between Parklands Creek and Brownhill Creek was also found in the sediment study conducted by the Department of Environment and Natural Resources (pH environment, 1995).

**Table 5-3 Water Quality Data for Parklands Creek (Median Values Shown)**

| Water quality parameter | Brownhill Ck AW504575 | Parklands Ck. PCWMB | Parklands Ck. Waterwatch | Water quality guideline A | Water quality guideline B |
|---|---|---|---|---|---|
| Faecal coliforms / 100 ml | - | - | - | <10 | <100 |
| BOD (mg/L) | - | - | - | <20 | <20 |
| PH | - | - | 7 | 5 - 9 | 6.5 – 9 |
| TDS (mg/L) | 160 | 290 | 100 | <500 | <1000 |
| Turbidity (NTU) | 54.5 | 7.2 | 17.5 | <2 | <10% change |
| Suspended solids (mg/L) | 90 | 13 | - | - | <30 |
| **Metals (mg/L)** | | | | | |
| Aluminium | - | - | - | - | 0.1 |
| Arsenic | - | - | - | 0.05 | 0.05 |
| Cadmium | 0.0002 | - | - | 0.005 | 0.005 |
| Chromium | 0.006 | - | - | 0.05 | 0.01 |
| Copper | 0.024 | 0.006 | - | 2 | 0.005 |
| Cyanide | - | - | - | 0.1 | 0.005 |
| Lead | 0.066 | 0.012 | - | 0.05 | 0.005 |
| Mercury | - | - | - | 0.001 | 0.0001 |
| Nickel | - | - | - | 0.1 | 0.15 |
| Selenium | - | - | - | 0.05 | 0.005 |
| Silver | - | - | - | 0.05 | 0.001 |
| Zinc | 0.333 | 0.257 | - | 3 | 0.05 |
| Ammonia | - | - | - | 0.5 | 0.03 |
| **Non-Metals (mg/L)** | | | | | |
| Nitrate | - | - | 0.245 | 10 | 0.1 |
| Nitrite | 0.163 | - | - | 1 | - |
| Nitrogen total | 1.435 | 0.72 | - | - | 0.01 |
| Phosphorus | 0.243 | 0.091 | 0.19 | - | 0.015 |

* Water quality guideline A – primary contact recreation guideline (ANZECC, 1992), (EPA, 1998)
** Water quality guideline B – secondary contact recreation and protection of aquatic ecosystems guidelines (ANZECC, 1992), (EPA, 1998)

The water quality data was compared to the maximum pollutant concentration requirements for guidelines adopted in this simulation. Pollutant concentrations found in samples from the three sampling sites compared favourably with guideline A, the guideline used for aquifer recharge. The only water quality parameters that exceeded the recommended maximum concentrations were lead and turbidity. The median lead concentration at the Brownhill Creek site only just exceeded the recommended levels, but the lead concentration in Parklands Creek was well below. Turbidity levels were substantially higher at all three sites. To meet the recommended level a 95% reduction was required when using the Brownhill Creek data, 90% reduction was required based on Waterwatch Parklands Creek data and 75% based on the PCWMB Parklands Creek data. Using the pollutant removal curve for suspended solids in Figure (2-1) a wetland retention time of between 10 and 20 days would achieve the desired concentration. A retention time of 10 days was used as the aquifer injection water quality requirement in the long-term simulation model.

Pollutant concentrations in samples from the three sites exceeded most recommended maximum concentrations for guideline B, the guideline used for downstream discharge. Reductions in concentrations of between 80 and 95% for heavy metals, 99% for nitrogen and 95% for phosphorus were required to reduce pollutant concentrations at the Brownhill Creek site below the guideline values. The corresponding values for Parklands Creek were, 15 and 80% for heavy metals, 99% for nitrogen and 85% for phosphorus. The water quality concentrations for guideline B are not achievable based on the pollutant removal curves for phosphorus and nitrogen in Figures (2-2) and (2-3). The minimum detention time for basin weir outflow was varied between five and fifty days to enable an examination of the functional trade-offs for the wetland system.

### 5.8.3 Costs

5.8.3 a) Construction Costs

5.8.3 a) i. Basin Excavation and Preparation

All wetland design reports for the South Parklands wetland simply group the costs associated with excavation and formation into one figure. This made it difficult to make an accurate estimate of the costs involved. To simplify cost calculations, basin excavation and formation costs were grouped into two categories; excavation and removal of soil off-site, and reuse of excavated soil on-site. Both of these values were calculated in $/m$^3$ of soil excavated. Reuse of excavated soil included construction and compaction of both the levee banks surrounding the wetland and the basin's clay liner.

B.C. Tonkin and Associates conducted a feasibility study into the construction of a stormwater wetland system for Morphettville Racecourse in Adelaide (BC Tonkin, 1998). They estimated the cost of excavating soil and depositing in spoil heaps on-site at \$5/m$^3$, the construction of the clay lining using heaped clay at \$6/m$^3$, and the removal of excess spoil off-site at \$10/m$^3$.

Begg (1998) estimated the cost of removing excess spoil offsite for the South Parklands wetland at \$15/m$^3$, accounting for approximately one-third of the total cost of the wetland. AACM consultants carried out an economic costs and benefits analysis for the South Park Land wetland design presented in Begg (1998) (AACM, 1999). They predicted that excess spoil could be sold, reducing the cost to $2.50/m$^3$. This estimate was based on experience with the construction of the Warriparinga wetland, in Adelaide's south.

Rawlinson's construction handbook provides cost estimates for soil excavation and formation for building construction projects (Rawlinson's, 1997). Their estimates are based on soil type. They estimate the cost for excavating in bulk to reduce levels and deposit, spread and level within 1 km including compaction to 90% in clay soil at $9.75/m$^3$. The similar cost for excavating and removing clay soil off-site was estimated at $4.90/m$^3$ plus an additional $0.34/m$^3$ per extra 1 km of cartage.

The costs discussed in this section are presented in Table (5-4). Excavation costs are not included in the costs given in B.C. Tonkin (1998), Begg(1998) and AACM (1999) for removal of excess spoil off-site. To cover the cost of excavation $5/m$^3$ was added to each of the costs. $5/m$^3$ was consistent with the excavation cost estimated in B.C. Tonkin (1998) and Rawlinson's (1997). There was considerable difference between the costs for the excavation and removal of soil off-site. $15/m$^3$ was used in the optimisation procedure because $20/m$^3$ was considered to be conservative (AACM, 1999) and the low cost of $7.50/m$^3$ relies on there being a buyer for the excess soil. The costs for excavation and reuse on-site were much closer, $11/m$^3$ was selected because this estimate was made specifically for wetland construction, whereas the Rawlinsons (1997) estimate is a general figure for all civil engineering projects.

**Table 5-4 Basin Excavation and Formation Cost Summary**

| | **B.C. Tonkin (1998)** | **Begg (1998)** | **AACM (1999)** | **Rawlinsons (1997)** |
|---|---|---|---|---|
| **Excavate and reuse on site** | $11/m$^3$ | - | - | $9.75/m$^3$ |
| **Excavate and remove offsite** | $15/m$^3$ | $20/m$^3$ | $7.50/m$^3$ | $4.90/m$^3$ plus $0.34/m$^3$ per km of cartage |

5.8.3 a) ii. Basin Outlet Structure

Excavation and scour protection costs associated with construction of the outflow weir were not included in the optimisation procedure. These costs are relatively constant and changes in the height and length of the weir would have negligible impact on them. Concrete and formwork costs however could differ considerably depending on the weir's dimensions.

The outflow weir comprised a 3000 mm high wall with a trapezoidal section cut out from the top and a strip footing base 3000 mm wide and protruding 500 mm past either side of the wall (see Figures 5-20 and 5-21). The depth of the wall and strip footing were 250 mm. The height of the trapezoidal section from the bottom of the wall (h) and the length of the trapezoidal section (l) were decision variables in the optimisation procedure. The costs per $m^3$ of reinforced concrete were estimated at \$181 and \$151, for the wall and strip footing respectively. Grade 3 formwork (\$97.5/$m^2$) was used for the exposed sections of the wall and grade 5 (\$78/$m^2$) elsewhere (Rawlinson's, 1997).

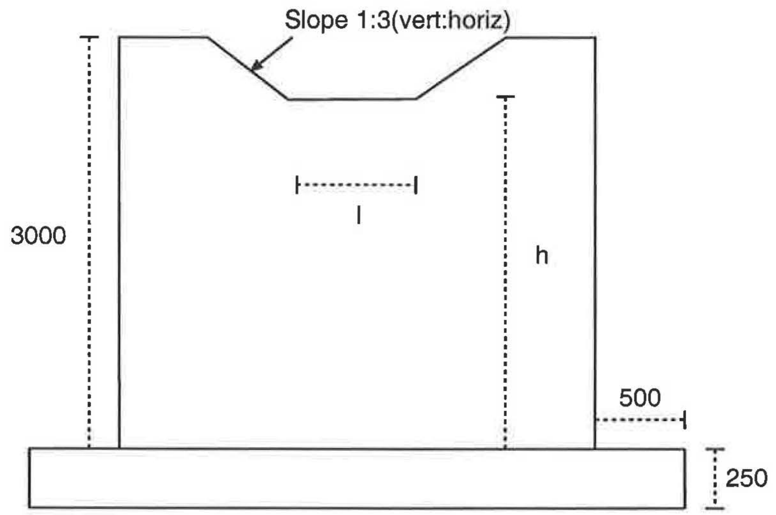


**Figure 5-20 Front View of Trapezoidal Weir (Dimensions in mm)**

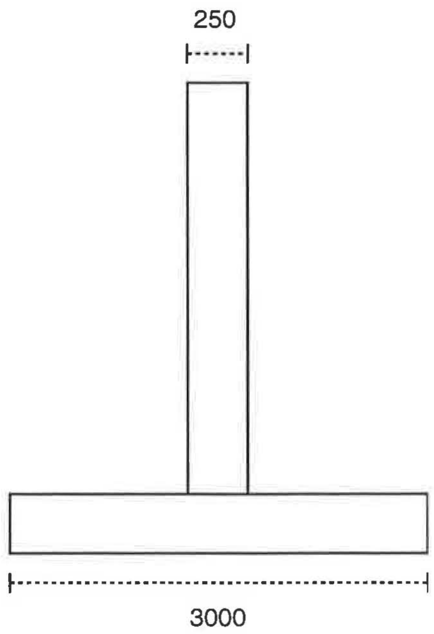


**Figure 5-21 Side View of Trapezoidal Weir (Dimensions in mm)**

5.8.3 b) Irrigation Water Benefits

Adelaide mains water is provided at a cost of \$0.90 per kL. Although at present the City of Adelaide is not charged for the water, the situation may change in the future. In the optimisation procedure, it is assumed that the City is charged for mains water. The volume of water extracted from thc aquifer for irrigation of the surrounding Parklands represents a cost saving. The annual irrigation water benefit ($B_{iw}$) was calculated by

$$B_{iw} = E \times 900 \tag{5-5}$$

Where E = average annual volume of water extracted from aquifer (ML) (averaged over the 10 year simulation period)

5.8.3 c) Operation and Maintenance Costs

Maintenance costs that varied with respect to the decision variables were pond maintenance, aquifer recharge pump operation, aquifer extraction pump operation and pump maintenance costs. These costs were taken from estimates presented in the economic analysis conducted by AACM (1999). A wetland pond maintenance cost of \$1300/yr/ha was used in the simulation ($C_{wm}$). This value was based on experience at the City of Salisbury Council. The hourly aquifer storage and aquifer recovery pump operation costs were calculated using Equation (5-6)

$$P = \frac{\rho g Q h}{\eta} \tag{5-6}$$

Where
P = power (W)
ρ = fluid density (kg/m$^3$)
g = acceleration due to gravity (m/s$^2$)
Q = flow rate (m$^3$/s)
h = head provided by the pump (m)
η = pump efficiency

The head provided by the pump was estimated at 10 m for injection and 30 m for extraction. Pump efficiency was estimated at 75%. The hourly operation costs for injection and extraction, derived from Equation (5-7), were \$0.10 and \$0.15 respectively, using an electricity cost of 14.5$^c$/kWH. Annual pump operation costs ($C_{po}$) were calculated using Equation (5-7)

$$C_{po} = \frac{I}{Q_s \times 3600} \times 0.10 + \frac{E}{Q_r \times 3600} \times 0.15 \tag{5-7}$$

Where
E = mean annual volume of water extracted from aquifer (L)
I = mean annual volume of water injected into aquifer (L)
$Q_s$ = storage pumping rate (L/s)
$Q_r$ = recovery pumping rate (L/s)

The pump maintenance cost used in the optimisation procedure was \$2000/pump/year ($C_{pm}$) (AACM, 1999).

The net present value for operating and maintenance costs and irrigation water benefits were calculated assuming the length of the project was 30 years and the discount rate was 5%.

### 5.8.3 d) Penalty Costs

Set penalty costs were applied to any chromosome that failed to meet any of the five wetland functional requirements detailed in Section 5.5. A set penalty cost of \$10,000,000 was used for each requirement, effectively preventing any offending solutions from taking further part in the GA procedure.

A proportional penalty cost was applied to the height difference between the weir height and the ASR pump off-take height

-

Section 5.8.3 d), page 97, equation 5-8, change:

$$P_{hd} = 100 \times (h_w - h_p)$$

Where $P_{hd}$ = weir, off-take differential penalty ($)

$h_w$ = weir height (mm)

hp = ASR off-take height (mm)

To:

$$P_{hd} = 10 \times (h_w - h_p)$$

Where $P_{hd}$ = weir, off-take differential penalty ($)

$h_w$ = weir height (cm)

hp = ASR off-take height (cm)

---

-

Section 5.8.3 d), page 97, paragraph 1, change:

This penalty encourages the ASR off-take height to be as close as possible to the weir, while still maximising the volume of water harvestable from the basin. This penalty was included into the objective function because pump off-takes are generally located as close to the outflow weir as possible. Water quality in a wetland is usually better in the upper portion of the water column.

To:

This penalty encourages the ASR off-take height to be as close as possible to the weir, where water quality is generally better, while still maximising the volume of water harvestable from the basin. Water quality in a wetland is usually better in the upper portion of the water column due to the fact that most stormwater pollutants are associated with sediments, that settle down through the water column over time.

---

and ASR off-take height used as decision variables in the GA.

The RRR model was used to derive the two flood hydrographs required in the flood simulation model. Simple water quality and rainfall runoff models were used in the long-term simulation due to a lack of data of sufficient quality.

Costing information for the optimisation runs was derived mostly from local wetland design experience.

# Chapter 6 Results

## 6.1 Introduction

Two wetland functions were studied in detail in this chapter, extreme flood mitigation and pollutant removal for downstream discharge. The design objectives corresponding to these functions, maximum peak outflow rate during the extreme flood event and the mean retention time, respectively, were altered. The objective to maximise the volume of water harvested was not altered. The GA optimisation framework described in Chapter 4 was used to determine the optimal wetland basin configuration for different combinations of peak outflow rate and mean retention time. The influence that changes in these design objectives had on the optimal wetland design is the focus of this chapter. Results from wetland simulations conducted for three different wetland configurations will also be discussed.

## 6.2 GA Optimisation Results

### 6.2.1 Problem Set-up

Maximum peak outflow rates of between 5 $m^3/s$ and 15 $m^3/s$, incremented by 1 $m^3/s$, were used in the GA optimisation runs. These applied for the 1 in 100 year flood, six-hour inflow hydrograph. The lower limit corresponds to the carrying capacity of the channel immediately downstream of the South Parklands wetland site, and the upper limit corresponds to a zero reduction in peak flow rate for the extreme flood event. The mean retention times used in the optimisation procedure ranged between 5 and 50 days, incremented by 5 days. These limits were selected as the minimum and maximum mean retention times for which urban wetlands could practicably be designed. A GA optimisation run was conducted for every combination of the two design objectives ($11 \times 10 = 110$ in total).

**Table 6-1 Decision Variable Ranges Used in the Optimisation Procedure**

| GA decision variable | Allowable range |
|---|---|
| Wetland volume ($m^3$) | 0 to 400,000 |

---

Section 6.2.1, page 100, paragraph 1, sentence 3, change:

A maximum weir length of 400 cm was considered a realistic value for a large wetland basin.

To:

A maximum weir length of 400 cm was considered reasonable based on the maximum 100 yr ARI inflow and the allowable range of weir heights.

---

Section 6.2.2, page 100, paragraph 1, change:

Figures (6-1), (6-2), (6-3), (6-4) and (6-5) show the improvement in the various components of fitness cost over the course of an optimisation run. Total cost (Figure 6-1) converges quickly at first and then declines slowly towards the near optimal solution. The shape of the construction cost (Figure 6-2) and operating and maintenance cost (Figure 6-3) convergence curves are very similar to the total cost curve. An explanation for this is that the decision variable that has the greatest impact on total fitness is wetland volume, it accounts for almost all of the construction cost (>95%) and the majority of the operating and maintenance cost (50 – 85%).

To:

Figures (6-1), (6-2), (6-3), (6-4) and (6-5) show the improvement in the various components of fitness cost over the course of an optimisation run. A maximum peak outflow rate of 12 $m^3/s$ for the 1 in 100 year flood event and a mean retention time of 40 days were used for this optimisation run. Total cost (Figure 6-1) converges quickly at first and then declines slowly towards the near optimal solution. The shape of the construction cost (Figure 6-2) and operating and maintenance cost (Figure 6-3) convergence curves are very similar to the total cost curve. An explanation for this is that the decision variable that has the greatest impact on total fitness is wetland volume, it accounts for almost all of the construction cost (>95%) and the majority of the operating and maintenance cost (50 – 85%).

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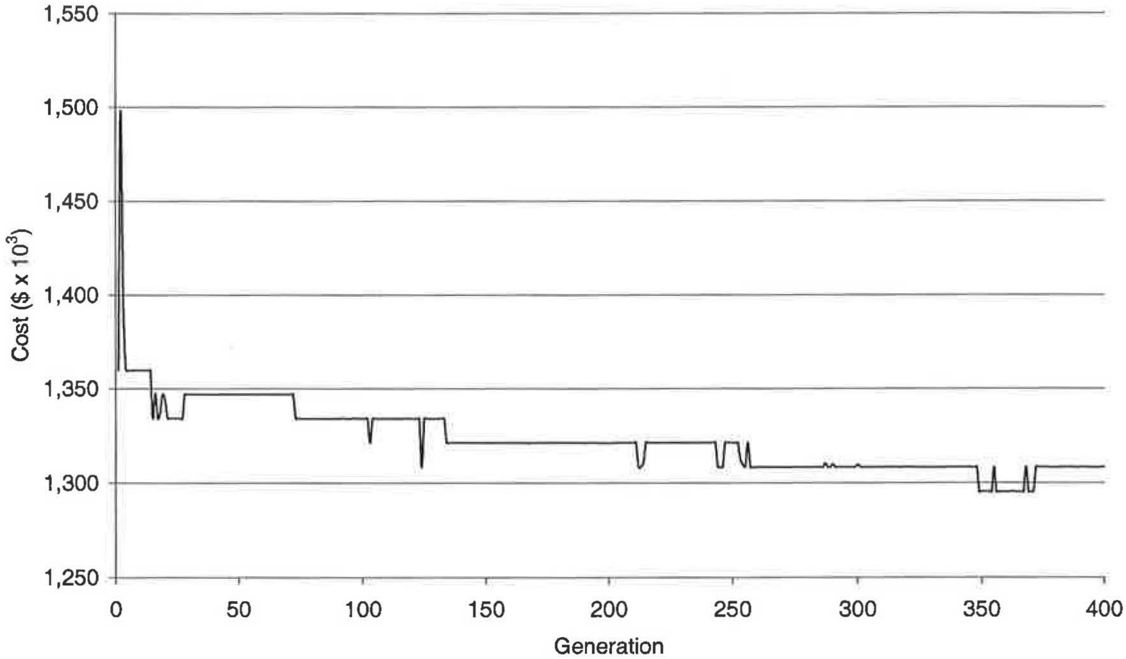



**Figure 6-1 Total Net Costs of the Fittest Solutions From Each Generation**

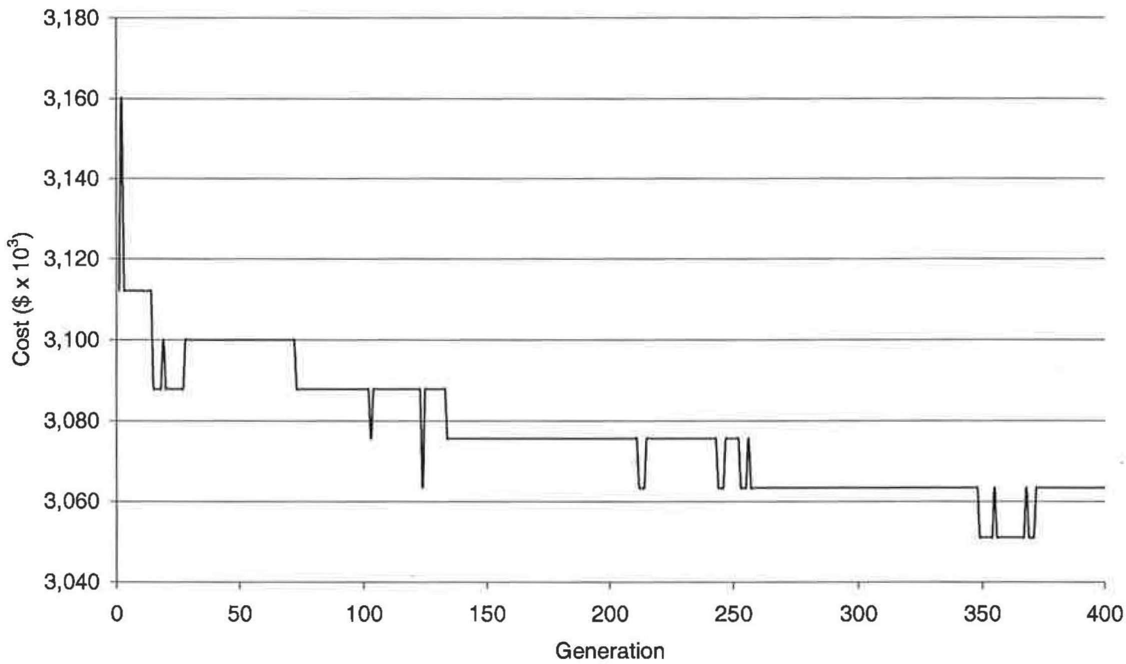
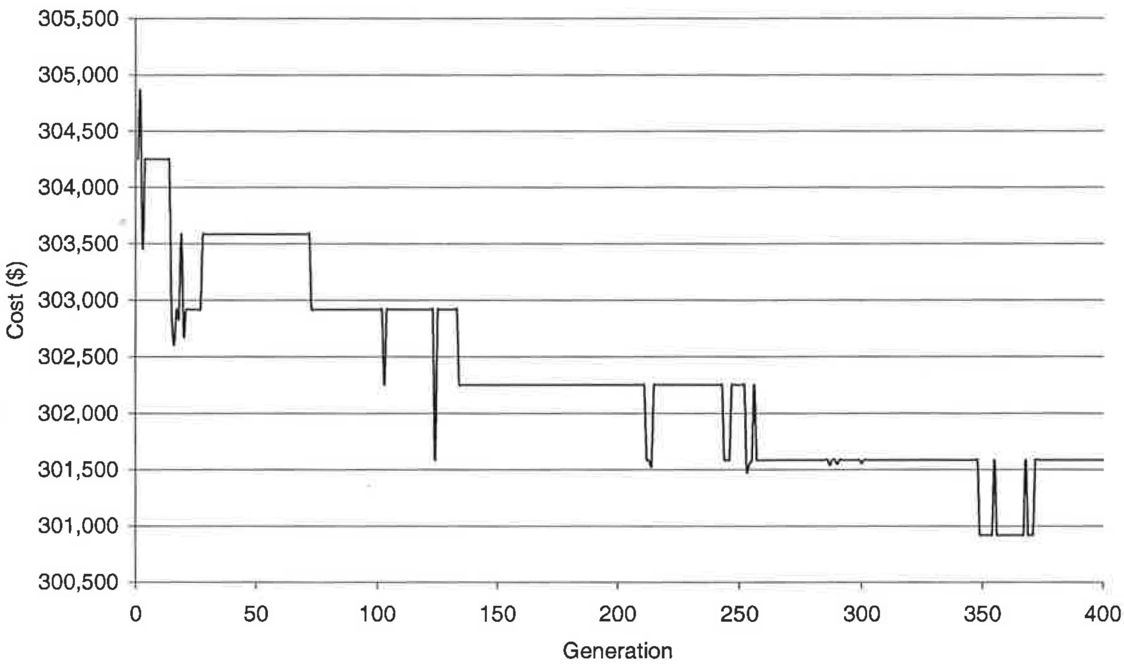



**Figure 6-2 Construction Costs of the Fittest Solutions From Each Generation**




Section 6.2.2, page 102, paragraph 1, change:

The water reuse benefit curve converges very quickly, much faster than the other cost components.

To:

The water reuse benefit curve converges very quickly, much faster than the other cost components. Demonstrating that the water re-use benefit has considerable leverage in

the optimisation process. It was the first component of the objective function to reach an optimal value.

**Figure 6-4 Present Value of Water Re-Use Benefits of the Fittest Solutions From Each Generation**

The penalty cost curve does not appear to converge at all; this could be because the penalty cost was small in comparison to the other cost components and hence had little effect on total net cost.

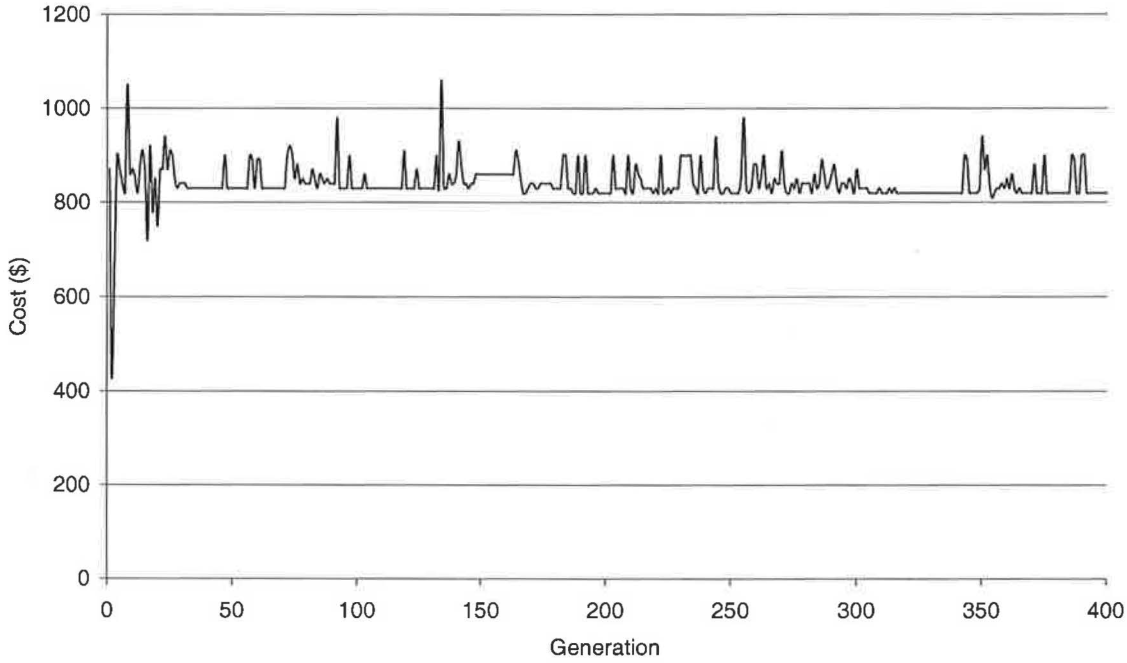



**Figure 6-5 Penalty Costs of the Fittest Solutions From Each Generation**

### 6.2.3 Results from Optimisation Runs

6.2.3 a) Optimal Decision Variable Plots

Results from the optimisation runs are shown in Figures (6-6) to (6-9). The optimal values for the four decision variables were graphed as contour plots with the mean retention time (MRT) and peak discharge rate (PDR) on the x and y-axes respectively. In this chapter, MRT refers to the mean retention time of water discharging over the outflow weir. The MRT is flow weighted and is equivalent to the $t_{50}$ statistic proposed in Walker (1996a).

Figure (6-6) shows how the optimal wetland volume changes with respect to the two design objectives. A distinct transition line can be seen on the graph, running diagonally from (PDR = 15 m$^3$/s, MRT = 10 days) to (PDR = 10 m$^3$/s & MRT = 50 days). Above this line, the optimal wetland volume was influenced by changes in the MRT. An increase in the value of the required MRT caused an increase in the optimal wetland volume. Below the line, the PDR had the greatest influence. Decreasing the required PDR lead to an increase in the optimal wetland volume.

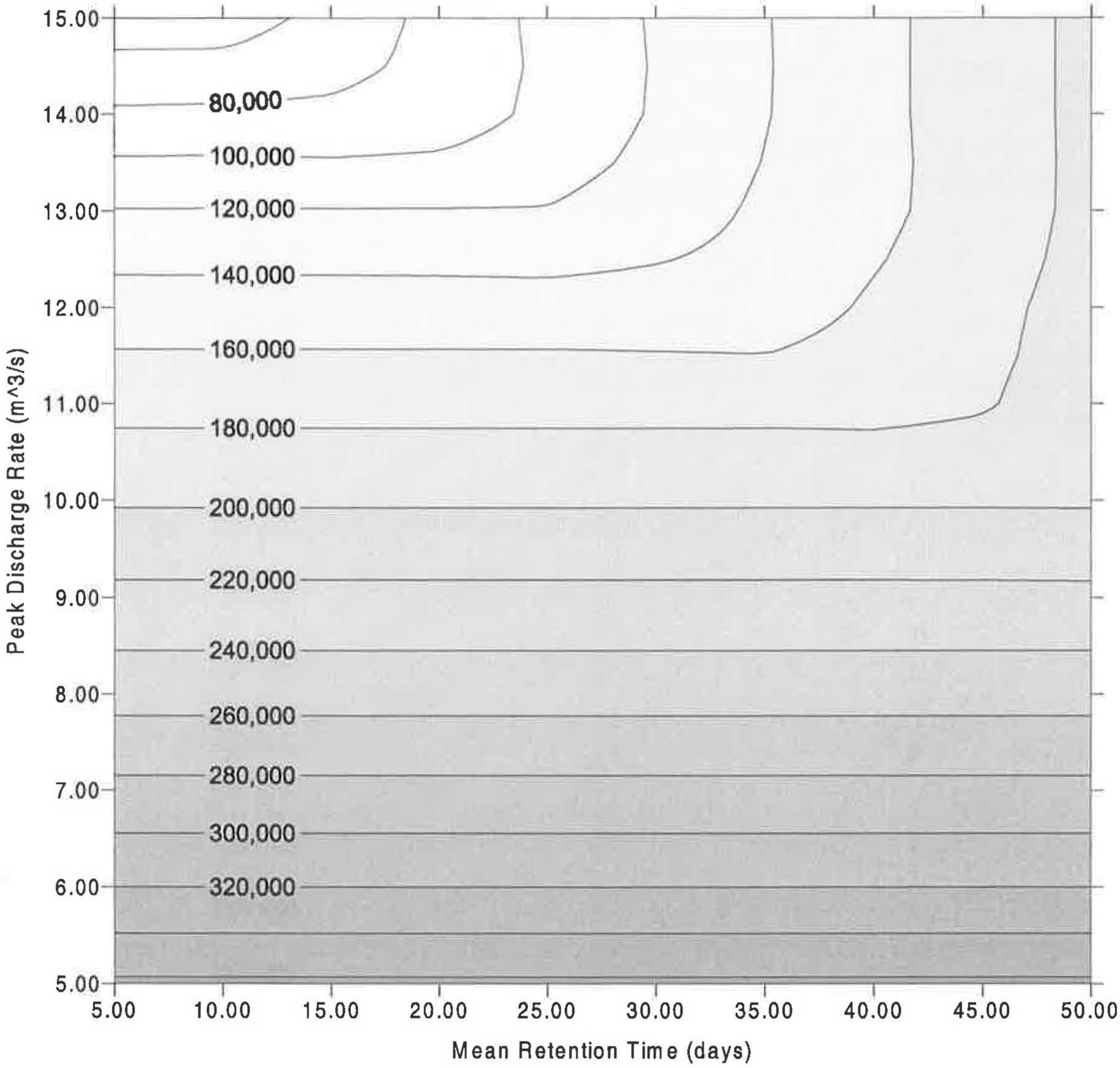



**Figure 6-6 Optimal Wetland Volumes ($m^3$)**

Figure (6-7) shows the graph for the optimal weir height. A transition line can also be seen, running in a straight line from (PDR=13, MRT=5) to (PDR=13, MRT=25) and then diagonally down to (PDR=10, MRT=50). The optimal weir height increases with increasing distance, in both directions from this line.

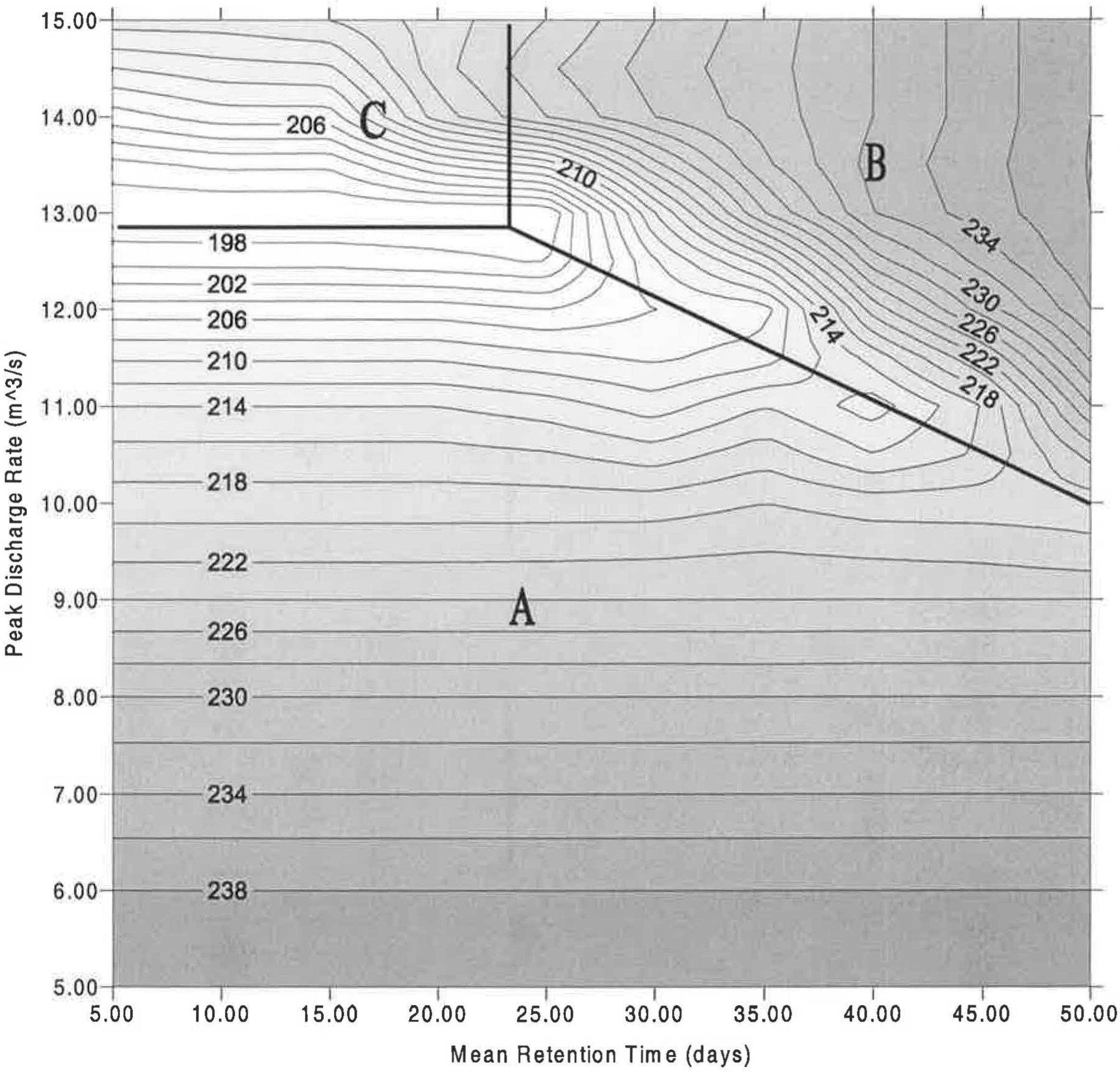



Figure 6-7 Optimal Weir Heights (cm)

-

Section 6.2.3, page 105, paragraph 1, change:

The transition line appearing in the optimal weir length graph (Figure 6-8) is the same as that for the optimal weir height graph. Points below the line have an optimal weir length of 0 cm (i.e. a vee notch weir). Above the line, the weir length increases away from the line, to the maximum length of 400 cm.

To:

The transition line appearing in the optimal weir length graph (Figure 6-8) is the same as that for the optimal weir height graph. Points below the line have an optimal weir length of 0 cm (i.e. a vee notch weir). The 0 cm contour could not be shown on the graph due to inadequacies with the graphing package used. Above the line, the weir length increases away from the line, to the maximum length of 400 cm.

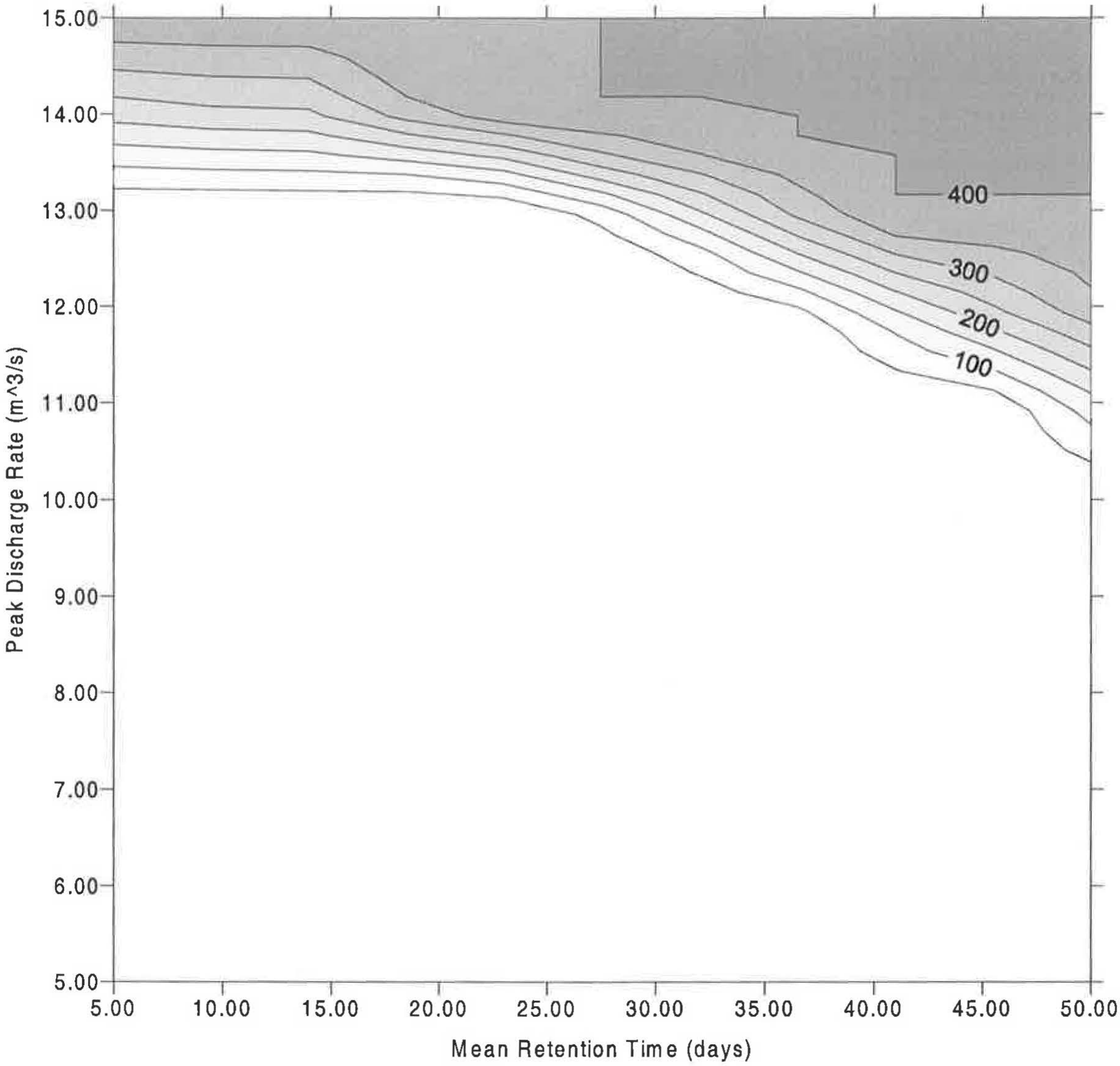



**Figure 6-8 Optimal Weir Lengths (cm)**

The shape of the optimal ASR off-take height graph (Figure 6-9) is very similar to the wetland volume graph. Optimal pump heights appear to correlate well with the required PDR below the line, increasing as the required PDR is lowered. Above the line, MRT appears to have the greatest influence on the optimal off-take height. The optimal off-take height increased as the required MRT was raised. Optimal ASR off-take height values in the white region (in the top left-hand corner of the graph) were 100 cm, the minimum height.

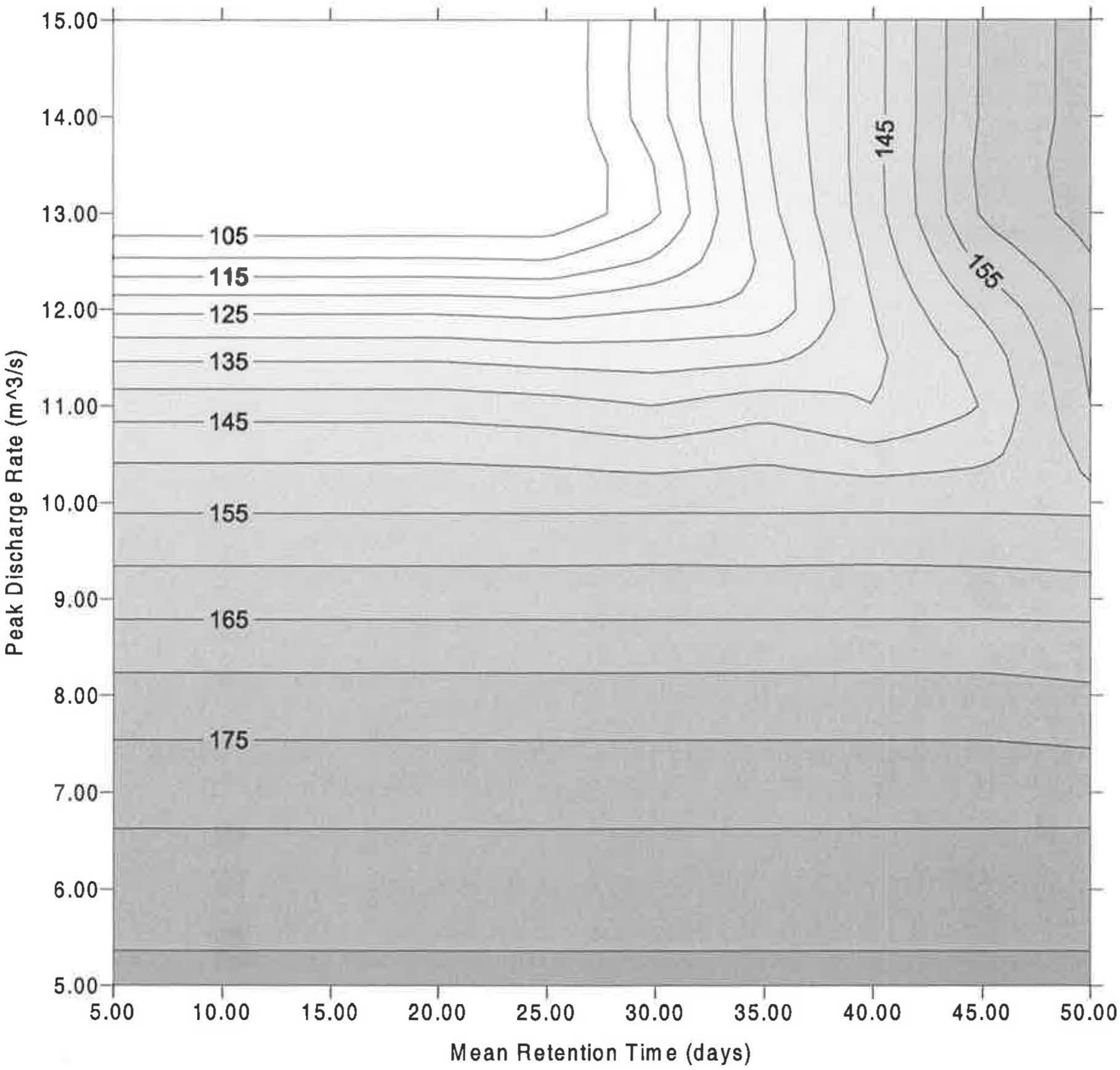



**Figure 6-9 Optimal ASR Off-Take Heights (cm)**

6.2.3 b) Cost Component Plots

Figures (6-10), (6-11) and (6-12) show how the total, construction and maintenance and operating costs of the optimal solution change with respect to the two design objectives.

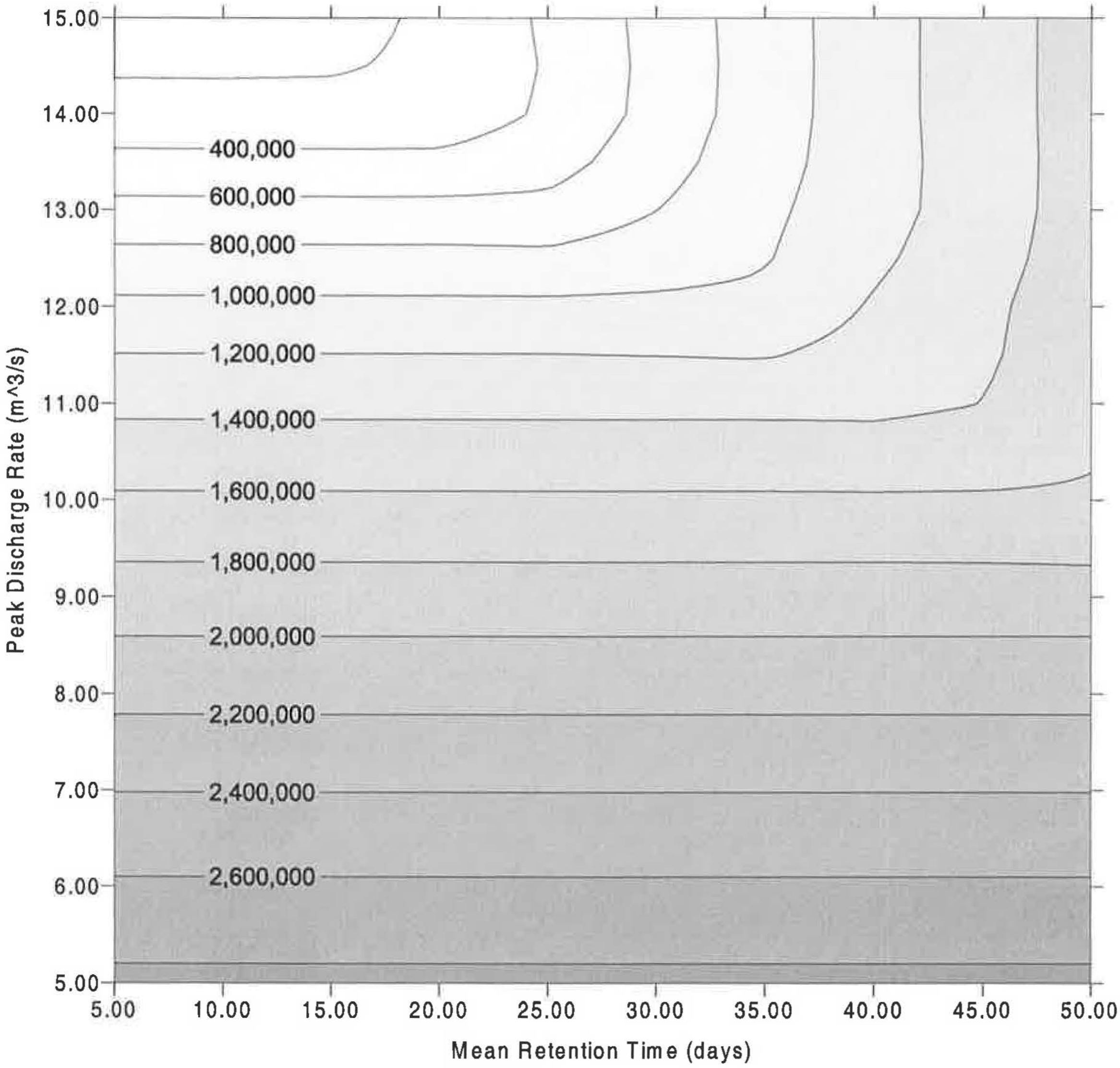



**Figure 6-10 Total Net Costs ($) for Optimal Solutions**

The shape of the total cost (Figure 6-10), construction cost (Figure 6-11) and the operating and maintenance cost plots resemble the optimal wetland volume plot (Figure 6-12). As explained above (see Section 6.2.2), costs associated with the wetland volume decision variable are the main components of construction cost and operating and maintenance cost, and hence the main components of total cost.

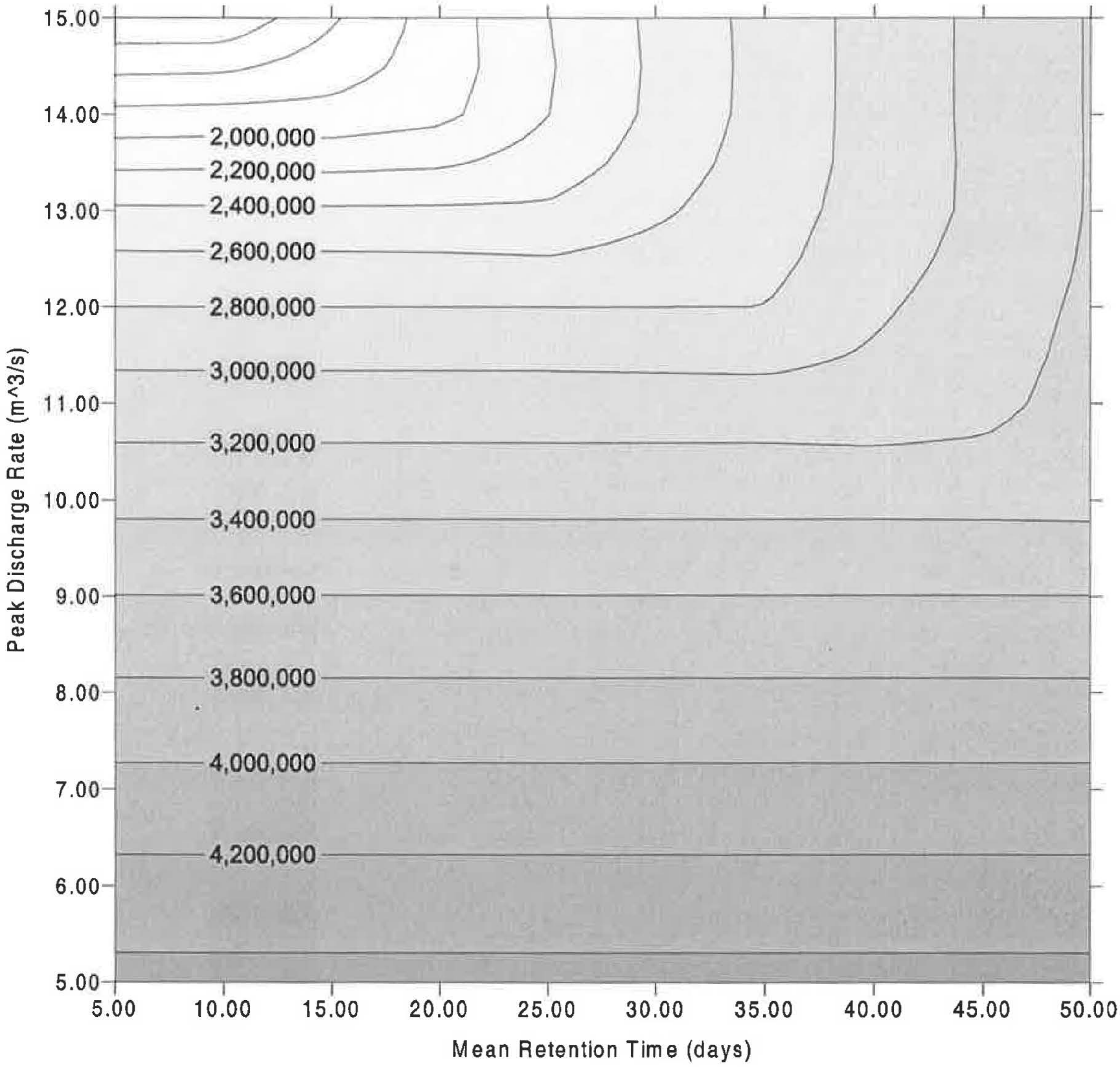



**Figure 6-11 Total Construction Costs ($) for Optimal Solutions**

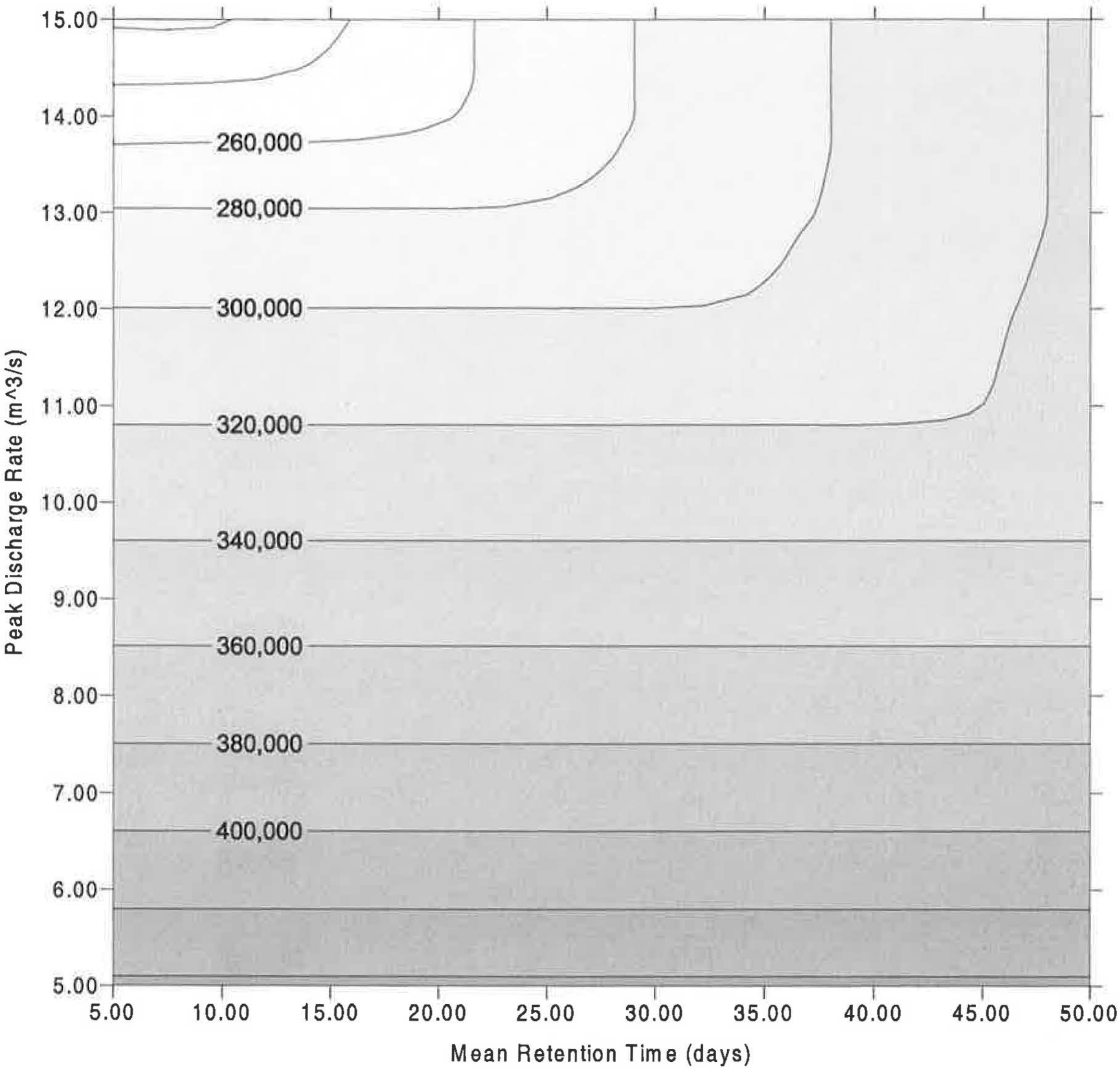



**Figure 6-12 Total Operating and Maintenance Costs ($) for Optimal Solutions**

The irrigation water benefit appears constant for most of the range of the two design objectives (Figure 6-13). A small region in the top left hand corner of the plot, where the benefit values decrease rapidly, is the only exception to this. The reason for the appearance of this region will be explained in the discussion that follows.

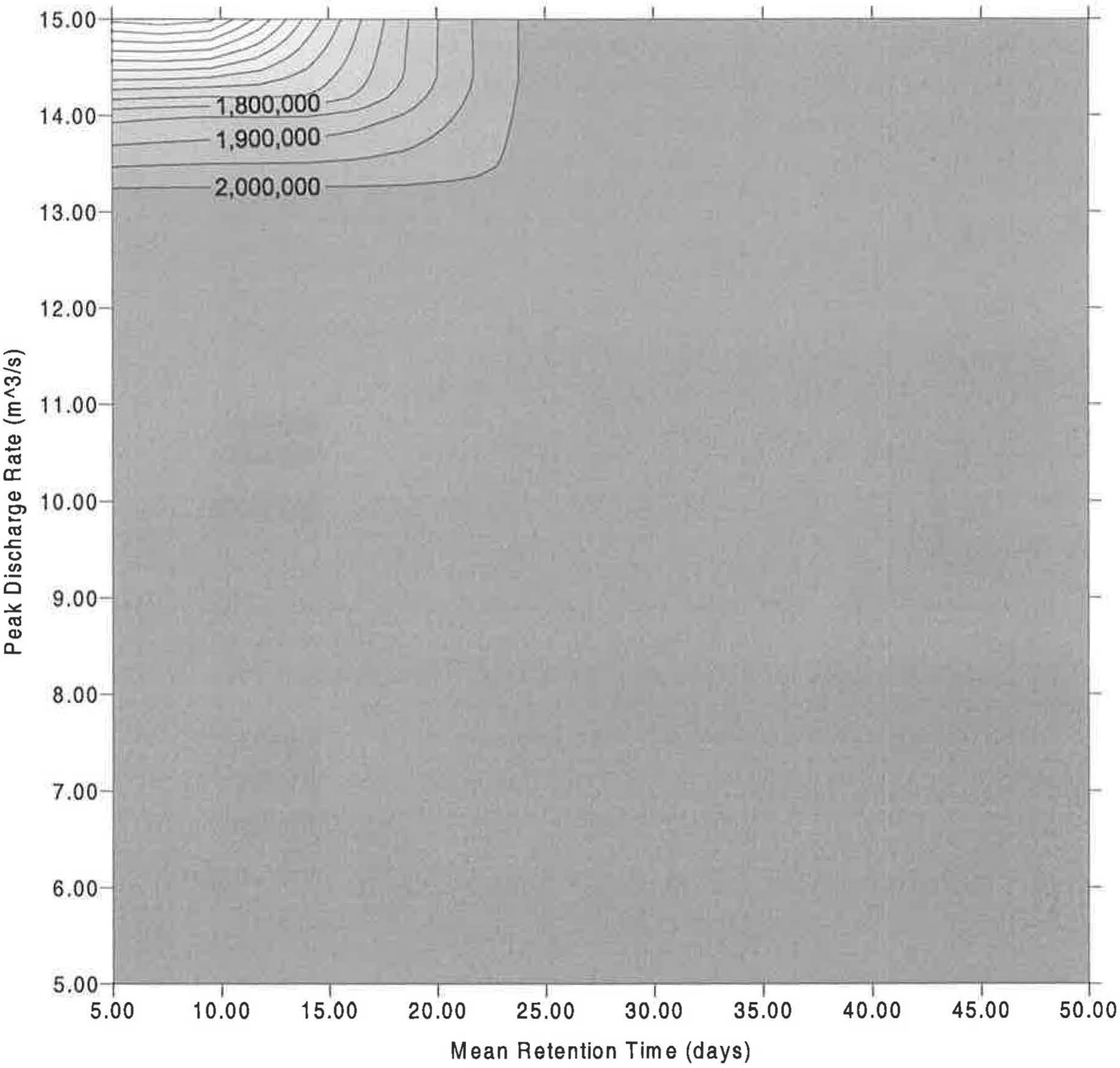



**Figure 6-13 Irrigation Water Benefits ($) for Optimal Solutions**

6.2.3 c) Discussion

Outflow from the wetland basin, over the trapezoidal weir, is governed by Equation (2-11). The only two non-constant variables in this equation are L, the length of the weir, and h, the head above the weir crest. The length of the weir is a decision variable in the optimisation procedure and therefore is constant for each individual wetland simulation. The head above the weir crest however, will change throughout each wetland simulation.

Three of the decision variables (wetland volume, weir height and weir length) play a part in determining the PDR during the extreme flood simulation. The effect that changing the weir length has on the PDR is obvious (see Equation 2-11); an increase in the weir length will lead to a larger PDR for a constant wetland volume and weir height. The other two decision variables, wetland volume and weir height affect the PDR by influencing the maximum head reached. The PDR over the trapezoidal weir, during an extreme flood simulation, occurs when the head above the weir crest is at its maximum. If weir height and weir length remain constant, increasing the wetland volume will decrease the maximum head reached, (and hence the PDR) due to the increased wetland surface area. Increasing the weir height, while keeping the other two decision variables constant also leads to a decrease in the maximum head reached. This effect occurs because the sides of the wetland basin are sloped; hence, the surface area increases with the water depth. Since the flood simulation commences with the water level at the weir crest, regardless of the weir height, and the extreme flood hydrograph used in all wetland simulations is the same, a lower maximum head will be reached if the weir crest is higher.

The MRT for a wetland configuration is influenced by two factors; the average discharge rate over the weir and the size of the permanent pond. The average discharge rate over the weir affects the MRT because it determines the time taken for the pond to drain down to weir level. A high average discharge rate will reduce the MRT because water leaving the wetland will have spent less time in the wetland. The average discharge rate is determined by the factors mentioned above for PDR. The size of the permanent pond influences the MRT because it determines the volume of water that remains resident in the wetland between flow events. A large permanent pond will increase the MRT because more water will be stored between flow events. The permanent pond size is determined by the wetland volume, weir height and the ASR off-take height. Increasing any one of these decision variables increases the permanent pond.

Three different zones called A, B and C can be identified on the plots of optimal values for the decision variables (see Figure 6-7). These zones are distinguished from each other by the functions that influence the optimal wetland configuration. In region A, the dominant function is flood mitigation. This region is characterised by horizontal lines in the optimal wetland volume, weir height and off-take height plots, and an optimal weir length of 0 cm. The dominant function in region B is pollutant removal for downstream discharge. This region is characterised by vertical lines in the optimal wetland volume and pump height graphs and increasing weir height and length away from the borders with the other two regions. In region

C, all three wetland functions influence the optimal wetland configuration. In this region, the ASR off-take height is at its minimum value of 100 cm.

Region A

As the PDR increases the wetland volume, weir height and ASR off-take height, all decrease and weir length remains constant at 0 cm over the entire region. The decrease in the ASR off-take height occurs indirectly, because of the drop in the weir height. The height difference between the off-take height and the weir height determines the volume of water that is available for aquifer recharge, known as the active storage volume. To maintain a certain total recharge volume the active storage volume must remain constant.

Region B

As the MRT increases, wetland volume, weir height and the off-take height all become larger, to increase the size of the permanent pond. Weir length also increases with the MRT, which is counter intuitive. As stated above a decrease in weir length leads to an increase in the MRT, by increasing the time taken for the pond to drain down to the weir level. There are two possible reasons why the reverse situation occurs. Firstly, the increase in MRT resulting from a decrease in the weir length may be small in comparison to the increase achieved through increasing the permanent pond volume. Secondly, the increase in the weir length may occur as a result of a more complex interaction with the weir height and wetland volume decision variables. As the weir height increases and the wetland volume decreases with the MRT, the storage volume above the weir crest will become smaller. In order to prevent water spilling over the basin during the 1 in 10 year design flood, the weir length must increase.

Region C

This region corresponds with the region of decreasing total injected volume or irrigation water benefit (see Figure 6-13). In this region, the contours of the optimal weir height and weir length plots are horizontal. As the PDR increases and with it the wetland volume, the weir height rises to maximise the active storage volume, because the off-take height can not be reduced below 100 cm. Weir length increases to prevent spillage from the basin during the design flood. The MRT begins to influence the optimal values for the decision variables in the right hand portion of this region.

## 6.3 Single Wetland Runs

Three different wetland configurations were selected, one from each region, to investigate their performance during the flood and long-term simulations.

### 6.3.1 a) Wetland Configuration 1 (Region A)

**Table 6-2 Configuration for Wetland 1**

| | |
|---|---|
| **Wetland volume** | 363,000 $m^3$ |
| **Weir height** | 239 cm |
| **Weir length** | 0 cm |
| **ASR off-take height** | 186 cm |
| **Total cost (excludes fitness penalty cost)** | $2,844,100 |
| **Construction cost** | $4,458,800 |
| **Operating and maintenance cost** | $442,800 |
| **Irrigation water benefit** | $2,057,500 |
| **Fitness penalty cost** | $530 |
| **Total volume injected** | 1,652,400 $m^3$ |
| **Outflow volume** | 5,074,400 $m^3$ |
| **Evaporation losses** | 1,764,400 $m^3$ |

This wetland configuration is an optimal solution when the PDR is 5 $m^3/s$ and the MRT is 50 days. Some results from the flood and long-term simulations conducted for this wetland configuration are shown in Figures (6-14), (6-15), (6-16) and (6-17).

Figure (6-14) shows the inflow and outflow hydrographs resulting from the 1 in 100 year flood simulation. There are three observations from the graph that are of particular interest. The PDR is reduced below the required value of 5 $m^3/s$. The time between the peaks of the inflow hydrograph and the outflow hydrograph was approximately 150 minutes. The long tail on the outflow hydrograph occurs because of the 0 cm weir length (i.e. it takes a long time for the basin to drain). A trapezoidal weir with a bottom length of 0 cm is effectively a v-notch weir.

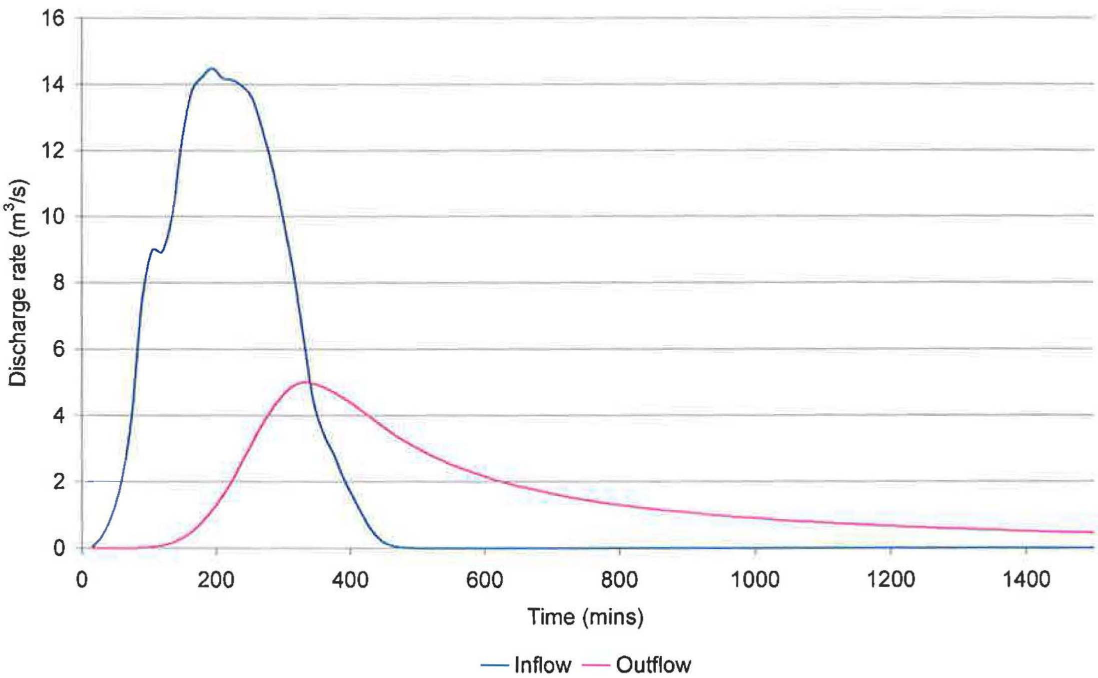



**Figure 6-14 1 in 100 Year Flood Hydrographs**

Figure (6-15) shows changes in the basin water storage volume over the duration of the long-term simulation. The purple and orange lines represent the volume in the basin below the weir crest and below the ASR off-take, respectively. The water level in the basin remains above the weir for much of the simulation period, due to the weir's small capacity. Several troughs can be seen in the graph, where the water level dips below the weir. These troughs occurred for about 4 months, each year, between 1989 and 1994. After this period the water level did fall below the weir but the drops were not as pronounced. The troughs generally occurred between January and May of each year. This period coincides with the time of year when evaporation rates are high and rainfall is low. During April and May water is harvested from the basin, which tends to keep the water level low. The smallest storage volume occurred during April of 1989, when the water height was at the ASR off-take height.

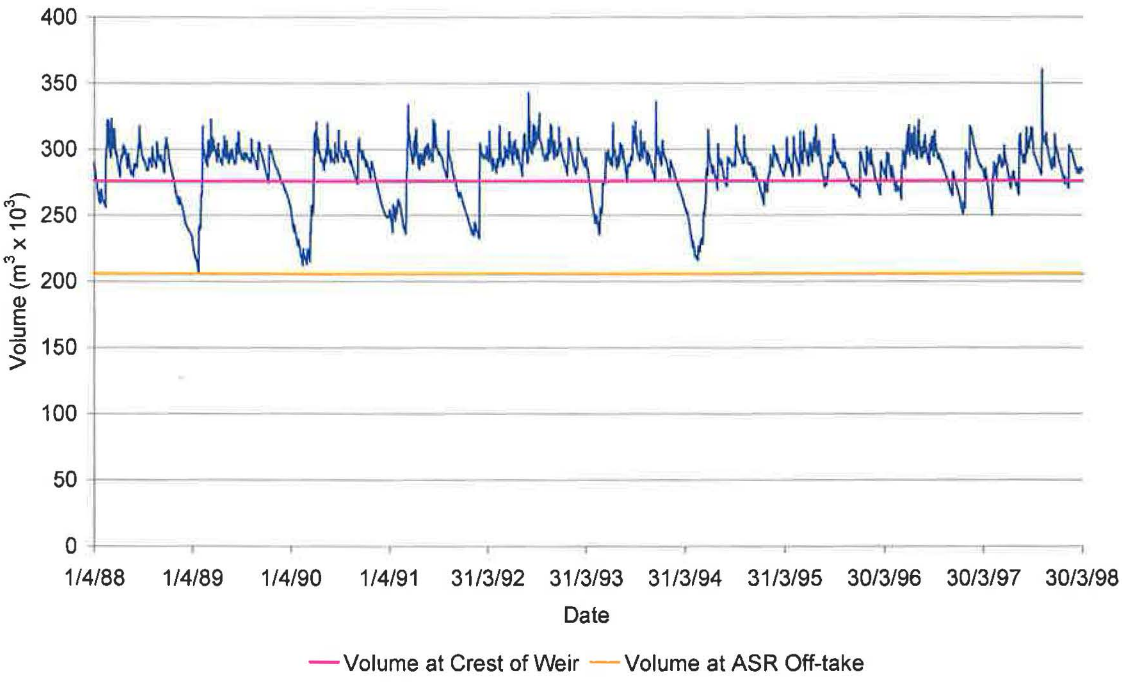



**Figure 6-15 Storage Volume Change for Wetland 1**

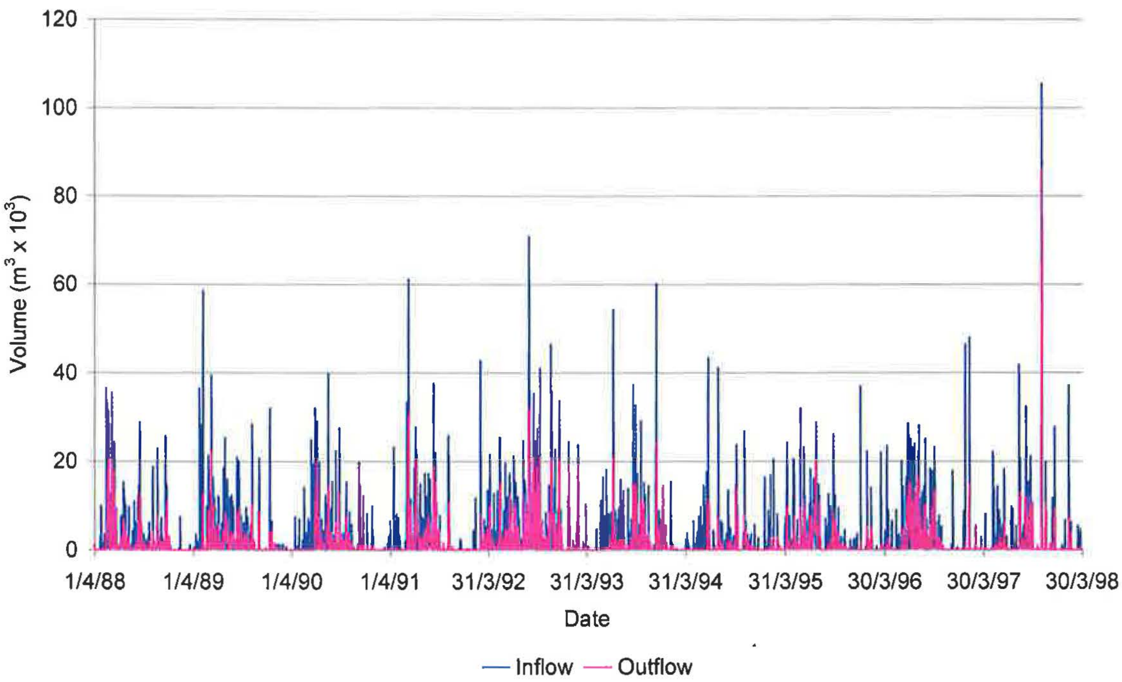



**Figure 6-16 Daily Inflow and Outflow Volumes for Wetland 1**

Figure (6-17) shows the water volumes injected into the aquifer during the simulation period. The peak of the injection volume columns represents the maximum possible daily injection volume, 1080 $m^3$. This volume is based on five ASR bores each operating at a constant injection rate of 2.5 L/s. The injection season (April to August of each year) is clearly visible in the graph. Days when injection did not occur are shown as thin clear lines in the blue injection columns. Injection occurred on almost every possible day, for this particular wetland configuration.

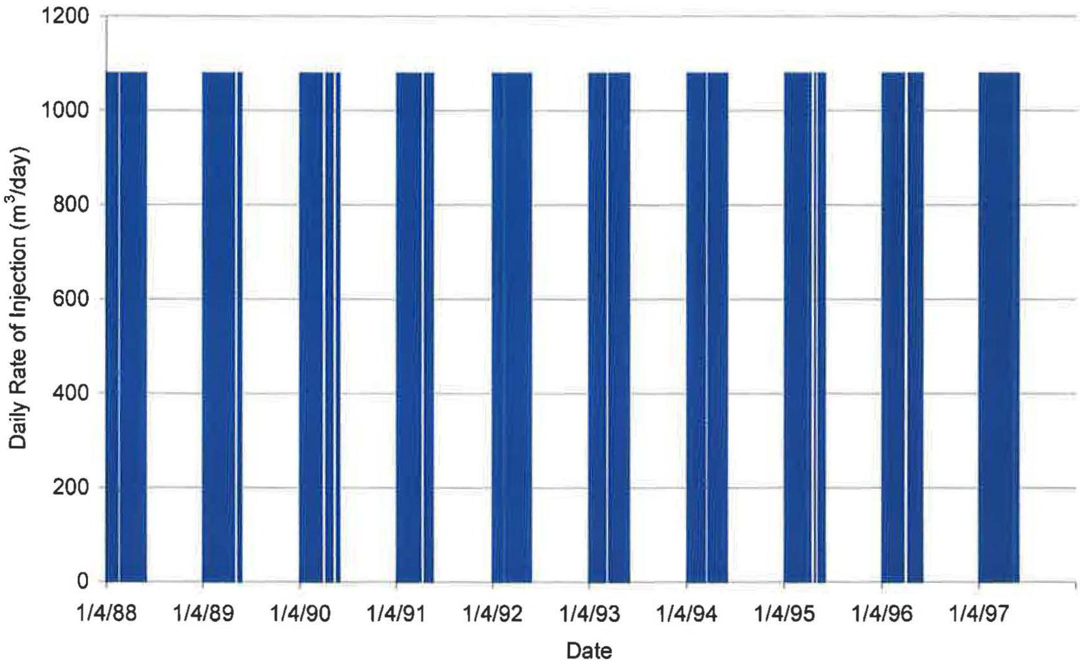



**Figure 6-17 Injected Volume for Wetland 1**

6.3.1 b) Wetland Configuration 2 (Region B)

This wetland configuration is an optimal solution when the PDR is 15 $m^3/s$ and the MRT is 50 days.

**Table 6-3 Configuration for Wetland 2**

| | |
|---|---|
| **Wetland volume** | 185,000 $m^3$ |
| **Weir height** | 238 cm |
| **Weir length** | 400 cm |
| **ASR off-take height** | 167 cm |
| **Total cost (excludes fitness penalty cost)** | $1,487,000 |
| **Construction cost** | $3,212,900 |
| **Operating and maintenance cost** | $324,100 |
| **Irrigation water benefit** | $2,050,000 |
| **Penalty cost** | $710 |
| **Total volume injected** | 1,646,300 $m^3$ |
| **Outflow volume** | 5,954,200 $m^3$ |
| **Evaporation losses** | 883,900 $m^3$ |

The PDR achieved during the 1 in 100 year extreme flood simulation (12.25 $m^3/s$) was well below the required PDR (15 $m^3/s$), because this wetland configuration was located in region B (see Figure 6-7). In region B, the required MRT plays the major role in determining the optimal wetland configuration. The time between the peak of the inflow hydrograph and the peak of the outflow hydrograph was 100 minutes (Figure 6-18).

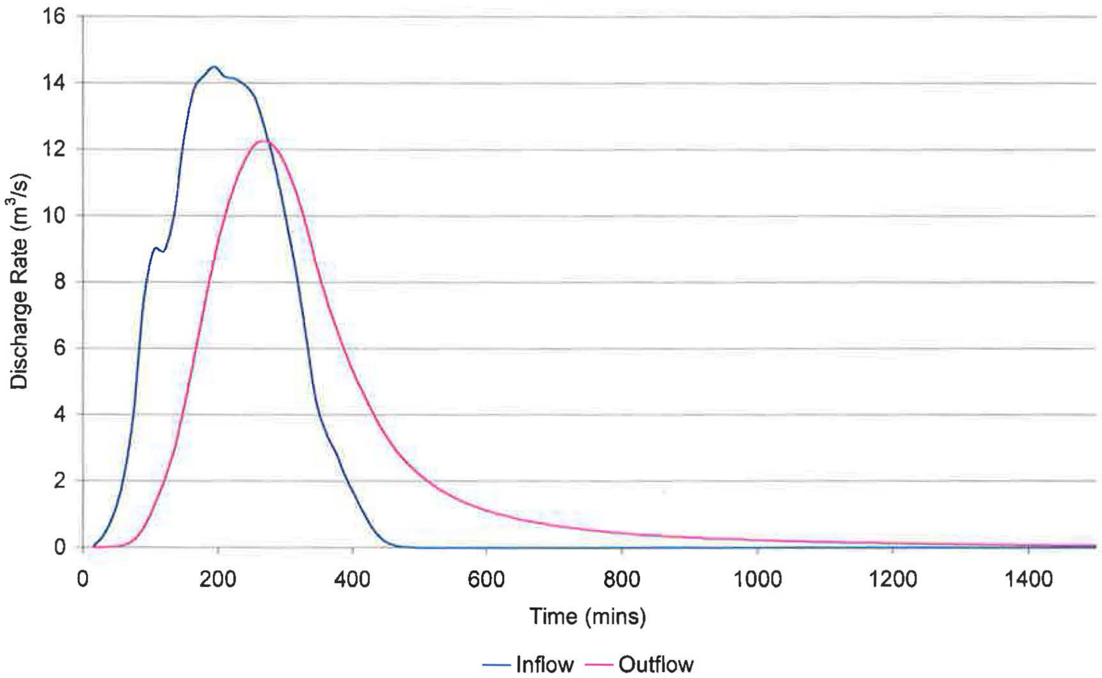



**Figure 6-18 1 in 100 Year Extreme Flood Hydrographs**

The water level stayed a lot closer to the weir during the simulation conducted for wetland 2 (see Figure 6-19) than it did during the simulation conducted for wetland 1. The large weir length reduced considerably the time taken to drain the pond down to the weir level. The minimum water height achieved during the simulation was close to the height of the ASR off-take.

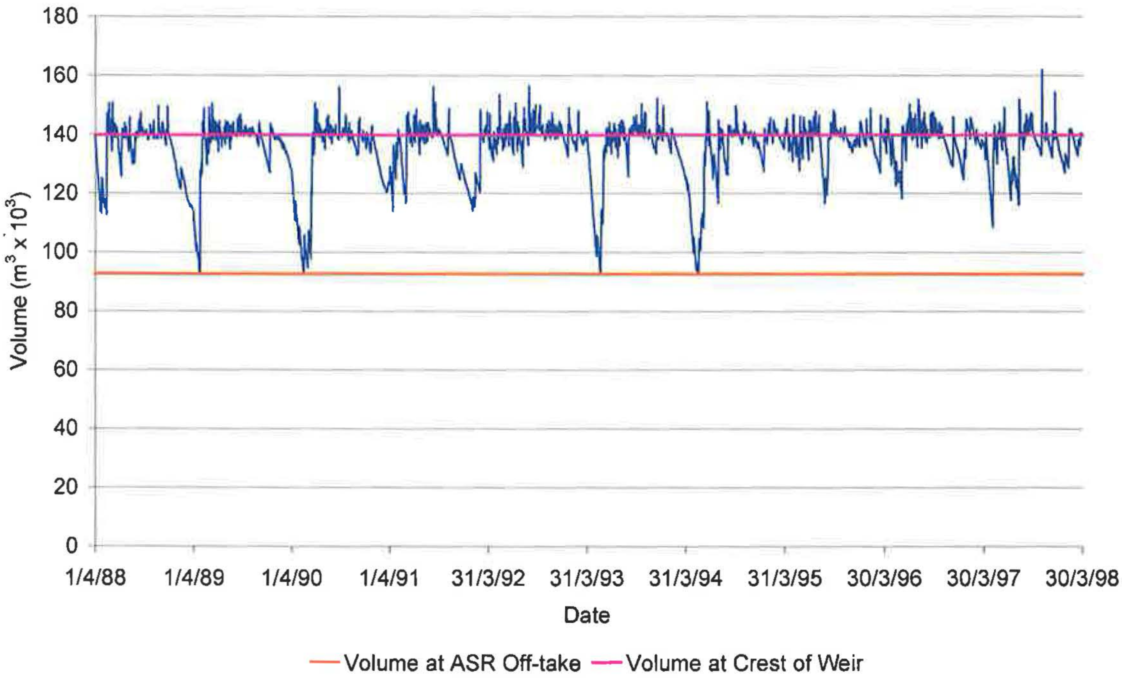



**Figure 6-19 Storage Volume Change for Wetland 2**

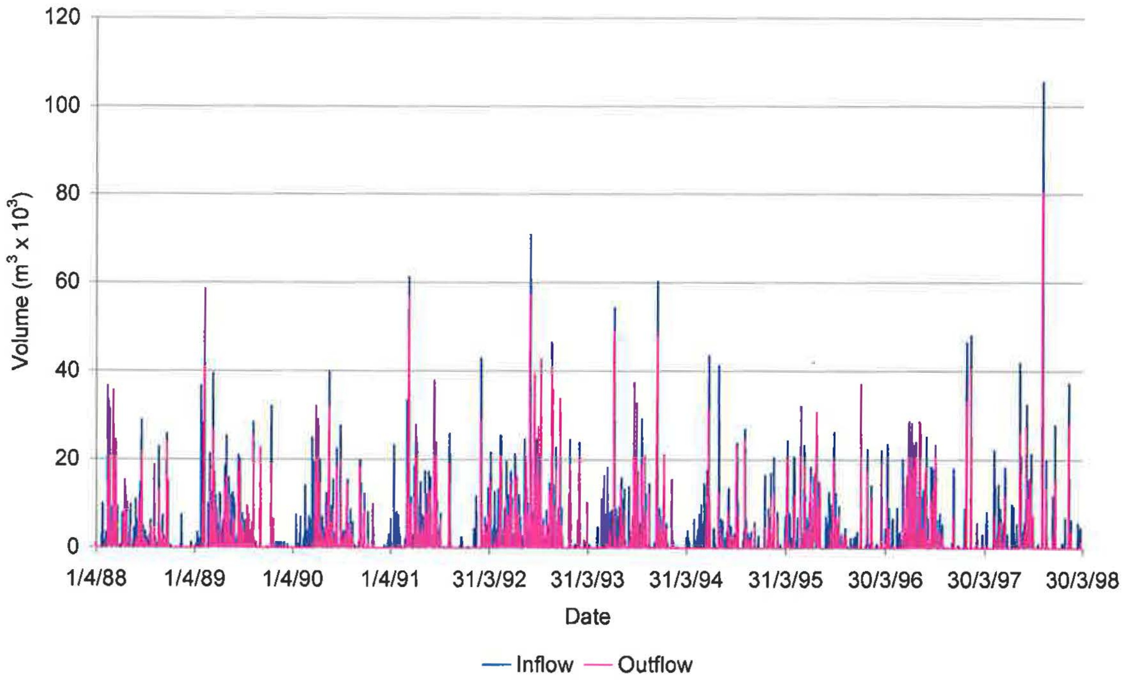



**Figure 6-20 Daily Inflow and Outflow Volumes for Wetland 2**

As in the long-term simulation conducted for wetland 1, there were very few days when injection did not occur (see Figure 6-21).

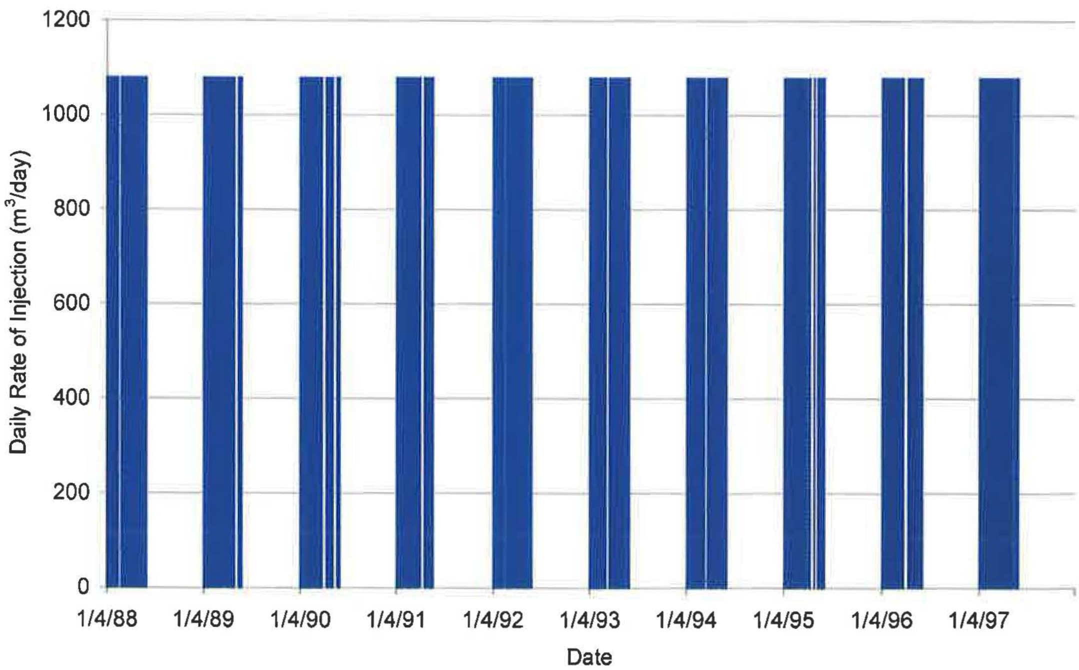



**Figure 6-21 Injected Volume for Wetland 2**

6.3.1 c) Wetland Configuration 3 (Region C)

This wetland configuration is an optimal solution when the PDR is 15 $m^3/s$ and the MRT is 5 days.

**Table 6-4 Configuration for Wetland 3**

| | |
|---|---|
| **Wetland volume** | 49,000 $m^3$ |
| **Weir height** | 215 cm |
| **Weir length** | 395 cm |
| **Pump height** | 100 cm |
| **Total cost (excludes fitness penalty cost)** | \$114,100 |
| **Construction cost** | \$1,234,100 |
| **Operating and maintenance cost** | \$217,100 |
| **Irrigation water benefit** | \$1,337,100 |
| **Penalty cost** | \$1150 |
| **Total volume injected** | 1,073,900 $m^3$ |
| **Outflow volume** | 7,182,100 $m^3$ |
| **Evaporation losses** | 226,100 $m^3$ |

As shown in Figure (6-22), the basin had negligible effect on reducing the peak of the inflow hydrograph. The inflow and outflow hydrographs from the extreme flood simulation are almost identical.

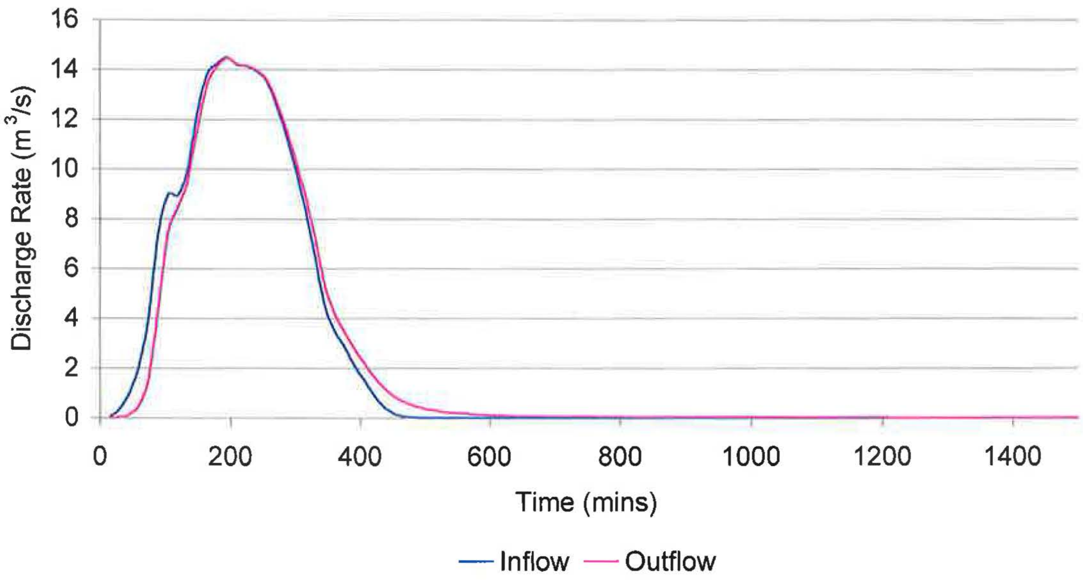



**Figure 6-22 Extreme Flood Hydrographs**

The graph of the storage volume change during the long-term simulation (Figure 6-23) is more variable than the graphs for the other two wetlands (Figures 6-15 and 6-19), due to the much smaller wetland volume. The proportion of time that the water level remains below the weir height was much higher for this wetland, than for wetlands 1 and 2. Again, the minimum water level reached was very close to the height of the ASR off-take. The water level was at the off-take height longer for this wetland than the other two wetlands.

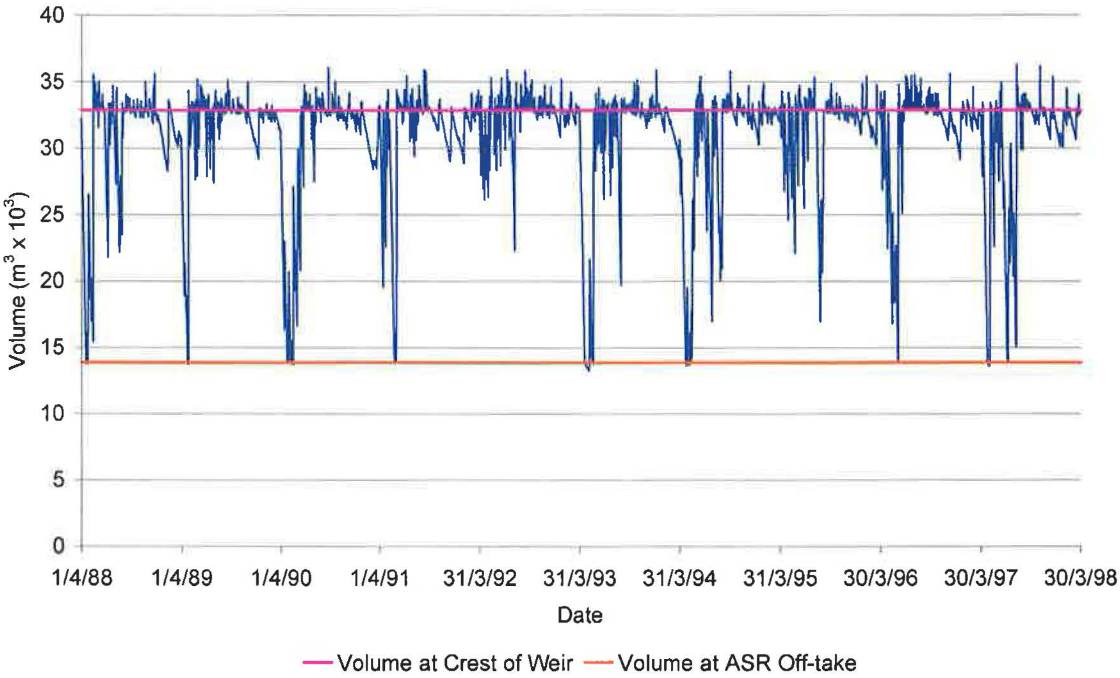



**Figure 6-23 Storage Volume Change for Wetland 3**

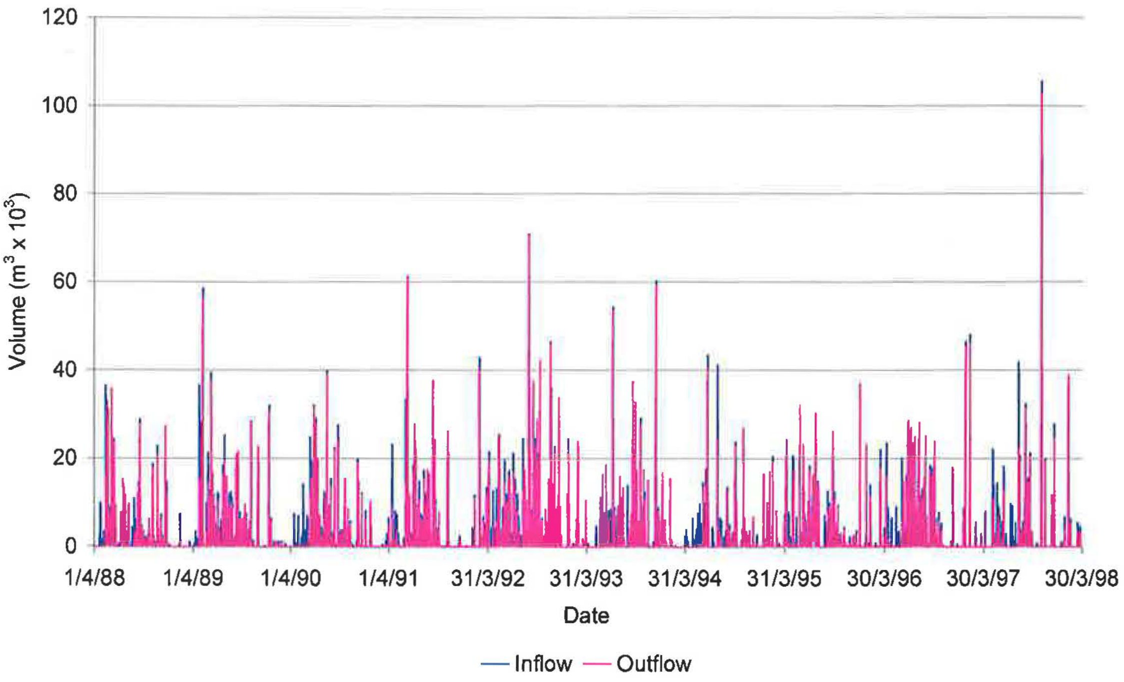



**Figure 6-24 Daily Inflow and Outflow Volumes for Wetland 3**

The number of days when injection occurred was less for this wetland, because the wetland configuration was located in region C. As explained earlier the active storage volume for region C wetlands is not large enough to supply the same volume of injected water as wetlands in the other regions.

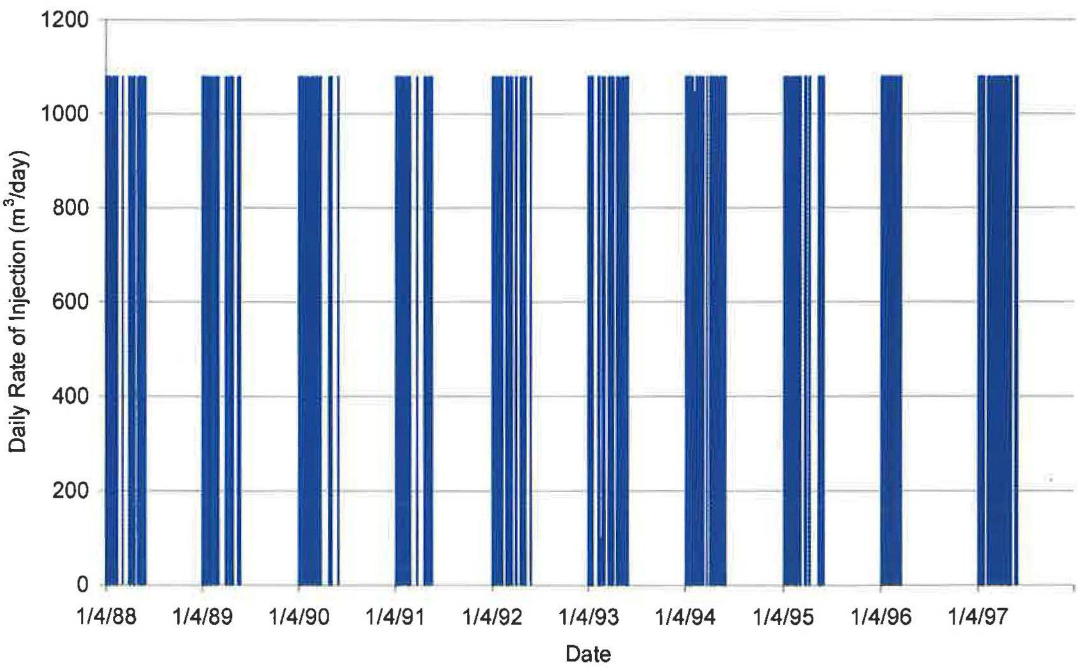



**Figure 6-25 Injected Volume for Wetland 3**

6.3.1 d) Summary Statistics for the 3 Wetland Configurations

Some output statistics obtained from the long-term simulations conducted for the three wetland configurations are summarised in Table (6-5). The total injected volumes for wetland's 1 and 2 were close, however, the total for wetland 3 was significantly less. The large wetland (configuration 1) experienced more evaporation loss and less outflow than the other two configurations. Outflow refers to the water volume flowing over the weir.

**Table 6-5 Summary Statistics**

| Configuration number | Wetland volume ($m^3$) | Weir height (cm) | Weir length (cm) | Off-take height (cm) | Outflow volume (ML) | Injected volume (ML) | Evaporation losses (ML) |
|---|---|---|---|---|---|---|---|
| 1 | 363,000 | 239 | 0 | 186 | 5,074.4 | 1,652.4 | 1,764.4 |
| 2 | 185,000 | 238 | 400 | 167 | 5,954.2 | 1,646.3 | 883.9 |
| 3 | 49,000 | 215 | 395 | 100 | 7,182.1 | 1,073.9 | 226.1 |

## 6.4 Summary

The effect that altering the required PDR and MRT had on the optimal wetland configuration was examined. Three different regions were observed in the optimal value plots of the four decision variables. The three regions correspond to areas of the plots where a different wetland function/s played the major role in determining the optimal wetland configuration.

# Chapter 7 Conclusions and Recommendations

## 7.1 Conclusions

An optimal wetland design framework has been developed that can be used to help identify the most efficient wetland configuration for a proposed multi-functional wetland site.

Four decision variables, weir length and height, basin volume and ASR off-take height were used in the simple one-basin configuration investigated in this study. The simple genetic algorithm proved to be an effective method for optimising this configuration. In most optimisation runs, a near-optimal solution was found within two hundred generations. The simple genetic algorithm is likely to be more useful for optimising a multi-basin wetland system with a greater number of decision variables due to the increased size of the solution space.

Two functional requirements, the reduction in the peak discharge rate during the extreme flood event below a specified discharge rate and the retention of water in the wetland prior to discharge for a specified mean period of time, were varied in this study. The four decision variables were sensitive to changes in these requirements. Restrictions on the range of values each decision variable could attain affected the optimal values of the other decision variables for different combinations of the two functional requirements. If more functional requirements were altered and/or more decision variables optimised, more complicated interactions between decision variables would be anticipated.

## 7.2 Recommendations for Further Research

The results presented in this thesis have highlighted the need for all relevant wetland functions to be taken into account when designing stormwater wetlands. The wetland configuration analysed here had a single basin. If a more detailed configuration is examined, the need for an integrated design procedure is even greater. It is recommended that research be carried out in applying this approach to multi-basin wetlands.

The usefulness of the wetland design framework presented here as a design tool is limited by the accuracy of the models used in the flood and long-term simulations. The main factor that restricts the selection of these models is computer run-time. Since the optimisation procedure requires simulations to be repeated thousands of times, it may not be practical to use detailed models due to time constraints. As computer speeds increase more detailed models could conceivably be used in the framework. Another factor that restricts model selection is the amount and quality of flow gauging and pollutant runoff data available for the catchment. There is no point using a detailed wetland model if the input data is not reliable. Current water quality and flow gauging monitoring networks in urban catchments need to be extended to improve the accuracy of input data.

Another area where improvements in model accuracy will increase the applicability of the framework presented is water quality modelling. Currently, most of the models used in wetland design are zero or one-dimensional. If accurate two-dimensional or even three-dimensional models were available then extra decision variables could be added to optimise design aspects such as basin shape and the location of vegetated zones. Existing two-dimensional models can model flow paths through basins and there are detailed pollutant mass balance models that simulate the major chemical and biological wetland processes, but the accuracy of these models when applied to multiple wetland sites has been inconsistent. More research needs to be conducted on developing improved water quality and flow models for wetlands to increase the applicability of the framework presented.

More data on the effect of different wetland configurations on flow paths and water quality parameters, such as suspended solids, faecal coliforms, nutrients and metals is essential for model development. There needs to be more monitoring conducted on existing wetlands to gain a greater understanding of these complicated systems.

The most useful extension of this research for local government authorities would be to further develop the cost and benefit component of the framework. This extension would comprise the inclusion of all construction, operating and maintenance costs as well as the incorporation of predicted costs and benefits such as, flood damage estimates, environmental costs associated with discharging polluted water into freshwater and aquatic ecosystems, and recreational and educational benefits resulting from construction of the wetland, thus enabling a complete benefit/cost study to be undertaken.

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