

University of Southern Queensland  
Faculty of Engineering & Surveying

**Current and Future Capacity of the Wirraglen Scout  
Reserve Wetlands to Improve Urban Stormwater Quality  
at Highfields**

A dissertation submitted by

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in fulfilment of the requirements of

**ENG4112 Research Project**

towards the degree of

**Bachelor of Engineering (Environmental)**

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# Abstract

The Wirraglen Scout Reserve Wetlands are located on the northern edge of the township of Highfields along Klein Creek. This creek forms part of the catchment area for Cooby Dam, which is part of Toowoomba's water supply. The catchment area is approximately 6.75km<sup>2</sup> in size with much of the Highfields township included within this area. The surrounding terrain is steep with small areas of native eucalypt forest.

The wetland was originally a waterhole but has since silted up to form a natural wetland. It is believed that development within the catchment area has led to this sedimentation. As a result, the storage capacity of the wetlands has been reduced in terms of the quality and quantity of water. Therefore, the aim of this project is to investigate the current capacity of the wetlands and design ways to improve this capacity. This issue will be examined through both the storage and assimilative capacities of the wetland. An analysis will be undertaken to determine the flows for design storm events. These results can then be used with sampling results to determine the effectiveness of the wetlands in treating and detaining urban stormwater.

The Klein Creek catchment upstream of the Wirraglen wetlands was split into subsections for the purpose of design calculations. Using *Australian Rainfall & Runoff* guidelines (Engineers Australia 1998), the Rational Method was used to determine the time of concentration and peak discharges for the catchment. The peak discharge was then compared with the discharge modelled using the RORB modelling software. Average Recurrence Intervals of 2, 10 and 100 years were used in the modelling process. The use of two methods was required due to the lack of available data for the catchment and therefore no possible way of calibrating either model. The results from the two

models correlated fairly well with each other and therefore these results were adopted as the design storms for the catchment.

A water quality analysis was undertaken within the wetland itself. This provided an indication of the current water quality inside the wetland. The absence of rainfall has resulted in no flow into or out of the wetland for the duration of this project. Therefore an understanding of the current assimilative capacity of the was unable to be obtained. This has hindered the design of an improved wetland system. Theoretical models have provided an indication of the water quality improvement as the water flows through the wetland, but more water quality testing is required to obtain an adequate assessment.

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TOBY MILLIKAN

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Date

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*November 2006*

# Contents

<b>Abstract</b>	<b>i</b>
<b>Acknowledgments</b>	<b>v</b>
<b>List of Figures</b>	<b>x</b>
<b>List of Tables</b>	<b>xiii</b>
<b>Nomenclature</b>	<b>xv</b>
<b>Chapter 1 Introduction</b>	<b>1</b>
1.1 Chapter Overview . . . . .	1
1.2 Wetland Overview . . . . .	2
1.3 Background of Study . . . . .	2
1.4 Methodology . . . . .	4
1.5 Overview of the Dissertation . . . . .	5
<b>Chapter 2 Literature Review</b>	<b>6</b>

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2.1	Chapter Overview . . . . .	6
2.2	What is a Wetland? . . . . .	7
2.3	Wetland Processes . . . . .	8
2.4	Surface Flow Wetlands . . . . .	10
2.4.1	Advantages of Surface Flow Wetlands . . . . .	12
2.4.2	Disadvantages of Surface Flow Wetlands . . . . .	12
2.5	Subsurface Flow Wetlands . . . . .	13
2.5.1	Advantages of Subsurface Flow Wetlands . . . . .	15
2.5.2	Disadvantages of Subsurface Flow Wetlands . . . . .	15
2.6	Water Quality Improvements in Surface Flow Wetlands . . . . .	16
2.7	Chapter Summary . . . . .	17
 <b>Chapter 3 Calculation of Peak Inflow into Wetland</b>		<b>19</b>
3.1	Chapter Overview . . . . .	19
3.2	Klein Creek Catchment . . . . .	20
3.3	The Rational Method . . . . .	22
3.3.1	Design Rainfall . . . . .	23
3.3.2	Time of Concentration Calculations . . . . .	24
3.3.3	Peak Discharge Calculations . . . . .	26
3.4	Chapter Summary . . . . .	28

---

<b>Chapter 4 RORB Model</b>	<b>29</b>
4.1 Chapter Overview . . . . .	29
4.2 Overview of RORB . . . . .	30
4.3 Application of RORB . . . . .	31
4.4 Results . . . . .	34
4.4.1 Critical Storm Duration . . . . .	34
4.4.2 Natural Condition Flows . . . . .	35
4.4.3 Current Condition Flows . . . . .	36
4.5 Recommendation . . . . .	39
4.6 Chapter Summary . . . . .	39
 <b>Chapter 5 Water Quality Analysis</b>	 <b>41</b>
5.1 Chapter Overview . . . . .	41
5.2 Water Sampling Locations . . . . .	42
5.3 Upstream Sample . . . . .	42
5.3.1 TS and TSS Test Results . . . . .	44
5.3.2 COD Test Results . . . . .	46
5.3.3 BOD Test Results . . . . .	47
5.4 Downstream Sample . . . . .	49
5.4.1 TS and TSS Test Results . . . . .	50
5.4.2 COD Test Results . . . . .	51

---

5.4.3	BOD Test Results . . . . .	52
5.5	Comparison of Sample Results . . . . .	52
5.6	MUSIC Water Quality Model . . . . .	54
5.7	Chapter Summary . . . . .	56
<b>Chapter 6 Recommendations and Conclusion</b>		<b>58</b>
6.1	Recommendation . . . . .	59
6.2	Achievement of Project Objectives . . . . .	60
6.3	Further Work . . . . .	61
<b>References</b>		<b>62</b>
<b>Appendix A Project Specification</b>		<b>66</b>
<b>Appendix B Klein Creek Catchment Data</b>		<b>68</b>
<b>Appendix C Design Storm Calculations and Results</b>		<b>72</b>
<b>Appendix D RORB Data and Results</b>		<b>89</b>

# List of Figures

1.1	An Aerial View of the Wirraglen Scout Reserve Wetlands . . . . .	3
2.1	The Wirraglen Scout Reserve Wetlands . . . . .	7
2.2	The Basic Principle Behind a Surface Flow Constructed Wetland . . . .	11
2.3	The Basic Principle Behind a Sub-Surface Flow Constructed Wetland .	14
2.4	A Newly Constructed Subsurface Flow Wetland . . . . .	14
3.1	Wirraglen Scout Reserve Wetlands . . . . .	21
3.2	Vegetation Present within Wetland . . . . .	22
4.1	The Football Grounds on Kuhls Rd used as a Detention Basin . . . . .	32
4.2	The Outlet Structure at the Wetlands . . . . .	33
5.1	Location of Samples Taken . . . . .	42
5.2	Location of Upstream Water Sample . . . . .	43
5.3	Location of Downstream Water Sample . . . . .	49
5.4	Diagram Used for MUSIC Model . . . . .	55

---

B.1	Location of Wetlands . . . . .	69
B.2	Wetland Catchment as part of the Cooby Dam Catchment . . . . .	70
B.3	Map of Land Use Type for Crows Nest Shire . . . . .	71
C.1	Coefficient of Runoff vs Development Category for Crows Nest Shire . .	73
C.2	Rainfall Intensity for Time of Concentration < 60 minutes for Highfields	74
C.3	Rainfall Intensity for Time of Concentration > 60 minutes for Highfields	75
C.4	Catchment Subdivision with Flow Paths . . . . .	76
C.5	IFD Curve for Highfields . . . . .	77
D.1	Catchment Subdivision for RORB Model . . . . .	90
D.2	Catchment File used in RORB Model for Natural Conditions . . . . .	91
D.3	Catchment File used in RORB Model for Current Conditions . . . . .	92
D.4	Storm Duration Plot for 2yr Design Storm . . . . .	93
D.5	Storm Duration Plot for 10yr Design Storm . . . . .	94
D.6	Storm Duration Plot for 100yr Design Storm . . . . .	95
D.7	Runoff Hydrograph for a 2yr Design Storm assuming Natural Conditions	96
D.8	Runoff Hydrograph for a 10yr Design Storm assuming Natural Conditions	97
D.9	Runoff Hydrograph for a 100yr Design Storm assuming Natural Conditions	98
D.10	Inflow and Outflow at Detention Basin for 2yr Design Storm . . . . .	99
D.11	Inflow and Outflow at Detention Basin for 10yr Design Storm . . . . .	100



D.12 Inflow and Outflow at Detention Basin for 100yr Design Storm . . . . .	101
D.13 Current Inflow and Outflow at Wetland for 2yr Design Storm . . . . .	102
D.14 Current Inflow and Outflow at Wetland for 10yr Design Storm . . . . .	103
D.15 Current Inflow and Outflow at Wetland for 100yr Design Storm . . . . .	104

# List of Tables

2.1	Long Term Removal Rates for Pollutants in Stormwater Wetlands . . .	17
3.1	Summary of Flow Times for each Sub-Area and ARI . . . . .	26
3.2	Summary of Design Flows for each Sub-Area and ARI . . . . .	27
4.1	Assumed Parameters for Wetland Outlet . . . . .	32
4.2	Results of Critical Storm Duration Run . . . . .	35
4.3	Summary of Natural Condition Simulation . . . . .	36
4.4	Summary of Results for Detention Basin . . . . .	37
4.5	Summary of Results for Wetland . . . . .	37
4.6	Comparison in Peak Inflow between RORB and Rational Method . . . .	38
5.1	Total Solids Test Results for Upstream Sample . . . . .	44
5.2	Total Suspended Solids Test Results for Upstream Sample . . . . .	45
5.3	Sample and Reagent Quantities used in COD Test . . . . .	46
5.4	Chemical Oxygen Demand Test Results for Upstream Sample . . . . .	47

---

5.5	BOD Test Results for Upstream Sample . . . . .	48
5.6	Total Solids Test Results for Downstream Sample . . . . .	50
5.7	Total Suspended Solids Test Results for Downstream Sample . . . . .	51
5.8	Chemical Oxygen Demand Test Results for Downstream Sample . . . . .	51
5.9	BOD Test Results for Downstream Sample . . . . .	52
5.10	Summary of Test Results . . . . .	53
5.11	Summary of Results for MUSIC Simulation . . . . .	56



# Nomenclature

ABS	Australian Bureau of Statistics
AR&R	<i>Australian Rainfall and Runoff</i>
ARI	Average Recurrence Interval
BOD	Biochemical Oxygen Demand
CNSC	Crows Nest Shire Council
COD	Chemical Oxygen Demand
DDROC	Darling Downs Regional Organisation of Councils
DEH	Department of Environment and Heritage
DNR	Department of Natural Resources
DO	Dissolved Oxygen
DWLBC	Department of Water, Land and Biodiversity Conservation
FAS	Ferrous Ammonium Sulfate
FWS	Free Water Surface
GCCC	Gold Coast City Council
IFD	Intensity-Frequency-Duration
IWA	International Water Association
MUSIC	Model for Urban Stormwater Improvement Conceptualisation
RORB	Runoff Routing on Burroughs
SF	Surface Flow
SSF	Sub-Surface Flow
TCC	Toowoomba City Council
TDS	Total Dissolved Solids
TN	Total Nitrogen
TP	Total Phosphorus
TS	Total Solids
TSS	Total Suspended Solids

# Chapter 1

## Introduction

### 1.1 Chapter Overview

The Wirraglen Scout Reserve Wetlands are located along Klein Creek near Highfields, North of Toowoomba. Klein Creek forms part of the Cooby Dam Catchment and therefore it is essential that a good quality of water is maintained in this watercourse. The Crows Nest Shire Council has developed this project in order to determine what role the wetlands play in improving urban stormwater quality flowing out of Highfields. This chapter provides a brief overview of the requirements of the project and the methodology used to complete these requirements. The specification for this project is provided in Appendix A.

## 1.2 Wetland Overview

There are two broad categories of wetlands: natural and constructed. Constructed wetlands can be split into surface flow or sub-surface flow wetlands. Wetlands used for urban runoff are generally surface flow wetlands as these mimic the processes that occur within a natural wetland. The Wirraglen Scout Reserve Wetlands are fairly unique in that a natural waterhole has been affected by human activity to affect its assimilative and storage capacities. Any alterations to the wetland would place it to some extent in the constructed surface flow wetland category. As a result, surface flow wetlands will be considered in most detail although the properties of sub-surface flow wetlands will also be discussed.

The Wirraglen Scout Reserve Wetlands are natural wetlands located in a fairly steep section of terrain just to the north of Highfields. The catchment area incorporates a large percentage of the township of Highfields which has recently experienced a very large growth in population. Crows Nest Shire in which Highfields is located is currently the 16th fastest growing local government body in Australia (ABS 2006). The resultant development is believed to have caused significant sedimentation in the wetland, which was originally a waterhole. Figure 1.1 shows the wetland when a significant volume of water was held in storage. The exact date of this image is unclear, although it is apparent that it was taken during a higher period of rainfall due to the flow that can be observed in Klein Creek. The water level for the duration of this project was much lower than what is shown in this image. There are now lots of reeds growing in the wetland which may or may not be the result of sedimentation. No flow into or out of the wetland was observed during the course of this project.

## 1.3 Background of Study

The Crows Nest Shire Council is concerned about the quality of water flowing off urban areas into surrounding waterways. The Klein Creek catchment is particularly important as this forms part of the Cooby Dam catchment area. Cooby Dam is part of the Toowoomba City water supply. As a result it is crucial that a satisfactory



Figure 1.1: An Aerial View of the Wirraglen Scout Reserve Wetlands

*Source: Crows Nest Shire Council*

quality of water is maintained, particularly considering the severe drought currently being experienced in the region.

Wetlands have the potential to provide a significant improvement in water quality from urban runoff. The Wirraglen Scout Reserve Wetlands, North of Highfields provides the opportunity to test the quality of urban runoff from Highfields and the role these wetlands can have in improving this quality. This project attempts to determine the current quality of water within the Wirraglen Scout Reserve Wetlands and how this ecosystem can be altered to ensure a sufficient standard of water flowing down Klein Creek. The project also examines the quantity of water flowing into the wetlands. The peak flow during a storm event is the critical time at which a wetland needs to be able handle the load placed upon it. This is the most likely time that the storage capacity of the wetland may be exceeded, possibly increasing the nutrient load in the outflow. These measures of quantity and quality are therefore interrelated as it is crucial to water quality improvement that the storm flow is able to be detained in the wetland for a sufficient period of time.



## 1.4 Methodology

Wetlands serve two purposes in handling stormwater runoff. Due to the aquatic life present within the ecosystem, they have the potential to improve the quality of stormwater runoff. They are also capable of performing the same role as a detention basin; that is to reduce the peak discharge flowing off an urban area as well as increasing the time taken to reach this peak. Both the assimilative and storage capacities therefore need to be taken into consideration when altering the current features of the wetland.

### Quantity

The Klein Creek catchment is ungauged and therefore no storm events can be used to test the accuracy of any models developed. Two separate models will therefore be used in order to provide a comparison of the results. The rational method will be used to provide an empirical estimate of the time of concentration and peak discharge estimates for the design storms. These results will be compared with the computer based RORB model. RORB has the advantage that the entire storm flow can easily be modelled. A detention basin on Kuhls Road in Highfields can also be included in the modelling. Weir structure parameters can also be entered which means that the outflow from the wetlands can also be modelled. In this case the roadway crossing Klein Creek acts as the weir structure. The difference between the inflow and outflow gives an estimate on the actual detention time in the wetland which will have an impact on water quality improvement. The values can then be easily altered in the model to find a combination that is capable of handling the design storm.

### Quality

Water samples were taken in at two of different locations within the wetland. It was decided that two samples would be required in the wetland to indicate the improvement in quality as the water passes through the wetland. Samples are required both upstream and downstream of the wetland. Unfortunately, no runoff has been measured into the wetland between March 2006 and the conclusion of this project. There has also been no flow out of the wetland during this period. This makes it very difficult to assess the role that the wetland plays in improving urban stormwater quality. A computer modelling package, MUSIC has been used to estimate the concentration of total suspended solids,

phosphorus and nitrogen flowing into and out of the wetlands. These results along with the results obtained from the water samples taken within the wetlands can help to assess the wetlands role in water quality improvement.

## 1.5 Overview of the Dissertation

This dissertation will have the following organisational structure:

**Chapter 2** describes the types of wetlands and the processes that occur within them.

**Chapter 3** discusses the the use of the Rational Method in finding the time of concentration and calculating the peak discharge out of the wetland.

**Chapter 4** discusses the RORB Model and compares the results obtained from this model and that from the Rational Method. The effectiveness of the wetlands in handling these design storms will also be discussed and any modifications made to the wetland design if required.

**Chapter 5** provides information on the current water quality within the wetlands through the analysis of water samples taken within the wetland and computer modelling results using the MUSIC software.

**Chapter 6** concludes the results and outcomes of this project and outlines further work that needs to be completed before a more accurate improved wetland design can be obtained.

## Chapter 2

# Literature Review

### 2.1 Chapter Overview

Wetlands have been used for both urban stormwater and wastewater treatment in various parts of the world, although most frequently through parts of Europe and North America. There is therefore a significant amount of literature describing the benefits of wetland systems, especially for the use of wastewater treatment. This is most likely due to the fact that wastewater places a greater load on the wetlands. The purpose of this thesis is to determine the use of wetlands in improving stormwater quality. Before going into any great detail on the processes involved in water quality improvement in wetlands, it is important to give a brief overview on the various types of wetland systems.

## 2.2 What is a Wetland?

A wetland does not have one set definition. There are many definitions depending on the wetland function of interest. One such definition is provided by Kent (1994, p. 5):

“Wetland is defined as land having the water table at, near, or above the land surface or which is saturated for a long enough period to promote wetland or aquatic processes as indicated by hydric soils, hydrophilic vegetation, and various kinds of biological activity which are adapted to the wet environment.”

This definition adequately describes the basic requirement for an area of land to be classified as a wetland. It should also be noted that wetlands may be called by many other names such as “swamps, marshes, billabongs, saltmarshes, mudflats, mangroves, fens and peatlands” (DEH 2004). The Wirraglen Scout Reserve Wetlands is shown in Figure D.2.



Figure 2.1: The Wirraglen Scout Reserve Wetlands

While this project will be focusing on water quality improvement and storage capability of wetlands, it should also be noted that wetlands also have a number of other advantages. Wetlands provide a habitat for a wide diversity of plant and animal species, are considered aesthetically pleasing and offer an area for recreational activities (DWLBC 2005). Wetlands should therefore not be considered a nuisance, or waste of space, but rather an asset for the surrounding community.

Wetlands are not always naturally occurring. Humans have also developed wetlands to mimic the natural process that occur in natural wetlands, such as Kakadu National Park. Constructed wetlands are used to improve the water quality of either stormwater or wastewater. There are two main types of constructed wetlands. These are Free Water Surface (FWS) or Surface Flow (SF) wetlands, and Sub-surface Flow (SSF) wetlands (IWA 2000). The advantages and disadvantages of these systems will be discussed after detailing the processes involved within a wetland system.

## 2.3 Wetland Processes

There are many naturally occurring processes that occur within a wetland system. These processes result in wetlands being effective mechanisms in treating biochemical oxygen demand (BOD), suspended solids, nitrogen and phosphorus (IWA 2000). This generally results in a much higher quality of effluent flowing out of the wetland system. There are three broad processes that occur within a wetland system that contribute to water quality improvement:

- Biological and chemical processes
  - Uptake of nutrients by epiphytes
  - Adsorption and desorption
  - Nitrification and denitrification
- Coagulation and filtration of small colloidal particles
  - Adhesion of colloids and particles on surface of aquatic vegetation

- Physical sedimentation of particles
  - Decrease in water velocity
  - Reduction in turbulence

(Wong, Breen, Somes & Lloyd 1999)

Wetlands are very beneficial in controlling stormwater flow. A heavy rainstorm event will cause a sharp peak in the flowrate of the watercourse. This is especially a problem in developed areas where the decreased permeability of paved areas results in a more intense peak in flowrate. Wetlands can be used to hold this water in detention and therefore lower the peak flowrate downstream of the wetland. This is achieved through a physical process within the wetland where water is allowed to spread out over a wider area and aquatic plants assist in attenuating the flowrate. The river system downstream will therefore benefit as the risk of erosion is reduced.

The reduction in flowrate within the wetland also has a number of other benefits. This reduced flow allows for sediments to drop out of suspension. Additional pollutants as a result of human activities, such as toxic organic materials and heavy metals will also be reduced (Graham 2003). These pollutants often build up on roadways. There will therefore be a higher concentration of these pollutants in highly built up areas. A storm event after a prolonged dry period will also result in a much higher pollutant loading during the initial runoff (IWA 2000). Vegetation within the wetland is important for pollutant control. A wide diversity of plants is preferred as this encourages a more diverse range of fauna to the wetland. The plants also assist in attracting bacteria to the wetland which help to break down the pollutants. This prevents a build up of these contaminants on the floor of the wetland which may then inhibit plant growth (Moshiri 1993).

Natural wetlands should be considered as a fragile environment. Like most natural processes, a dramatic change to the environment may degrade the effectiveness of that process. The same may occur in natural wetlands when a large change to the surrounding environment occurs. Wetlands will most likely be affected when untouched land is altered for human purposes, such as agriculture or urban development. Natural wetlands are therefore not recommended to be used for pollution control, as this

could cause a significant disruption to the natural ecosystem (Kent 1994). The use of constructed wetlands are preferred as these can be custom made to cope with the pollutant demand. Such projects have occurred in many parts of the world, including Adelaide where numerous wetland projects have been used for pollution control, biodiversity, local amenity, education, water reuse and flood storage and control. Significant improvements in water quality flowing out of these wetlands were recorded soon after the construction of the wetlands, including a 90 percent removal of some pollutants (DWLBC 2005).

The processes that occur within wetlands are a combination of physical, chemical and biological. These three broad processes lead to stormwater detention, nutrient and pollutant removal and the breakdown of some of these pollutants with the assistance of bacteria. There are however limits to the volume of inflow and pollutant concentration that wetlands can handle. It is therefore important that natural wetlands are not abused in this way, but rather use purpose built constructed wetlands for these processes.

## 2.4 Surface Flow Wetlands

A surface flow (SF), or free water surface (FWS) constructed wetland is designed to mimic the processes which occur within a natural wetland. The inlet and outlet structures must be designed to handle the peak flow, while still providing a long enough detention time to remove pollutants. Typical hydraulic loading rates for SF wetlands range from 0.7 to 5.0cm/d (Kadlec & Knight 1996). The water in SF wetlands is above ground and therefore the plant species must be able to cope with a submerged root zone. The wetland basin must be shallow and the water control structure must be able to maintain this shallow depth. The soil type within the wetland must also be sufficient to be able to support the roots of vegetation (IWA 2000). It has been found that a wide diversity of plant species is generally more responsive to variations in pollutant loading than monocultures. The main use of the vegetation in SF wetlands is to provide a breeding ground for microorganisms which act to break down pollutants in the water (Moshiri 1993). Although there are these similarities with natural wetlands, there are also a number of significant differences which need to be considered:



- Constructed wetlands remain constant in size;
- They are not directly connected with groundwater;
- They accommodate greater volumes of sediment;
- They more quickly develop the desired diversity of plants and associated organisms.

(Magmedov n.d.)

From these points it is made apparent that natural wetlands should not be used specifically for sediment and stormwater control. Their use for this purpose would harm the wetland ecosystem as well as providing a less efficient system. Surface flow wetlands have also been found to be more effective than SSF wetlands in treating stormwater. A basic design of a surface flow wetland is shown in Figure 2.2.

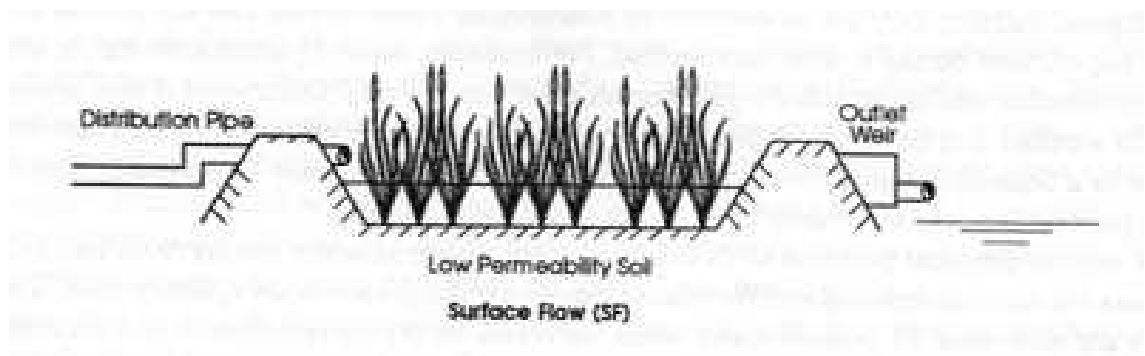


Figure 2.2: The Basic Principle Behind a Surface Flow Constructed Wetland  
(Kadlec & Knight 1996)

As can be seen from this figure, the inlet and outlet structures are two of the most important features in this type of wetland. The design of these structures must be able to handle the peak flow so that pollutants do not flow over into the waterway. The depth of water within the wetland is also important. It is generally shallow, typically between 0.3m and 0.6m (Newman 1994). These shallow depths mean that sediment is more easily trapped as all the sediments are passing close to the root zone. The outlet pipe should therefore be positioned so that the water level does not exceed a predetermined level. This is important in maintaining the growth and survival of



the aquatic plants and the functioning capacity of the wetland (Moberly 2001). The permeability of the soil is also an important factor to be considered. A low permeable soil is necessary to stop large volumes of water seeping beneath the wetland and possibly into the water table below. This water may be high in pollutants and there is therefore a possibility that the water table below may also become contaminated.

### 2.4.1 Advantages of Surface Flow Wetlands

Surface flow wetlands are most commonly used for stormwater treatment. There are many benefits of using constructed wetlands for the treatment of urban runoff, and have been outlined by the DWLBC (2005):

- Less impact of stormwater on the aquatic environment resulting from reduced stormwater volumes, flow peaks and pollutants;
- Improved biodiversity;
- Improved amenity and recreational benefits;
- Opportunity to use wetland facilities to help educate communities about catchment management issues and to encourage their involvement;
- Increased opportunity for water reuse;
- Opportunity for improved drainage and flood management.

Many of these advantages would not occur with SSF wetlands as all the water flows beneath the ground surface. Having the water above the ground surface provides for many recreational benefits and improves the visual quality of a community.

### 2.4.2 Disadvantages of Surface Flow Wetlands

There are a number of factors that need to be addressed when designing a SF wetland. The DWLBC (2005) has also outlined two potential issues which should be addressed

during the design stage:

- Mosquito Breeding;
- Public Safety.

The control of mosquitos is very important as they may pose a serious threat to public health. They are a carrier of debilitating diseases such as Dengue Fever and Ross River Virus and should be kept under strict control. Mosquitos breed well in standing water; it is therefore important that a reasonable flow can be maintained through the wetland. There is also a species of fish, the air-gulping mosquito fish (*Gambusia affinis*), that feed on mosquito larvae (Mitsch & Gosselink 2000). These fish have the potential to reduce the mosquito population. However, this fish is considered a pest in Australia as they also feed on the eggs on native fish species. It is therefore illegal to relocate or release *Gambusia* into Australian waters (DNR 2000).

Public safety is a major issue in wetland construction. The main risk is with drowning incidents. Surface Flow Wetlands should therefore be designed to minimize this risk. There may also be a problem with undesirable animals migrating to the wetland. A control program may need to be implemented to control these animals.

## 2.5 Subsurface Flow Wetlands

Subsurface flow wetlands differ from SF wetlands due to the water flowing beneath the ground surface. A bed of soil or gravel, typically no more than 0.6m deep is used for plant growth and as a substrate for the water to flow through. Subsurface flow wetlands are able to handle higher flowrates than SF wetlands. Typical hydraulic loading rates for sub-surface flow wetlands range from 2 to 20cm/d (Kadlec & Knight 1996). The design of a SSF wetland is shown in Figure 2.3.

Like a SF wetland, the design of a SSF wetland must be able to handle the design flow. The major limiting factors in the design are once again the inlet and outlet structures. Sufficient time must be provided to bring any sediments and pollutants out

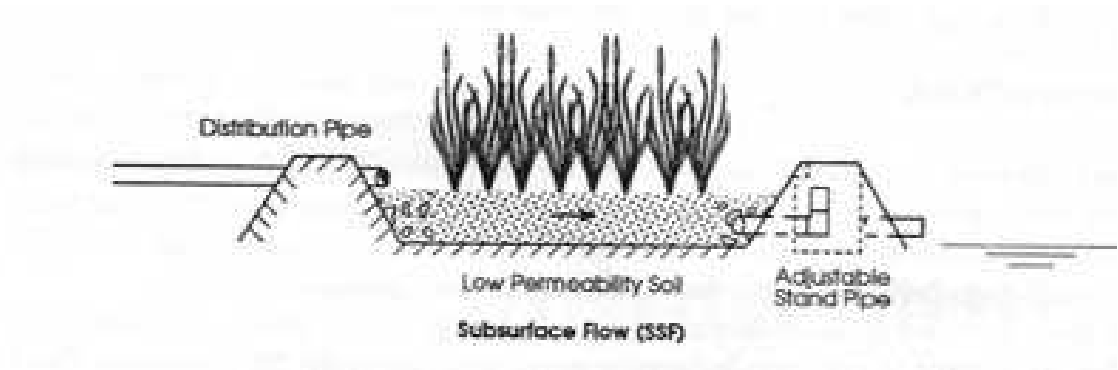


Figure 2.3: The Basic Principle Behind a Sub-Surface Flow Constructed Wetland (Kadlec & Knight 1996)

of suspension, while maintaining a water level below the ground surface. There is also a common problem with an inadequate hydraulic gradient. This means that the resulting surface flows are reduced (Kadlec & Knight 1996). Sub-surface flow wetlands are often used only when low flow rates are required, such as with small wastewater treatment (IWA 2000). These wetlands are therefore rarely used to treat stormwater where high flowrates often occur. Due to the porous medium and no flow of water above the ground surface, SSF wetlands appear more like a wastewater treatment facility than a wetland, such as in Figure 2.4.



Figure 2.4: A Newly Constructed Subsurface Flow Wetland (City of Austin 2001)

### 2.5.1 Advantages of Subsurface Flow Wetlands

Many of the problems involved with SF wetlands are overcome with SSF wetlands, as there is no above ground water to create any drowning hazards. There is also no standing water as the water flows laterally through the medium (Mitsch & Gosselink 2000). This prevents creating a mosquito breeding ground. Other advantages of SSF wetlands have been provided by Halverson (2004).

- Higher rates of contaminant removal per unit of land than SF wetlands, thus they require less land to achieve a particular level of treatment;
- Lower total lifetime costs and capital costs than conventional treatment systems;
- Less expensive to operate than SF wetlands;
- Minimal ecological risk due to absence of an exposure pathway;
- More accessible for maintenance because there is no standing water; and
- Odors and insects not a problem because the water level is below the media surface.

### 2.5.2 Disadvantages of Subsurface Flow Wetlands

Subsurface flow wetlands also have a number of disadvantages. One of the main disadvantages is that they do not provide an attractive area for recreational activities. This is one of the reasons why this type of wetlands is often limited to treating wastewater rather than stormwater. The biodiversity of the area would not be significantly improved as it is with SF wetlands. Other disadvantages of SSF wetlands according to Halverson (2004) include:

- Requires more land than traditional treatment methods;
- May be slower to provide treatment than conventional treatment;
- More expensive to construct than SF wetland on a cost per area basis; and

- Water containing high suspended solids may cause plugging.

Subsurface flow wetlands are not common in Australia, and only tend to be used for wastewater treatment. Subsurface flow wetlands can not really be considered in the case of the Wirraglen Scout Reserve Wetlands as there is already a wetland currently located on the site and therefore this solution would not be practical. Future discussion on constructed wetlands will therefore be referring to surface flow wetlands.

## 2.6 Water Quality Improvements in Surface Flow Wetlands

Significant studies have occurred to determine the effectiveness of wetlands in the treatment of urban runoff. There are a number of factors that influence the effectiveness of a wetland system. These have been outlined by Wong et al. (1999, p.2) as follows:

- Catchment runoff characteristics of respective site
  - climate
  - catchment size
  - land use
- Design and surface area of wetland system

It has been found that vegetation plays a key role in the removal efficiency of pollutants. In general, multiple plant species would be more responsive to load variations than would monocultures (Moshiri 1993). A list of some effective wetland plant species has been provided by *Australian Wetlands* (n.d.) and is shown below:

- *Schoenoplectus validus*
- *Baumea articulata*
- *Schoenoplectus mucronatus*

- *Carex appressa*
- *Lepironia articulata*
- *Juncus usitatus*

These factors affect the removal efficiency of pollutants such as suspended solids, nitrogen, phosphorus and carbon. Research has been conducted in the United States of America to determine long term removal rates of these pollutants. These results are shown in Table 2.1. As these results are valid only for the mid-Atlantic region of the

Table 2.1: Long Term Removal Rates for Pollutants in Stormwater Wetlands

TSS	75%
Total Nitrogen	25%
Total Phosphorus	45%
Organic Carbon	15%
Lead	75%
Zinc	50%
Bacteria	$2\log(10^{-2})$ decrease

(IWA 2000, p. 30)

USA, they only provide an indication of the water quality improvement that may occur in South-East Queensland. According to Graham (2003) results have been inconsistent throughout South-East Queensland. There is therefore no data to compare with the results from the Wirraglen Scout Reserve Wetlands. Significant testing is required at the wetland site with samples required both within the wetland, as well as upstream and downstream of the inlet and outlet respectively. Results from the samples will be used to propose what can be done in the future to improve the storage and assimilative capacities of the wetland. The opportunity of taking these samples will depend entirely on sufficient rainfall during the course of this project.

## 2.7 Chapter Summary

This chapter has provided an explanation of what a wetland is and introduced the different types of constructed wetlands that can be used for water quality improvement.

Surface flow and subsurface flow wetlands were both discussed including advantages and disadvantages of each. It is apparent that subsurface flow wetlands are rarely used for stormwater treatment and particularly in the case of the Wirraglen Scout Reserve Wetlands this is not an option for an improved design.

## Chapter 3

# Calculation of Peak Inflow into Wetland

### 3.1 Chapter Overview

The Klein Creek Catchment is ungauged and therefore no measured data is available to test how the wetland handles peak flows. A design storm can be used for the purpose of determining the peak inflow into the wetland and the time that this peak occurs. This can then be compared with corresponding values for flows out of the wetlands. The time difference between the peak inflow and outflow can help give an estimation of whether or not the runoff is held in the wetland for a sufficient period of time to provide an acceptable improvement in water quality.

This chapter will firstly discuss the Klein Creek catchment in greater detail. The characteristics of the catchment are important in being able to determine the parameters to calculate a design storm event. The process used in calculating the design storm and the results obtained from these calculations will then be explained. The design storm was analysed using the Rational Method and the RORB runoff routing modelling software. The RORB model will be discussed further in Chapter 4.



## 3.2 Klein Creek Catchment

The Klein Creek is located near the town of Highfields, 10 kilometres North of Toowoomba. The creek forms part of the catchment for Cooby Dam, which supplies Toowoomba and surrounding towns with drinking water. The location of the wetland in relation to Highfields and Toowoomba, and the catchment area for the wetlands are presented in Figures B.1 and B.2 in Appendix B respectively. Highfields is predominantly residential and therefore high quantities of heavy metals would not be expected in runoff. Crows Nest Shire Council has provided a map of the land use types for the shire, which is included as Figure B.3 in Appendix B. Unfortunately much of the land around Highfields has an undetermined classification, which is of little assistance except for the area around the wetlands. This figure shows that the land is mainly used for residential and grazing purposes. Animal wastes, fertilizer and natural organic matter would be considered to be the main substances influencing the quality of water flowing out of the catchment.

Highfields lies on a ridge along the Great Dividing Range. The township itself is on a reasonably flat plateau, with an increase in steepness moving downstream along the creek. Figure 3.1 shows the wetland and surrounding land. The steep slope of the land can be seen at the top right of the image. The two side slopes converge further up the wetland until the wetland is only a couple of metres wide. This results in a very long, narrow wetland which provides a longer detention time for the runoff from Highfields. The runoff to the East of the wetland flows off the steeper section of catchment and enters the wetland near the downstream end. A similar phenomenon occurs on the shorter western side. Due to the fact that these sections of the catchment join Klein Creek near the downstream end of the wetland, it is assumed that the detention time is fairly short. The wetland would also be less efficient in treating the water from these areas. However, it is unlikely that this has a major impact on the quality of outflow from the wetlands due to the much smaller volumes flowing off these sections.

Figure 3.1 also shows some of the natural plant species present within the wetland. The wetland area itself is largely filled with reeds, particularly at the downstream end where this image was taken. The upstream end has mainly short grasses due to little or no water being present above the ground surface. The central area of the wetlands



Figure 3.1: Wirraglen Scout Reserve Wetlands

is shown in Figure 3.2. This area is shallower than the downstream end but is still fairly choked with reeds. This figure also shows in the centre of the image a noticeably different species of reed or grass. This was the only stand of this species recorded in the wetland. There were also a few small native trees and shrubs growing close to the edge of the water surface. The surrounding slopes had native eucalypt forest to the West and dense scrub to the East. The presence of this vegetation surrounding the wetlands means that erosion of the surrounding banks is unlikely. No land or marine animals were sighted around or in the wetland except for some birds. It is expected that native animals such as kangaroos and snakes may sometimes inhabit the site.

Highfields is a high growth area and as a result has seen significant development in recent times. The population of Crows Nest Shire increased 4.7% between 2004 and 2005, the 16th fastest growing local government area in Australia (ABS 2006). There is no longer any significant land available for development in the Klein Creek catchment area of Highfields. Future development has therefore not been considered as a major factor that needs to be taken into account in the design. The majority of the urban





Figure 3.2: Vegetation Present within Wetland

area of Highfields have large allotment sizes of approximately  $3000\text{m}^2$ , with more recent subdivisions approximately  $1000\text{m}^2$  (Gray 2006). The larger allotment sizes have an “Urban Residential B” development classification while the smaller allotments are classified as “Urban Residential A” in accordance with the *Regional Standards Manual* (DDROC 2000). All rural residential allotments were given a “Rural Residential B” classification. The summary of “Deemed-to-Comply” criteria has been included as Figures C.1 to C.3 in Appendix C. These table provides coefficient of runoff and fraction impervious values for each development category (Figure C.1) and rainfall intensities for standard frequencies and durations (Figures C.2-C.3). The relevant values were used in the design calculations.

### 3.3 The Rational Method

The Rational Method is a widely used procedure for calculating the rate of surface runoff from a storm event. This method works on the basis that the peak discharge occurs when the time of concentration of the catchment is equal to the duration of the

storm. Equation 3.1 is used to calculate the maximum rate of discharge.

$$Q = FCAI \quad (3.1)$$

where,

**Q** = the designed flow rate,  $m^3/s$

**F** = factor of proportionality

**C** = dimensionless runoff coefficient

**A** = catchment area,  $ha$  or  $km^2$

**I** = rainfall intensity,  $mm/h$

The method works on the basis that the same amount of infiltration occurs throughout the storm. Initial losses are therefore not taken into account. The runoff coefficients used in this model are in accordance with the *Regional Standards Manual* (DDROC 2000). The rainfall intensity was calculated in accordance with *Australian Rainfall & Runoff* (Engineers Australia 1998). The catchment is broken up into a number of sub-areas and the longest flow path for each sub-area is used for the calculations. Both the sub-areas and flow paths for these areas are shown in Figure C.4 in Appendix C. This figure does not show the allotments or contours for the whole catchment. The remaining sections of sub-areas 2 and 6 were interpreted from other sources and added to this image.

### 3.3.1 Design Rainfall

The rainfall intensities for all standard ARIs and durations were taken from the *Regional Standards Manual* in Appendix C and plotted to form an IFD chart. Error in the data was observed for the 10 year ARI, 48h and 72h storms where the intensities for these storms were larger than for a 20 year storm. This is a phenomenon that cannot possibly occur. It is assumed that all other rainfall intensities have been recorded correctly. Due to the size of the catchment, storms of such long duration would not cause the peak

discharge and therefore these errors did not affect the results of the modelling. The IFD chart for Highfields is included as Figure C.5 in Appendix C.

It was decided to test ARIs of 2, 10 and 100 years. These Average Recurrence Intervals give a good range of design flows whilst providing a large enough difference between values to make it worthwhile testing each ARI. The intensities for these ARIs are then used to find the time of concentration for each section of the catchment.

### 3.3.2 Time of Concentration Calculations

The catchment area was broken up into six different sub-areas, as shown in Figure C.4 in Appendix C. Each sub-area had its longest flow path determined in order to make the time of concentration calculations. It should be noted that the contour plan provides insufficient data for the upstream end of the catchment. The area, slope of terrain and length of longest flow path were derived from alternative sources such as digital elevation models and aerial photography.

The time of concentration was calculated using the *Kinematic Wave Equation*. This equation is provided as Equation 3.2.

$$t = \frac{6.94 (Ln^*)^{0.6}}{I^{0.4} S^{0.3}} \quad (3.2)$$

where,

**t** = Travel time, *mins*

**L** = Length of flow path, *m*

**n\*** = Surface roughness coefficient

**I** = Rainfall intensity, *mm/h*

**S** = Slope of surface, *m/m*

As the time and intensity are both interrelated and therefore unknown, the equation is solved for  $tI^{0.4}$ . The design rainfall intensity values are also adjusted in order to be

compatible with the calculated values. For non-standard rainfall durations, Equations 3.3 to 3.5 are used to bring the result to within an accuracy of one minute. The data is then interpolated to find the total overland flow time for each sub-area.

$$P_D = \log_{10}(D) + 0.103(\log_{10}(D))^2 - 0.0710(\log_{10}(D))^3 + 0.0108(\log_{10}(D))e^5 \quad (3.3)$$

$$N = \frac{P_D - P_L}{P_U - P_L} \quad (3.4)$$

$$I_D = I_L \left( \frac{I_U}{I_L} \right)^N \quad (3.5)$$

A more detailed explanation of these equations are provided in *Australian Rainfall and Runoff* (Engineers Australia 1998). The calculations for finding the time of concentration using Equations 3.2 to 3.5 are included in Appendix C.

The slope for each reach was approximated from the contour plan provided by Crows Nest Shire Council. Reach lengths were also measured from this plan. The surface roughness coefficient ( $n^*$ ) in Equation 3.2 is approximated for each length. Recommended values are provided in *Australian Rainfall and Runoff* (Engineers Australia 1998) and Argue (1986). Values adopted were in the range of 0.15 for “Short Grass Paddocks” in areas of little development, and 0.05 for the more impervious areas. This parameter is very sensitive to change and therefore significantly affects the concentration time calculations. Previous knowledge or experimentation is often required to obtain accurate values for  $n^*$ . As this is not in the scope of this project, assumed values based on observed land type were adopted. A summary of the overland flow times calculated for each sub-area for the 2, 10 and 100 year design storms is shown in Table 3.1. As predicted, the area with the longest flow path produced the longest time of concentration. These values are shown in **bold** text in the table. The results of these calculations produced a reasonably good comparison with the results obtained from the RORB model (see Chapter 4).

The time of concentration calculations are based on the fact that there are no barriers resisting the flow off the land surface. A certain degree of error would then be expected in the time of concentration of Areas 2 and 6. The recreation grounds located on Kuhls Road act as a detention basin and therefore lengthening the time of concentration. The

Table 3.1: Summary of Flow Times for each Sub-Area and ARI

Sub-Area	Flow Time (mins)		
	2yr Storm	10yr Storm	100yr Storm
1	165.88	149.20	118.57
2	215.16	181.22	148.97
3	215.69	178.75	134.42
4	136.67	117.62	97.64
5	155.15	133.54	111.03
6	<b>232.10</b>	<b>195.85</b>	<b>161.39</b>

RORB model is able to take this detention basin into account, but the rational method calculations do not consider this basin. The times of concentration for each sub-area are then used to find the peak discharge for the catchment.

### 3.3.3 Peak Discharge Calculations

The peak discharge was calculated with the time of concentration equal to the duration of the storm, as in accordance with the Rational Method. Equation 3.1 was used to find the discharge based on the full area of the catchment. The time of concentrations shown in Table 3.1 relate to a full area calculation. However, large impervious areas and different slopes in the catchment can sometimes lead to a time shorter than the time of concentration causing the greatest runoff. This phenomenon is known as the *Partial Area Effect*.

There is the potential for a great deal of error when performing partial area calculations. The time of concentration for partial area is unknown and must therefore be assumed. Without significant knowledge of the catchment, the value adopted is entirely an assumption. As no data is available to assist in determining the time of concentration for partial area in the Klein Creek catchment, values have been assumed for modelling purposes.

Peak discharge calculations were performed for each ARI for each section of catchment. These results are included in Appendix C. The full area calculation uses the time of

concentration as shown in Table 3.1 rounded to the nearest minute. A design rainfall intensity for this time is adopted for the purpose of these calculations. Each sub-area is split into sections according to the land use type. The runoff coefficient ( $C$ ) and fraction impervious ( $f$ ) are applied to each section accordingly. Equation 3.1 is applied to the area and a peak flowrate obtained.

The partial area calculations work in the same manner. The time of concentration is first assumed, as well as the percentage of pervious area also contributing within this time period. This assumed time is the time taken for all the impervious to contribute to the flow as well as part of the pervious area (50% in this case). The time will always be less than the time for the full area calculations. The catchment area is broken up into pervious and impervious area. A fraction impervious value of 1 is applied to the impervious area and a value of 0 for the pervious area. The runoff coefficients are also adjusted for these values. The flow is then calculated for each area. For each sub-area, the largest flow is adopted as the design flow. Catchments that have significant impervious areas tend to be controlled by the partial area. This was the case with most of the sub-areas in this catchment. Table 3.2 provides a summary of the design flows calculated for each sub-area. In each case, it is indicated whether the partial (p) or full (f) area calculations were adopted as the design flow.

Table 3.2: Summary of Design Flows for each Sub-Area and ARI

Sub-Area	Discharge ( $m^3/s$ )		
	2yr Storm	10yr Storm	100yr Storm
1	6.650 (p)	11.063 (p)	19.802 (p)
2	7.061 (p)	11.989 (p)	21.670 (p)
3	0.681 (p)	1.175 (f)	3.356 (f)
4	0.772 (p)	1.302 (p)	2.413 (p)
5	2.070 (p)	3.504 (f)	6.699 (p)
6	3.438 (p)	5.791 (p)	10.792 (p)
<b>Total</b>	20.672	34.824	64.732

In some sub-areas the partial and full area calculations produced almost the same discharge. Other sub-areas recorded substantial differences in the results. In both



cases the accuracy of the assumption for the time of concentration in the partial area calculations must be questioned. The RORB model in the following chapter was used to provide a comparison of the results to determine the accuracy of the assumptions.

### 3.4 Chapter Summary

This chapter first outlined the features of the Klein Creek Catchment contributing to the flow into the Wirraglen Scout Reserve Wetlands. The location of the wetlands was identified along with the geography and land uses within the catchment. This data is available in Appendix B. The identification of existing flora and fauna was also covered within this section.

The use of a design storm in calculating peak inflow into the wetlands was then discussed including a description of the Rational Method and how it applies to this project. The catchment characteristics have also been examined to help explain the features of the catchment and how these features affect the design calculations. The data available for the catchment is by no means comprehensive. This means that quite a number of assumptions have had to be made, which subsequently increases the likelihood of errors in the calculations. The calculations also did not take into consideration the detention basin on Kuhls Road, which acts to increase the detention time of that area of the catchment. Appendix C includes all the calculations and data used in the design calculations.

## Chapter 4

# RORB Model

### 4.1 Chapter Overview

RORB is a computer runoff routing model used to estimate hydrographs on a catchment. It is preferable to have known storm rainfall and runoff measurements that can be used for calibration and validation of the model. However, if these are not known, the parameters required for the model can be estimated from different theories, such as the Weeks Estimate for Queensland catchments. As there is no recorded measurements for the Klein Creek catchment, these approximations will be applied to the model.

Data was obtained on the detention basins in the catchment. These were incorporated into the model to determine the appropriateness of their design. An interactive weir design was used in the model to adjust the outlet structure from the wetlands to cope with the peak discharge from a design storm. Finally, the results from the RORB model were compared with those calculated from the Rational Method to provide an indication of the accurateness of the two models.

## 4.2 Overview of RORB

RORB is a routing model used to estimate hydrographs on a catchment. The program can be used for both runoff routing as well as stream and reservoir routing. In order to estimate the runoff from a catchment, known rainfall hyetographs are required. The user must separate the catchment into sub-areas and an estimated total volume of rainfall is assigned to each sub-area from the known rainfall data. The nearest rain gauging station is also assigned to each sub-area and this is used to apply the rainfall pattern for the storm event. It is assumed that all rain entering each sub-area does so at the centroid of each sub-area. If no rainfall data is available, then a design storm rainfall event can be used, assuming an equal distribution of rainfall across the catchment.

The model converts the rainfall into rainfall excess using either the initial-continuing (IL-CL) or initial-proportional (IL-PL) loss models. User defined data is then used to route the rainfall excess along a flow path to a stream junction. The hydrograph is stored at this point while another sub-area is analysed before this other hydrograph is also routed to the same point. The two hydrographs are then added together and the process continues downstream until the outlet is reached. The RORB model has the ability to be able to plot the hydrograph at any point in the catchment, such as before and after storages. This makes the model useful for determining the size of weir required to reduce the peak discharge back to near-natural conditions after urban development has occurred.

In order for the model to calculate the hydrographs for the catchment, a number of parameters are required. The two main parameters are the coefficient  $k_c$  and exponent  $m$ , both of which are dimensionless. Pilgrim (1997) defines the exponent  $m$  as “a measure of the catchment’s nonlinearity, and the same value is used for all reach storages in the catchment.” This value is generally left as 0.8 but may range from 0.6 to 1.0. The coefficient  $k_c$  is an empirical coefficient relevant to the entire catchment. This coefficient can change substantially between catchments. The RORB program is able to provide estimates of this parameter based on a number of different theories depending on the location of the catchment. For Queensland, Engineers Australia (1998) recommends using the “Week’s Method” to obtain  $k_c$  if calibration is not possible.

### 4.3 Application of RORB

The first stage in the RORB modelling is to subdivide the catchment so that it can be more accurately modelled. Contour plans provided by Crows Nest Shire Council were used to assist with this process. The subdivisions should be aligned normal to the contours. Major tributaries should also be given their own sub-area. Figure D.1 in Appendix D provides a copy of the catchment separated into 10 sub-areas. This plan was used to determine the area and reach length of each sub-area.

The catchment data for each sub-area is entered into a separate '\*.cat' file which is then loaded by RORB. A catchment file for natural conditions with no development, and a catchment file for the current conditions were created. The natural conditions file was used to determine the critical storm, as well as to provide a comparison for peak runoff. This will help assess how well the wetlands are able to cope with an increase in flow due to development. Figures D.2 and D.3 in Appendix D provide the code used for the catchment data for natural and current conditions respectively. Chapters 4.2 and 5 in Laurenson, Mein & Nathan (2006) explain the meaning of the code used in these files.

Often known storm data is also used in the RORB simulation. Rainfall and runoff data would be included in the storm files. However, for the Klein Creek catchment, no known data exists. Design storms have therefore been used in the simulation. RORB is capable of calculating these design storms from IFD data entered by the user. The method recommended in Engineers Australia (1998) is used for these calculations.

The Week's Method was used to determine the ' $k_c$ ' parameter. Equation 4.1 was used for the calculations. The parameter 'm' was left as 0.8 for the purpose of this simulation.

$$k_c = 0.88A^{0.53} \quad (4.1)$$

From this equation, and using an area of 6.765 produced a value of  $k_c$  of 2.42. An Initial-Continuing (IL-CL) model was used, using the recommended values of IL = 15mm and CL = 2.5mm/h.

The simulation of this model has also included the detention basin on Kuhls Road. The location of this detention basin is shown in Figure D.1. This detention basin covers a

large proportion of the Highfields area of the catchment. Very little data was known by Crows Nest Shire Council for this detention basin except it has a 600mm nominal diameter outflow pipe and doubles as the Football Grounds. The detention basin is shown in Figure 4.1.



Figure 4.1: The Football Grounds on Kuhls Rd used as a Detention Basin

The outlet for the wetland is Meringandan Road. This is shown in Figure 4.2. Once again, Crows Nest Shire Council does not have a large amount of information on the outlet structure. The pipe for the outlet has a nominal diameter of 1825mm and is 12m long. No other data is known and so this has been approximated for use in RORB. The assumed parameters include pipe and weir coefficients and storage-elevation parameters. The values adopted are shown in Table 4.1.

Table 4.1: Assumed Parameters for Wetland Outlet

Parameter	Adopted Value
Weir Coefficient	1.9
Pipe Entrance Loss Coeff.	0.5
Pipe Bend Loss Coeff.	0
Storage Coefficient 'a'	2840
Storage Exponent 'b'	2.7
Zero Storage Elevation	50m





Figure 4.2: The Outlet Structure at the Wetlands

It was found that the parameters shown in Table 4.1 only had minimal effect on the actual outflow from the wetlands. These values were therefore not considered critical for this particular modelling procedure.

An interactive approach can be used in RORB for the design of the weir. The parameters mentioned previously can be altered at this point in the model. Pipe and weir dimensions can also be changed to find an improved design. This particular approach was used in this project to find an improved design if necessary.

The model needed to be run three separate times in order to consider the 2, 10 and 100 year design storms. The critical storm duration was determined for each ARI and then the adequacy of the wetland and detention basin were analysed using these design storms. The program outputs runoff hydrographs for each simulation. Both the inflow and outflow for the wetland and detention basin can be modelled.

## 4.4 Results

The results from RORB can be split into three separate categories: critical storm duration, natural condition flows and current condition flows. Each of these will be discussed in turn.

### 4.4.1 Critical Storm Duration

A sensitivity analysis was conducted to determine the critical storm duration. This was performed for the 2, 10 and 100 year ARI storms. The storm duration that produces the greatest peak discharge is considered to be the critical storm duration. The peak discharge is considered to be more important than the total volume of flow during a storm event as this is the time when the flow is most likely to exceed the wetland or detention basin's capacity.

The storm duration will have an impact on the intensity of the storm and therefore the amount of runoff that will occur. A shorter storm duration will always have a higher rainfall intensity for a particular frequency. This phenomenon can be seen from the IFD curve shown in Figure C.5 in Appendix C. Although a shorter storm produces a higher intensity rainfall, the storm may be too short for the entire catchment to contribute to the flow out of the catchment all at the same time. The furthest upstream point of the catchment normally needs to be contributing to the overall flow for the highest discharge to be achieved.

The results of the critical storm duration runs are provided in Appendix D, Figures D.4 to D.6. These results are in the form of a calculated hydrograph for durations ranging from 1 hour to 24 hours and assuming the catchment is in a natural state without any development. A summary of the results are provided in Table 4.2. From this table it can be seen that a 3hr duration storm produced the peak discharge for all three storm frequencies. These values have been highlighted in the table. It should be noted that the peak values should not be compared with those calculated from the Rational Method as they are not taking into consideration any development within the catchment.

Table 4.2: Results of Critical Storm Duration Run

Duration	2yr ARI		10yr ARI		100yr ARI	
	Rain (mm)	Peak (m <sup>3</sup> /s)	Rain (mm)	Peak (m <sup>3</sup> /s)	Rain (mm)	Peak (m <sup>3</sup> /s)
1hr	36.44	15.3	48.18	26.7	69.50	49.6
1.5hr	41.35	16.8	55.06	28.4	79.94	51.3
2hr	45.04	17.3	60.28	28.9	87.94	51.5
3hr	50.65	17.8	68.29	29.4	100.28	52.6
4.5hr	56.88	17.1	77.25	28.7	114.21	49.7
6hr	61.78	16.4	84.33	28.3	125.28	48.3
9hr	69.46	13.8	95.52	24.7	142.84	42.5
12hr	75.51	15.5	104.38	26.9	156.84	45.8
18hr	86.73	11.0	122.74	19.6	188.41	30.9
24hr	95.44	13.0	137.37	21.3	214.15	34.9

#### 4.4.2 Natural Condition Flows

The simulations for natural conditions are all based on the 3 hour critical storm duration determined from the sensitivity analysis. The catchment file created for this run is provided as Figure D.2 in Appendix D. Due to the fact that there is no development in the catchment, this catchment file does not include the detention basin on Kuhls Road. The results of this analysis will be used to test the outflow of the wetlands under current conditions. If the wetland is capable of reducing the outflow down to the natural condition flow, then the capacity of the wetland to detain the required discharge is sufficient. No change to the current dimensions is therefore required.

RORB once again needed to be run three times to cater for the three different frequency storms. The rainfall hyetographs and runoff hydrographs are provided as Figures D.7 to D.9 in Appendix D. The results show the peak outflow occurring at approximately 1.75 hours for the 2 and 10 year storms, and 1.5 hours for the 100 year storm. The results are summarised in Table 4.3. The rainfall hyetographs simulated in RORB show the total rainfall for each time step. The time step varies depending on the length of the storm. In accordance with Engineers Australia (1998), a 3 year design storm has 12



Table 4.3: Summary of Natural Condition Simulation

ARI	Time to Peak (h)	Peak Flow ( $\text{m}^3/\text{s}$ )
2yr	1.75	17.79
10yr	1.75	29.37
100yr	1.5	52.59

time steps each of 0.25 hours duration. The unshaded area on the hyetographs indicate the losses that occurred for each time step. This includes the initial loss of 15mm and the continuing loss of 2.5mm/h. It is assumed that the difference in time between the peak rainfall and peak runoff occurring is the time taken before the most upstream point of the catchment begins to contribute.

#### 4.4.3 Current Condition Flows

The current condition simulation was also based on the critical 3 hour storm duration. The catchment was altered to include the urban development, detention basin and wetland properties. The degree of permeability was the main variable changed to incorporate the urban development. For the wetland structure, an interactive weir design was used. The initial parameters from Table 4.1 are included in the catchment file. These values can later be changed within the RORB model. A copy of the catchment file is included as Figure D.3 in Appendix D.

The RORB model showed an increase in the peak discharge as a result of the urban development. The time taken to reach this peak was also reduced. This results in a greater strain placed upon Klein Creek and the wetland itself.

#### Detention Basin

The simulations also took into consideration the detention basin on Kuhls Road. The hydrographs for each ARI for the detention basin are provided as Figures D.10 to D.12 in Appendix D.

Crows Nest Shire Council was unsure on what ARI the detention basin was designed to handle. The results from the RORB model suggest that the design was for a 2 year

ARI. This is shown by the fact that the detention basin is very successful in reducing the peak discharge for a 2 year storm event. However this efficiency reduces as the frequency of storm event increases. A 100 year storm event is almost unaffected by the detention basin. A summary of the results for the detention basin is provided in Table 4.4. These results show reductions of 19%, 8% and 4% for the 2, 10 and 100 year design

Table 4.4: Summary of Results for Detention Basin

ARI	Peak Inflow		Peak Outflow	
	Time (h)	Flowrate (m <sup>3</sup> /s)	Time (h)	Flowrate (m <sup>3</sup> /s)
2yr	1.00	11.66	1.25	9.42
10yr	1.00	17.23	1.00	15.86
100yr	1.00	26.49	1.00	25.51

storms respectively. The detention basin is therefore much less successful in handling the larger 100 year flows. The time taken to reach the peak is also not reduced for 10 and 100 year storms. The outflow values from the detention basin are used by RORB to continue the downstream routing procedure.

### Wetlands

The wetland catchment area is much larger than that for the detention basin. As a result, the detention basin had only a small effect on reducing the peak discharge into the wetlands. The hydrographs for each ARI for the wetland are provided as Figures D.13 to D.15 in Appendix D.

The results of the model suggest that the wetland is capable of handling all three average recurrence intervals. A noticeable reduction in peak flow occurred for all three storm events. A summary of the results for the wetlands is provided in Table 4.5. The

Table 4.5: Summary of Results for Wetland

ARI	Peak Inflow		Peak Outflow	
	Time (h)	Flowrate (m <sup>3</sup> /s)	Time (h)	Flowrate (m <sup>3</sup> /s)
2yr	1.75	18.66	2.50	16.85
10yr	1.50	29.95	2.25	27.85
100yr	1.50	52.75	2.00	49.70

results in this table show a 10%, 7% and 5% reduction in flow for the 2, 10 and 100 year design storms respectively. This shows the wetland is successful in handling the design flows for all three design storms. A comparison with the natural flow results in Table 4.3 shows that the outflow from the wetlands with development is actually lower than the natural flow through Klein Creek before development. The wetlands are therefore successful in detaining the flow in Klein Creek to a point where excessive erosion is highly unlikely.

### Comparison with Rational Method

A comparison between the RORB model and Rational Method was conducted to determine the accuracy of the two models. This was performed because no calibration or validation was possible on the catchment. Table 4.6 compares the two methods and highlights any differences in the results. These results shows that the accuracy

Table 4.6: Comparison in Peak Inflow between RORB and Rational Method

ARI	Rational	RORB	Difference
2yr	20.67	18.66	2.01
10yr	34.82	29.95	4.87
100yr	63.73	52.75	10.98

decreases as the ARI increases. This difference will be a result of errors in the estimation of certain values particularly in the Rational Method. Due to the fact that most of the sub-areas produced a higher peak discharge for the partial area affect rather than for full area, the times of concentration needed to be assumed. It is likely that these times have been incorrectly estimated. The surface roughness coefficient,  $n^*$  also needed to be assumed. A small change in this value results in a large change in the time of concentration which could be affecting the final discharge calculation.

The RORB model has smaller room for error. Experience with the model plays a large role in the accuracy of the initial steps in the modelling procedure, in particular the subdivision of the catchment into sub-areas. Reliance must also be placed on the accuracy of the model itself, although the model has proven to be able to accurately model other catchments. The fact that this model has not been able to be calibrated or validated also leaves room for error.

## 4.5 Recommendation

The results of the RORB model has shown that the Wirraglen Scout Reserve Wetlands are capable of providing a noticeable reduction in flow during a design storm event. The outflow from the wetlands has resulted in a reduction in flow to below pre-development flow levels. The wetlands are therefore capable of handling the required discharges and no modification to the wetlands is required at this point in time to improve the detention capacity.

If the catchment area is developed significantly in the future, the wetlands capacity to handle the required flow may reduce. This will be due to either a significant increase in flow off the catchment area, or further sedimentation within the wetlands. If this occurs, then the wetland structure will need to be modified. Increasing the depth of the wetland would be the most efficient option, either through removing the sediment in the wetland, or by increasing the height of the weir at the outlet. A change in height of 0.5m for a 2 year design storm in RORB has indicated an in excess of  $2m^3/s$  decrease in the peak outflow. This scale of alteration would be sufficient for the size of the catchment and its potential for future development.

## 4.6 Chapter Summary

This chapter first discussed the RORB model including a description of the model and the parameters and data that need to be included to run the simulation. The model provides an approximation of the actual flow that would occur within a catchment during a particular storm event. If possible, the model is first calibrated using known rainfall and streamflow data. In this case no data was available and therefore design storms only were used. The Week's Method was used to find the parameter  $k_c$  which would usually be determined during the calibration procedure.

The RORB model was run for both pre-development and current catchment conditions. The pre-development run was used to find the critical storm duration using a sensitivity analysis. A duration of 3 hours was found for the 2, 10 and 100 year storm events.

The current catchment run was then used to determine the current contribution of the wetland in reducing the peak discharge along Klein Creek. During this run, the detention basin on Kuhls Road was also considered.

It was found that the detention basin was effective in reducing the peak flow for the 2 year design storm. Its effectiveness decreased as the intensity of the storm increased. The detention basin almost had no effect in reducing the peak discharge for a 100 year storm event. The wetland was much more capable of handling the design flows for all average recurrence intervals. A noticeable decrease was observed with the effectiveness only decreasing slightly with an increase in peak discharge. A comparison with the pre-development flows in Klein Creek showed that the wetland was able to reduce the peak outflow to below pre-development levels. This means that it was able to negate the effect of development on the peak discharge within the catchment area. It was therefore determined that the wetlands were sufficient in handling the design flow.

A comparison with the Rational Method showed some differences between the calculated values. The discharges were within the same range of values, although it was determined that there was some error in the calculations, most likely in the Rational Method. A recommendation was still able to be developed from the results. It was shown that the wetlands are currently able to handle the design flows. If there is future development in the catchment area or there is an increase in sedimentation in the wetland, then increasing the height of the outlet weir would be the most effective means of improving the wetlands capability of handling the peak discharge.

## Chapter 5

# Water Quality Analysis

### 5.1 Chapter Overview

Water samples were obtained from two different locations in the Wirraglen Scout Reserve Wetlands. The wetlands are very long and narrow and therefore it was decided to determine the improvement in water quality as the runoff passed through the wetlands. The Project Specification (Appendix A) also includes taking samples upstream and downstream of the wetlands. For the duration of the project, no inflow or outflow has been recorded at the wetland site. These samples would be able to give a better indication of the role of the wetlands in improving urban stormwater quality. To overcome this problem to some degree, the MUSIC software has been used to model the water quality improvement within the wetlands.

A number of different tests were performed on the two samples. These included Biochemical Oxygen Demand (BOD), Chemical Oxygen Demand (COD), Total Solids (TS) and Total Suspended Solids (TSS). Due to the lack of flow through the wetlands, it was not expected that there would be a significant difference in results between the two samples.

## 5.2 Water Sampling Locations

Two locations within the wetland were chosen to obtain water samples. The downstream location was located approximately 20 metres upstream of the outlet. The location of these sampling sites are shown in Figure 5.1. Access was not possible at any location closer to the outlet. The second test was located at the upstream end near the cessation of water storage. Both of these locations will be analysed in turn including the results obtained from the tests.



Figure 5.1: Location of Samples Taken

*Source: Crows Nest Shire Council*

## 5.3 Upstream Sample

The upstream end of the wetland is very dry with almost no water left at all. The sample was taken in a very narrow section of remaining water. The location is shown in Figure 5.2. A sample was easily obtained at this location due to the narrow, shallow section of water. However, because the water was so shallow, it was difficult to take a



sufficient sized sample without disturbing the soil. Some errors may therefore result in the sample due to this disturbance, in particular with TS and TSS. The width of water would be much wider after a storm event and it is assumed that this width decreases fairly quickly after the cessation of flow into the wetland due to the geography of the location. This gives the wetland a good capacity to expand in size during a storm event which assists in its ability to treat the required volume of water.



Figure 5.2: Location of Upstream Water Sample

The water sample initially looked very clear with very little in the way of suspended material. After the sample had been allowed to settle for a period of time, a very small amount of soil matter was visible on the bottom of the container. The initial reaction to this sample was that there appeared to be very little carbonaceous matter present and therefore the oxygen demand would be quite low. This is unsurprising considering the substantial period of time the water would have been stored within the wetland.



### 5.3.1 TS and TSS Test Results

Total Solids and Total Suspended Solids provide an indication of the amount of particulate matter present. The results are based on a milligram weight of solid per litre sample. Equation 5.1 is used to calculate weight of Total Solids per litre sample.

$$TS = \frac{(A - B) \times 1000}{V} \quad (5.1)$$

where,

**TS** = total solids, *mg/L*

**A** = weight of dried residue + dish, *mg*

**B** = weight of dish, *mg*

**V** = sample volume, *mL*

Total Solids is a measure of the total amount of particular matter present in the sample. This includes both suspended and dissolved particles. An aluminium dish was used for the tests. The dish was first weighed and a 10mL sample was placed in the dish and allowed to dry at approximately 105°C. The dish was then weighed again and Equation 5.1 used to obtain the results shown in Table 5.1. The results of the first two samples

Table 5.1: Total Solids Test Results for Upstream Sample

<b>TS</b>	Sample		
	1	2	3
A (mg)	930.8	938.1	915.9
B (mg)	929.4	937.1	920.0
Volume (mL)	10	10	10
TS (mg/L)	140	100	-
<b>Average (mg/L)</b>	<b>120</b>		

are fairly consistent, producing an average TS at the upstream end of the wetlands of 120mg/L. The third sample produced an error whereby the weight after drying was less than the initial weight of the empty dish. There was most likely an error in weighing

this sample due to the variance in values with the other two samples. This sample was therefore not considered when calculating the average Total Solids concentration.

Total Suspended Solids is calculated using the same formula, with a filter being used in place of a dish. The TSS is a measure of the total suspended matter in the water sample. TSS is found by forcing the water sample through a filter using a vacuum. The sample is then dried at approximately 105°C for one hour. The sample is once again weighed and Equation 5.1 used to find the weight of TSS per litre sample. The results of the Total Suspended Solids test for the upstream sample are provided in Table 5.2.

Table 5.2: Total Suspended Solids Test Results for Upstream Sample

TSS	Sample					
	1	2	3	4	5	6
A (mg)	107.8	106.0	107.4	105.5	105.7	106.8
B (mg)	108.2	106.1	107.5	105.4	105.6	106.4
Volume (mL)	10	10	10	10	10	10
TSS (mg/L)	-	-	-	10	10	40
<b>Average (mg/L)</b>	<b>20</b>					

The results of these tests show an error in the first three samples tested. The results show the filter paper alone weighed more than the filter paper with the suspended solids. This result can obviously not occur and as a result these values were omitted from the average TSS concentration. It is assumed that this error occurred due to the incorrect weighing of the filter paper or the sample and filter paper. The remaining results were fairly consistent with one another with sample 6 being slightly higher than sample 4 or 5. This 30mg/L difference was not considered to be as a result of any errors in measurement. The average Total Suspended Solid concentration for the upstream sample was measured at 20mg/L. TSS concentrations for urban residential areas are provided by GCCC (2006). This document shows that TSS can range from 20mg/L for roofs to 270mg/L for roads for the Southeast Queensland region. This indicates that the measured results are slightly lower than would be expected for the catchment. It is likely that during a storm event, the water flowing into the wetland would be higher than the measured value of 20mg/L. The length of time the water has been in the

wetland may be the cause of the lower value.

### 5.3.2 COD Test Results

The Chemical Oxygen Demand (COD) is a measure of the oxygen consumption exerted by matter in the water. COD tests are conducted by placing a predetermined volume of sample, digestion solution and sulfuric acid reagent in a digestion vessel. The quantities used for this test are shown in Table 5.3. The mixture is then sealed and refluxed at

Table 5.3: Sample and Reagent Quantities used in COD Test

Sample (mL)	Digestion Solution (mL)	Sulfuric Acid Reagent (mL)	Total Final Volume (mL)
2.0	1.2	2.8	6.0

150°C for two hours. Three upstream samples and two blank samples were used for the COD test. The blank samples use distilled water and as a result no oxygen demand should be exerted on these samples. This produces a point of reference for the wetland samples to determine the oxygen demand. Once the samples have been refluxed, they are then titrated using a standard Ferrous Ammonium Sulfate (FAS) solution. These recorded values are then used in Equation 5.2 to obtain the oxygen demand exerted during the test.

$$COD = \frac{(A - B) \times M \times 8000}{V} \quad (5.2)$$

where,

**COD** = Chemical Oxygen Demand exerted on the sample,  $mgO_2/L$

**A** = Volume of FAS used for the Blank,  $mg$

**B** = Volume of FAS used for the sample,  $mg$

**M** = Molarity of FAS = 0.12445

**V** = Volume of Sample,  $mL$

The results of the tests are provided in Table 5.4. The two blank samples produced an average FAS volume of 1.225mL. This average volume was used for the COD calculations for each of the wetland samples.

Table 5.4: Chemical Oxygen Demand Test Results for Upstream Sample

COD	Sample		
	1	2	3
A (mg)	1.225	1.225	1.225
B (mg)	1.2	1.15	1.15
V (mL)	2	2	2
COD (mg/L)	12.445	37.335	37.335
<b>Average (mg/L)</b>	<b>29</b>		

The results show a very good correlation between each of the three samples. It also shows that the COD is very low for this sample location due to the fact that the FAS volumes used in the titration for the wetland samples are very similar to the blank samples. These results therefore support the initial observation of very little apparent biological activity within the water sample.

### 5.3.3 BOD Test Results

The Biochemical Oxygen Demand is another method used to calculate the oxygen demand exerted on a water body. Only the biodegradable organic matter is analysed in the test. The BOD test is a simple procedure where the dissolved oxygen (DO) is measured at the beginning and end of the test. The difference between these two measurements gives the oxygen demand exerted over the specified time. The standard length BOD test of 5 days was used for these tests. In some situations such as with wastewater, the oxygen demand may be so high that the water becomes depleted of oxygen and therefore results cannot be obtained. If this is the case, the sample should be diluted prior to the test. In the case on these samples, it was expected that the BOD would be quite low and therefore the samples were not diluted for the test. Once the two DO values have been obtained, the BOD can be calculated from Equation 5.3.

$$BOD_5 = \frac{(D_1 - D_2)}{P} \quad (5.3)$$

where,

$BOD_5$  = Biochemical Oxygen Demand exerted on the sample over a 5 day period,  
 $mg/L$

$D_1$  = Initial Dissolved Oxygen (DO) of sample,  $mg/L$

$D_2$  = DO of sample after 5 days incubation,  $mg/L$

$P$  = Decimal volumetric fraction of sample used

The results of the BOD test for the upstream sample are provided in Table 5.5. The fridge that the samples were stored in for 5 days was set at 20°C, however the samples came out of storage at approximately room temperature. It is unknown why this occurred. This could have an affect on the final results of the test, although the fact that the water has been in the wetlands for a considerable period of time means that this may have only caused minimal error.

Table 5.5: BOD Test Results for Upstream Sample

<b>BOD<sub>5</sub></b>	Sample	Temp (°C)
$D_1$ (mg/L)	7.9	23.1
$D_2$ (mg/L)	5.9	25.8
P	1	
$BOD_5$ (mg/L)	2	

Only one sample was analysed for the upstream location. The test produced a 5 day BOD value of 2mg/L. This value is quite low which is to be expected for a largely residential catchment. Wikipedia (2006) states that most pristine rivers have a 5 day BOD of less than 1mg/L, with moderately polluted rivers between 2mg/L and 8mg/L. The test results therefore indicate a slight polluting of the waterway caused by biodegradable organic matter. This level of pollution is not a major concern for Klein Creek.

The results from the four upstream tests are compared with each other and the downstream sample tests later in this chapter.

## 5.4 Downstream Sample

The downstream end of the wetland is a relatively wide section of water with dense reeds and grasses growing in it. This made it very difficult to find a location to take the sample. The sample could not be taken near the waters edge due to the possibility of an inaccurate sample. A horizontally growing tree located 20m upstream of the outlet provided an ideal location, as shown in Figure 5.3. The sample was able to be taken near the centre of the wetland in an area free of the dense vegetation.



Figure 5.3: Location of Downstream Water Sample

The water sample initially looked very clear with very little in the way of suspended material. After the sample had been allowed to settle for a period of time, a very small amount of soil matter was visible on the bottom of the container. There was also a small amount of algal growth floating on the surface of the water. The initial reaction to this sample was that there appeared to be very little carbonaceous matter present and therefore the oxygen demand would be quite low. This is unsurprising considering the substantial period of time the water would have been stored within the wetland. The algal presence may result in some variance with the TS and TSS results due to

the fact that some samples may include this algae while other may not.

#### 5.4.1 TS and TSS Test Results

The Total Solids and Total Suspended Solids for the downstream sample were calculated from Equation 5.1. It was difficult to mix the algae in with the rest of the water before the TS and TSS samples were extracted. This resulted in some samples having algae present while others did not. Some variance was therefore observed with the results as a result of this algae. There was little other suspended matter present within the samples.

The results of the Total Solids test for the downstream sample are provided in Table 5.6. The large variance in results in this test is likely to be due to the presence of small algal particles floating on the surface of the water in the wetland.

Table 5.6: Total Solids Test Results for Downstream Sample

TS	Sample		
	1	2	3
A (mg)	935.0	934.8	929.3
B (mg)	927.8	934.5	921.3
Volume (mL)	10	10	10
TS (mg/L)	720	30	800
<b>Average (mg/L)</b>	<b>520</b>		

The results of the TS test for the downstream sample gave an average value of 520mg/L. There was a distinct variance between the three samples tested with results ranging from 30mg/L to 800mg/L. This difference is most likely due to the presence of the algae at this location. Sample 2 was therefore also included in the calculation of the average value.

As a result of the presence of the algae, it would be expected that TSS would also be higher than at the upstream location. The results of the Total Suspended Solids test for the downstream sample is provided in Table 5.7.



Table 5.7: Total Suspended Solids Test Results for Downstream Sample

TSS	Sample					
	1	2	3	4	5	6
A (mg)	106.0	106.9	107.7	106.7	107.6	107.5
B (mg)	106.0	107.3	107.7	106.7	107.4	107.3
Volume (mL)	10	10	10	10	10	10
TSS (mg/L)	0	-	0	0	20	20
<b>Average (mg/L)</b>	<b>20</b>					

A large amount of error was once again present in a number of these samples. One of the samples had a final weight less than the weight of the filter paper alone while three of the samples weighed the same after drying. This either indicates no suspended solids present or an error in the test procedure. In this case, solids were clearly visible on the filter paper which would have noticeably increased the final weight. As a result, samples 1-4 were discarded and not included in the final average.

#### 5.4.2 COD Test Results

The COD tests for the downstream were calculated using Equation 5.2. The results of the tests are provided in Table 5.8. The same blanks were used for this analysis as was used for the upstream samples.

Table 5.8: Chemical Oxygen Demand Test Results for Downstream Sample

COD	Sample		
	1	2	3
A (mg)	1.225	1.225	1.225
B (mg)	1.2	1.15	1.2
V (mL)	2	2	2
COD (mg/L)	12.445	37.335	12.445
<b>Average (mg/L)</b>	<b>20.75</b>		

The results show a very good correlation between each of the three samples. It also shows that the COD is very low for this sample location due to the fact that the FAS



volumes used in the titration for the wetland samples are very similar to the blank samples. These results therefore support the initial observation of very little apparent biological activity within the water sample.

### 5.4.3 BOD Test Results

The results of the BOD test for the downstream sample are provided in Table 5.9. These results were obtained using Equation 5.3.

Table 5.9: BOD Test Results for Downstream Sample

<b>BOD<sub>5</sub></b>	Sample	Temp (°C)
$D_1$ (mg/L)	7.4	23.4
$D_2$ (mg/L)	6.3	25.4
P	1	
$BOD_5$ (mg/L)	1.1	

There was a problem with the temperature control in the fridge that the samples were being stored and therefore the tests were conducted at approximately room temperature. This may have had an effect on the final results. The  $BOD_5$  value of 1.1mg/L tested indicates a very good quality at the downstream end of the wetland. According to Wikipedia (2006), this value places the water in a near-pristine category. There is therefore no problem with pollution in the water currently in the wetland.

## 5.5 Comparison of Sample Results

The results for both the upstream and downstream samples are compared in Table 5.10. There was no decrease in TSS and TS increased as the water flowed downstream. This increase is assumed to occur due to the algae present on the surface of the wetland at the downstream location. During periods of flow, this algae would most likely adhere to the surface of the reeds and therefore have no effect on the TS measurement. From these results it is impossible to tell how the wetland contributes to the reduction in TS and TSS during a storm event.

Table 5.10: Summary of Test Results

	Upstream	Downstream
TS (mg/L)	120	520
TSS (mg/L)	20	20
COD (mg/L)	29	20.75
BOD <sub>5</sub> (mg/L)	2	1.1

The difference between the TS and TSS concentrations is known as the Total Dissolved Solids. These are the solids that are not readily visible in a sample and are able to flow through the filter paper during the TSS test. The results of the TS and TSS tests show a TDS of 100mg/L for the upstream sample and 500mg/L for the downstream sample. It is possible that the algal presence at the downstream location may be affecting the results. A small amount of flow between the upstream and downstream locations may have also encouraged particulates to become dissolved between the two locations. Active flow through the wetlands would be required to determine the exact cause of this anomaly and whether or not the same results occurred during an actual storm event.

The table shows from the two samples that the oxygen demand in the water decreased as it flowed through the wetland. The wetlands are therefore effectively able to treat the stormwater that is currently in the wetland. However, these results do not indicate the oxygen demand for the inflow or outflow during a storm event. The results may be entirely different under these circumstances and further testing is required when there is flow in Klein Creek to determine the actual contribution of the wetlands in improving urban stormwater quality.

The results indicate a significant difference between the oxygen demand for the chemical and biochemical tests. The COD test will always produce a higher result than the BOD test because COD also decomposes some non-biodegradable matter that would otherwise remain after a BOD test. From these tests, the BOD/COD ratios of 0.07 and 0.05 for the upstream and downstream samples respectively are slightly lower than expected. This indicates a fairly high level of non-biodegradable organic matter in comparison to readily biodegradable organic matter.

## 5.6 MUSIC Water Quality Model

The MUSIC model is used to simulate the quality of runoff from a catchment. It is able to analyse a range of area types such as urban agriculture and forest and conveys the discharge from these areas to a treatment facility. These facilities include wetlands, ponds, sedimentation basins and gross pollutant traps. The program was developed by the Cooperative Research Centre for Catchment Hydrology. More information on the use of this model can be found in Wong, Coleman, Duncan, Fletcher, Jenkins, Siriwardena, Taylor & Wootton (2005).

The same catchment subsections were used as in Figure C.4 in Appendix C. Areas 1, 2 and 6 were assigned an urban zoning, while areas 3, 4 and 5 were given an agricultural zoning. MUSIC requires the input of pervious and impervious areas as a percentage of the total area. The percentages used are the same as for the design storm calculations in Appendix C. The other parameters required to be entered include:

- Rainfall-Runoff Parameters;
- Total Suspended Solids;
- Total Phosphorus; and
- Total Nitrogen.

These values must be inputted for each sub-area individually. The values used in the modelling were obtained from GCCC (2006). The values used in this document were derived by the Brisbane City Council and are considered to be the best available data for South-East Queensland (GCCC 2006).

The treatment nodes also required data inputted such as inlet, storage and output properties. Some of these values, such as depth of permanent water storage needed to be assumed in the modelling. The layout of the model used for the Wirraglen Scout Reserve Wetlands is shown in Figure 5.4. Along with the 6 sub-areas, the detention basin on Kuhls Road and the wetland itself are also depicted. The junctions are used to combine the flow coming off the relevant areas. The receiving node at the top of the

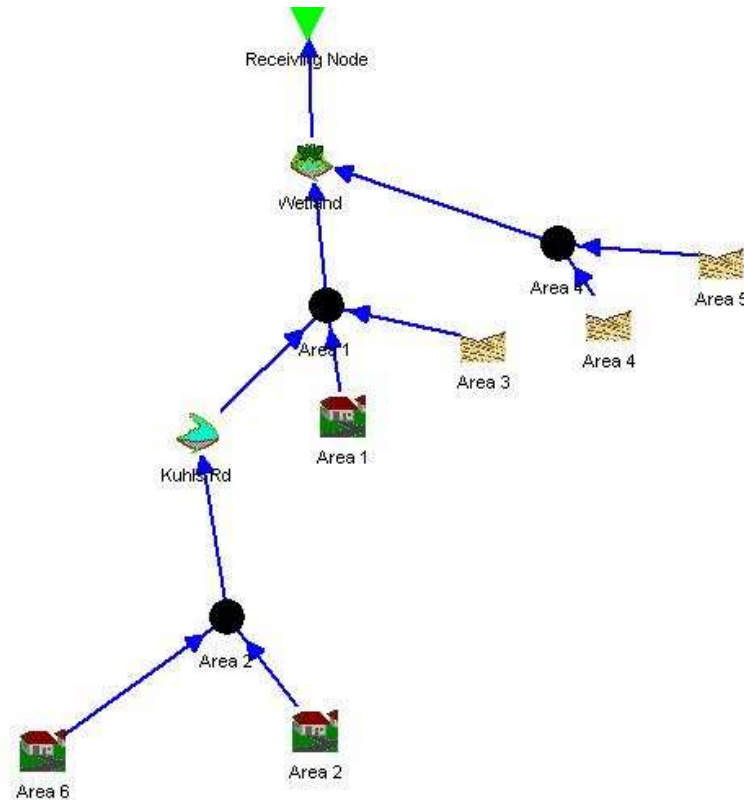


Figure 5.4: Diagram Used for MUSIC Model

image is required in order to model the quality of water flowing out of the wetlands. The model can also be used to simulate quantity of flow, however the model used for this project only considers water quality.

The model works on the basis of a concentration of pollutants and therefore the peak inflow is not required. Total Suspended Solids, Total Phosphorus and Total Nitrogen were all simulated. Phosphorus and Nitrogen were not tested from the water samples obtained from the wetland. There are therefore no physical measurements to test the accuracy of the results from this simulation. The results of the MUSIC model are summarised in Table 5.11. It can be seen from these results that the detention basin on Kuhls Road has no effect on improving the water quality. The wetlands however are effective in reducing TSS, TP and TN.

Comparing the results from the MUSIC simulation to those results obtained from the water samples, indicates a very good correlation between the TSS values. The test results showed a TSS concentration of 20mg/L which fall in between the inflow and

Table 5.11: Summary of Results for MUSIC Simulation

	Detention Basin		Wetlands	
	Inflow	Outflow	Inflow	Outflow
TSS (mg/L)	19.4	19.4	25.7	14.7
TP (mg/L)	0.0972	0.0972	0.0995	0.0409
TN (mg/L)	1.04	1.04	0.894	0.656

outflow values obtained with MUSIC. Tests should be conducted on a future runoff event to determine the TSS concentration of the inflow and outflow in the wetlands to find the actual correlation between the theoretical and practical results. However from the data and resources available, the MUSIC simulation results should be adopted until more accurate measurements are able to be taken on site.

## 5.7 Chapter Summary

This chapter has discussed the sampling locations used for the water quality analysis, the tests conducted and the results obtained from these tests. A theoretical computer model was used to compare the Total Suspended Solids results, and to obtain results for phosphorus and nitrogen concentrations.

The results obtained from the water tests were generally within acceptable limits. The BOD test in particular showed that the quality of water within the wetland was very good, with some improvement as the water flowed through the wetland. If the same results were observed during a storm event, then the outflow water quality would be near pristine levels. The COD test was able to confirm this water quality improvement, although the difference between BOD and COD results was slightly larger than expected.

Some errors occurred during the TS and TSS tests. Some samples showed the final weight with the solids to be less than the dish or filter paper itself. These results are not possible and were therefore not included in the calculation of the average TS and TSS concentrations. The remaining TS samples showed an increase in TS concentration

as the water moved downstream. This is likely to be due to the presence of algae on the surface of the water at the downstream location. During periods of flow it is assumed that this algae would not be present and therefore different results would be obtained. The TSS results were very consistent with no change observed between the upstream and downstream locations. Once again, during periods of flow it is assumed that the results would be different to those obtained.

The MUSIC model was used to simulate the water quality improvement in the wetlands during a storm event. TSS, TP and TN were all tested and an improvement recorded for each. The TSS concentrations were very consistent with those results obtained from the samples. It is however recommended that further sampling is conducted within the wetlands. This would need to be done shortly after a storm event when the wetlands is experiencing both inflow and outflow. These future samples should be taken within the wetland as well as just upstream and just downstream. The results from these tests will be much more useful in determining whether the Wirraglen Scout Reserve Wetlands is providing adequate water quality improvement and whether or not any changes need to be made to the structure of the wetlands to improve the water quality improvement.

## Chapter 6

# Recommendations and Conclusion

## 6.1 Recommendation

The current wetland structure was tested on its storage and assimilative capacities. The Klein Creek catchment is ungauged and therefore no recorded streamflows were available. The storage capacity has therefore been determined entirely on theoretical models. The results of the analysis show that the wetlands are currently able to reduce the peak discharge for 2, 10 and 100 year design storms to below a pre-development level. As far as storage capacity is concerned, this is the critical point where the wetland must be able to detain the flow.

From these results it was determined that the wetland design is currently satisfactory in terms of storage capacity based on the current level of development in the catchment and sedimentation in the wetland. Future development will result in an increase in the peak discharge and possibly further sedimentation in the wetland. If this occurs, it is recommended that the weir height be increased to cater for the extra flow along the watercourse. The RORB model was used to test varied weir designs, and it was found for a 2 year storm that the peak outflow could be reduced from  $16.85\text{m}^3/\text{s}$  to  $14.46\text{m}^3/\text{s}$  by increasing the weir height by 0.5m. This would be the most simple solution to an increase in peak discharge in the catchment.

Tests were conducted to determine the assimilative capacity of the wetland. The results of these tests were not entirely conclusive. A lack of rainfall over the course of the project has resulted in no flow in Klein Creek and therefore meaningful water samples were not able to be obtained. Samples were taken within the wetland itself and the results from the tests on these samples showed that the wetland currently has a reasonably good water quality. However, these tests were unable to show the water quality improvement that occurs within the wetland during stormflow. These tests are required to provide a recommendation on what changes need to occur to improve the assimilative capacity of the wetlands. The MUSIC model was used to help provide an estimation of the water quality during periods of runoff. The results of this model showed good water quality improvement within the wetlands. No further recommendation for the assimilative capacity was possible based on the tests that could be carried out during the course of this project.



From the results obtained from the available data, it is recommended that no change to the design of the wetland should occur at this point in time. However, future changes to the nature of the catchment area may mean that the capacity of the wetlands is no longer sufficient. If this occurs, the wetland outlet structure should be modified to cater for these changes.

## 6.2 Achievement of Project Objectives

The specification for this project is provided in Appendix A. All the criteria outlined in this specification were addressed, excluding the testing of water samples upstream of the inlet and downstream of the outlet. This was not possible due to the lack of streamflow during the course of this project. Background information on wetlands is provided in Chapter 2. This chapter also includes descriptions and advantages and disadvantages for both SF and SSF wetlands. The role of wetlands in improving water quality was also discussed.

Chapter 3 discussed the Klein Creek catchment in some detail, including the geometry of the catchment, the land use and area of this catchment contributing to the inflow into the Wirraglen Scout Reserve Wetlands. The same section also discussed the flora and fauna observed within and immediately surrounding the wetlands. Chapter 3 then explained the process used in calculating the peak inflow using the Rational Method. The results obtained from this method were also discussed. The RORB model was used to confirm these results and this was discussed in Chapter 4. A sensitivity analysis of the critical flow through the wetlands first needed to be conducted before runoff hydrographs for the catchment could be calculated using RORB.

A water quality analysis was conducted in the catchment, using samples taken from within the wetland, and using the MUSIC software. The processes used and results obtained are discussed in Chapter 5. Testing was not undertaken for TN or TP. These were taken into consideration using the MUSIC software. In addition to the tests outlined in the project specification, Total Solids and Chemical Oxygen Demand tests were also undertaken.

The design of an improved wetland system was hindered due to the lack of rainfall. A recommendation based on the available data was provided in the previous section.

### 6.3 Further Work

There are two areas to this project that require further work. The main area of study that needs to be completed is determining the actual improvement in water quality as a result of the wetlands. This can only be conducted when there is observed inflow and outflow at the wetlands. The same tests performed on the samples from the wetland should be conducted on these additional samples to help provide comparisons between the samples. Tests on further samples taken from within the wetlands would also help to refine the data obtained from the initial samples. From these future samples, a design for an improved wetland system can be more accurately conducted if the current design is found to be unsatisfactory.

A refinement of the models used to determine the runoff hydrographs should also be conducted. This would include a more accurate measurement of the wetland area include the weir at the wetland outlet. An engineering survey may be required for this purpose. It is not expected that this procedure would make a large difference to the adequacy of the storage capacity of the wetlands. However, a closer correlation between the Rational Method and RORB calculations may be able to be obtained as a result. This needs to occur due to the absence of rainfall and streamflow data for the Klein Creek catchment.

# References

- ABS (2006), *Regional Population Growth*, Australian Bureau of Statistics, Commonwealth of Australia, Canberra, <http://www.ausstats.abs.gov.au/ausstats/>, viewed 13 September 2006.
- Argue, J. R. (1986), *Storm Drainage Design in Small Urban Catchments: A Handbook for Australian Practice*, Australian Road Research Board, Special Report SR34.
- Australian Wetlands* (n.d.), Australian Wetlands Pty Ltd, Sydney, <http://www.wetlands.com.au>, viewed 10 May 2006.
- Brock, M. A., Boon, P. I. & Grant, A., eds (1994), *Plants and Processes in Wetlands*, CSIRO Australia, East Melbourne, Victoria.
- City of Austin (2001), *Alternative Wastewater Treatment Demonstration Project*, City of Austin, Texas, <http://www.ci.austin.tx.us/wri/gophoto.htm>, viewed 10 May 2006.
- CNSC (n.d.), *Broad Land Use Types for Crows Nest Shire*, Crows Nest Shire Council.
- DDROC (2000), *Regional Standards Manual*, Darling Downs Regional Organisation of Councils.
- DEH (2004), *Wetlands*, Department of Environment and Heritage, Commonwealth of Australia, Canberra, <http://www.deh.gov.au/water/wetlands/about.html>, viewed 10 May 2006.
- DNR (2000), *Guidelines for Using Free Water Surface Constructed Wetlands to Treat Municipal Sewage*, Department of Natural Resources, State of Queensland, Bris-

- bane, [http://www.nrm.qld.gov.au/about/policy/documents/1023/pdfs/wic\\_2002\\_1023.pdf](http://www.nrm.qld.gov.au/about/policy/documents/1023/pdfs/wic_2002_1023.pdf), viewed 10 May 2006.
- DWLBC (2005), *Constructed Wetlands*, The Department of Water, Land and Biodiversity Conservation, Government of South Australia, Adelaide, [http://www.dwlbc.sa.gov.au/urban/catchments/constructed\\_wetlands.html](http://www.dwlbc.sa.gov.au/urban/catchments/constructed_wetlands.html), viewed 10 May 2006.
- Engineers Australia (1998), *Australian Rainfall and Runoff: A Guide to Flood Estimation, Volume 1*, The Institution of Engineers, Australia, Barton, ACT.
- GCCC (2006), *MUSIC Modelling Guidelines*, Gold Coast City Council.
- Gelt, J. (1997), *Constructed Wetlands: Using Human Ingenuity, Natural Processes to Treat Water, Build Habitat*, Vol. 9 of *Arroyo*, University of Arizona, Tucson, <http://ag.arizona.edu/AZWATER/arroyo/094wet.html>, viewed 10 May 2006.
- Google<sup>TM</sup> (2006), *Google Earth<sup>TM</sup> Image*, Google Inc., Mountain View, California, <http://www.google.com>, viewed 10 May 2006.
- Graham, R. (2003), *The Application of Constructed Wetlands to Achieve Improved Stormwater Water Quality*, University of Southern Queensland, Toowoomba.
- Gray, S. (2006), *email*, Crows Nest Shire Council, 16 August.
- Halverson, N. V. (2004), *Review of Constructed Subsurface Flow vs Surface Flow Wetlands*, Westinghouse Savannah River Company, Aiken, South Carolina, <http://www.osti.gov>, viewed 10 May 2006.
- IWA (2000), *Constructed Wetlands for Pollution Control*, IWA Publishing, London.
- Jenkins, G. A. (2004), *AUS-IFD Version 2.0*, School of Environmental Engineering, Griffith University, Nathan, Queensland, <http://www.ens.gu.edu.au/eve/research/AusIfd/AusIfdVer2.htm>, viewed 16 June 2006.
- Kadlec, R. H. & Knight, R. L. (1996), *Treatment Wetlands*, Lewis Publishers, Boca Raton, Florida.

- Kenny, M. (2004), *Seen Objects - A Photoblog - Wetland*, Photoblogs.org, <http://www.seenobjects.org/2004-12-10-wetland>, viewed 10 May 2006.
- Kent, D. M., ed. (1994), *Applied Wetlands Science and Technology*, Lewis Publishers, Boca Raton, Florida.
- Laurenson, E. M., Mein, R. G. & Nathan, R. J. (2006), *RORB Version 5 Runoff Routing Program User Manual*, Monash University, Melbourne and Sinclair Knight Merz Pty Ltd., <http://civil.eng.monash.edu.au/expertise/water/rorb>, viewed 28 August 2006.
- Magmedov, V. (n.d.), *Constructed Wetlands for Low-Cost Wastewater Treatment and Nature Conservation*, Environment Probe, Energy Probe Research Foundation, Toronto, <http://www.constructedwetlands.org/cw/index.cfm>, viewed 10 May 2006.
- Mitsch, W. J. & Gosselink, J. G. (2000), *Wetlands*, 3rd edn, Wiley, Chichester, New York.
- Moberly, C. (2001), *Phytoremediation & Constructed Wetlands: The Natural Treatment Alternative*, Colorado State University, Fort Collins, [http://rydberg.biology.colostate.edu/Phytoremediation/2001/Cindy\\_web/outline.html](http://rydberg.biology.colostate.edu/Phytoremediation/2001/Cindy_web/outline.html), viewed 10 May 2006.
- Moshiri, G. A., ed. (1993), *Constructed Wetlands for Water Quality Improvement*, Lewis Publishers, Boca Raton, Florida.
- Newman, G. M. (1994), *Construction of an Experimental Artificial Wetland for Nutrient Removal*, University of Southern Queensland, Toowoomba.
- Pilgrim, D. (1997), Runoff routing methods, in 'DH Pilgrim (ed.), *Australian Rainfall and Runoff: A Guide to Flood Estimation, vol. 1*', The Institution of Engineers, Australia, Barton, ACT, pp. V.43–V.57.
- TCC (n.d.), *Cooby Water Catchment*, Toowoomba City Council, <http://www.toowoomba.qld.gov.au/ebiz/artis/coobydam.php>, viewed 18 September 2006.

- Wikipedia (2006), *Biochemical Oxygen Demand*, Wikimedia Foundation, Inc, St Petersburg, Florida, [http://en.wikipedia.org/wiki/Biochemical\\_oxygen\\_demand](http://en.wikipedia.org/wiki/Biochemical_oxygen_demand), viewed 28 August 2006.
- Wong, T., Coleman, J., Duncan, H., Fletcher, T., Jenkins, G., Siriwardena, L., Taylor, A. & Wootton, R. (2005), *MUSIC Version 3 User Manual*, Cooperative Research Centre for Catchment Hydrology, Monash University, Clayton, Victoria, <http://www.toolkit.net.au>, viewed 28 August 2006.
- Wong, T. H. F., Breen, P. F., Somes, N. L. G. & Lloyd, S. D. (1999), *Managing Urban Stormwater Using Constructed Wetlands*, Cooperative Research Centre For Catchment Hydrology, Monash University, Clayton, Victoria.

## Appendix A

# Project Specification

University of Southern Queensland  
Faculty of Engineering and Surveying

**ENG4111/2 Research Project  
PROJECT SPECIFICATION**

FOR: **Toby MILLIKAN**

TOPIC: Current and Future Capacity of the Wirraglen Scout Reserve Wetlands to Improve Urban Stormwater Quality at Highfields

SUPERVISORS: Dr Ernest Yoong  
Stephen Gray, Crows Nest Shire Council

SPONSORSHIP: Crows Nest Shire Council

PROJECT AIM: This project aims to investigate the capacity of the Wirraglen Scout Reserve Wetlands and its role in improving urban stormwater quality at Highfields.

PROGRAMME: **Issue A, 24<sup>th</sup> March, 2006**

1. Research background information on wetlands and their role in improving the quality of urban stormwater.
2. Identify the catchment area and existing flora and fauna for the wetlands.
3. Calculate the flows for design storms of varying intensities and durations for Highfields taking into consideration any future development in the catchment area.
4. Take samples of water from the wetlands and test TSS, BOD, Nitrogen and Phosphorus levels to obtain the current quality of the water in the wetlands.
5. Obtain water samples both upstream and downstream of wetlands and conduct similar tests to (iv) in order to determine the role of the wetlands in improving water quality.
6. Design an improved wetlands system to increase capacity and improve nutrient removal efficiency.

*As Time Permits:*

7. Use a sensitivity analysis to determine the critical flow through the wetlands.

AGREED: \_\_\_\_\_ (Student) \_\_\_\_\_ (Supervisor)

\_\_\_/\_\_\_/\_\_\_

\_\_\_/\_\_\_/\_\_\_



## Appendix B

# Klein Creek Catchment Data

Included in this Appendix:

1. Location of Wetland
2. Klein Creek Catchment Area
3. Map of land use type for Crows Nest Shire

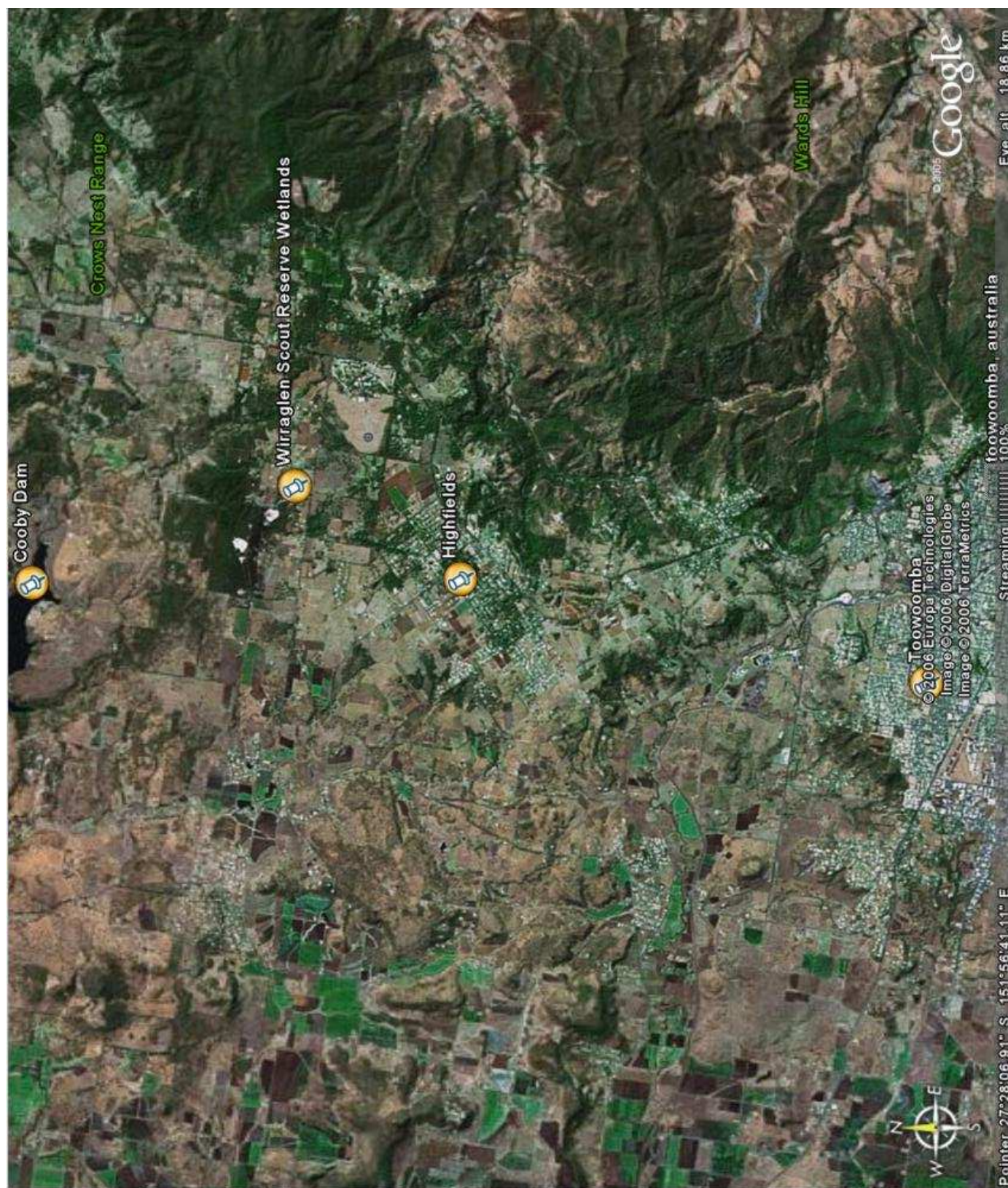


Figure B.1: Location of Wirraglen Scout Reserve Wetlands in relation to Highfields and Toowoomba (Google<sup>TM</sup> 2006)



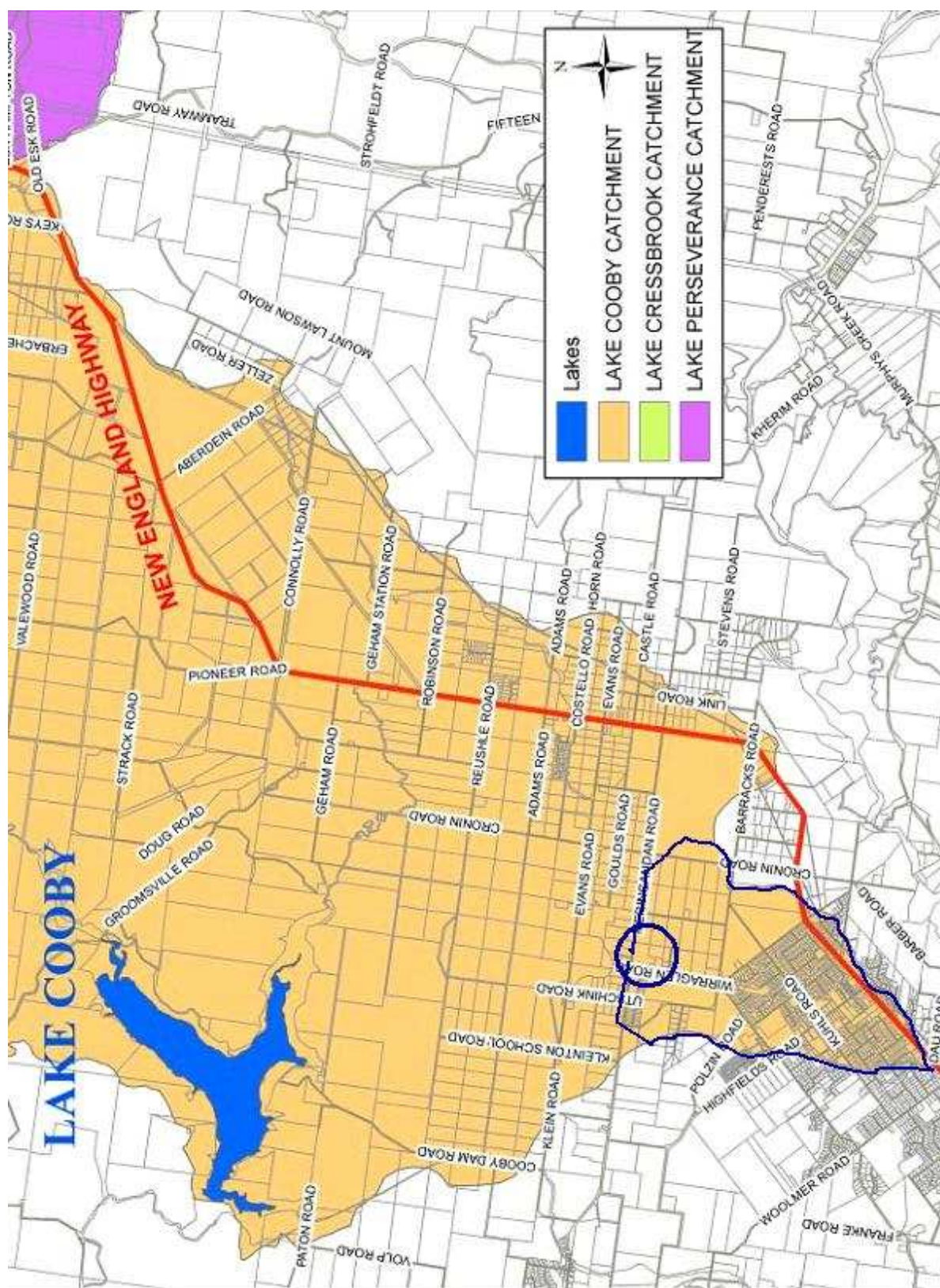


Figure B.2: Wirraglen Scout Reserve Wetlands Catchment as part of Cooby Dam Catchment (TCC n.d.)

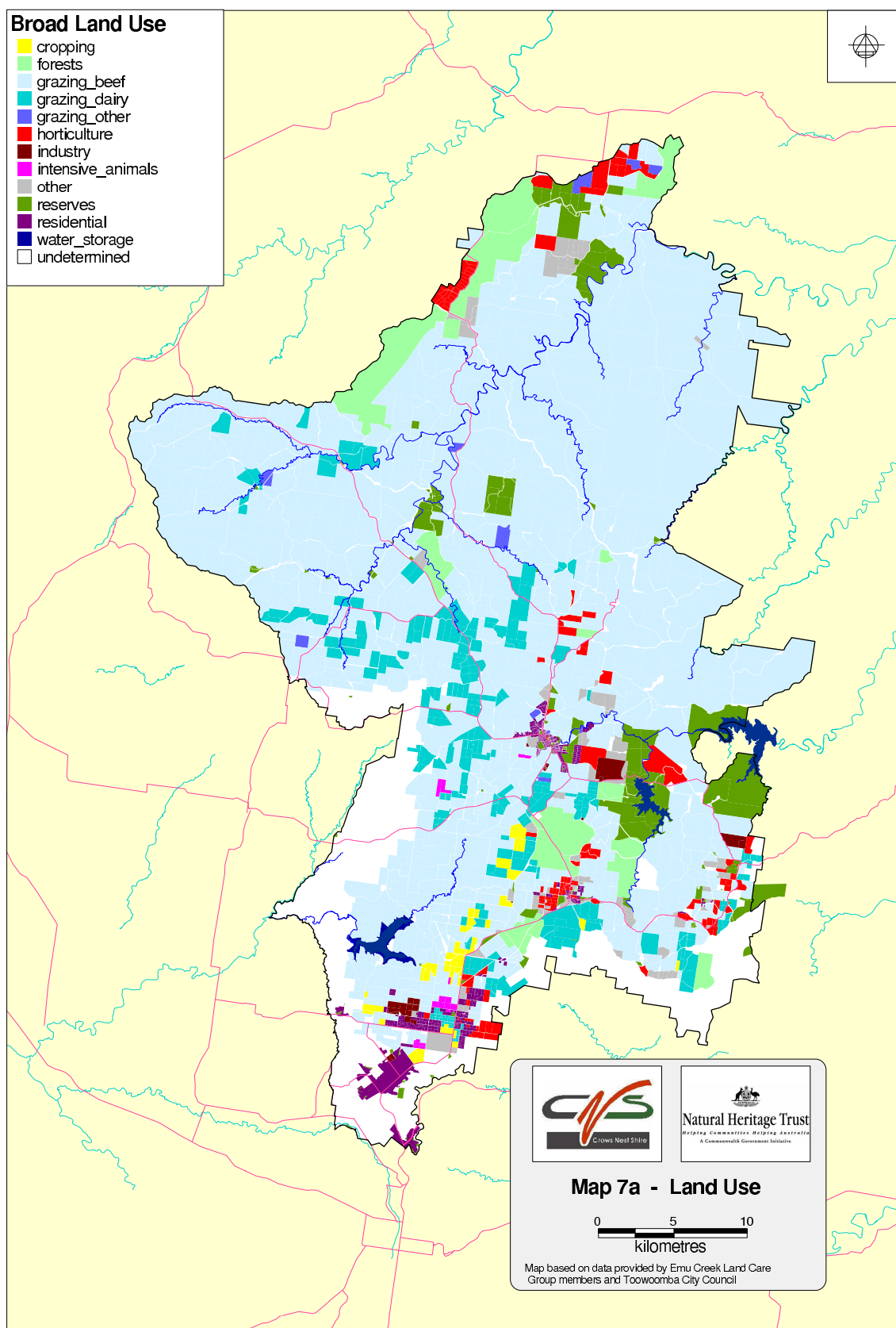


Figure B.3: Map of Land Use Type for Crows Nest Shire  
(CNSC n.d.)

## Appendix C

# Design Storm Calculations and Results

Included in this Appendix:

1. Coefficients of Runoff for Crows Nest Shire
  2. Rainfall Intensity for Time of Concentration  $< 60$  minutes for Highfields
  3. Rainfall Intensity for Time of Concentration  $> 60$  minutes for Highfields
  4. Catchment Subdivision for Rational Method Calculations
  5. IFD Chart
  6. Time of Concentration Calculations
  7. Peak Discharge Calculations for 2yr Design Storm
  8. Peak Discharge Calculations for 10yr Design Storm
  9. Peak Discharge Calculations for 100yr Design Storm
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**SECTION 5.22 B: CROWS NEST SHIRE  
STORMWATER DRAINAGE  
SUMMARY OF "DEEMED-TO-COMPLY" CRITERIA  
Q.U.D.M. SECTION 5.04**

**Table 5.04.4  
Coefficient of Runoff vs Development Category**

Development Category	Fraction Impervious	C <sub>1</sub>	C <sub>2</sub>	C <sub>5</sub>	C <sub>10</sub>	C <sub>20</sub>	C <sub>50</sub>	C <sub>100</sub>
(Y1) [X1] [X1]	(1.00) [.....] [.....]	(0.72)	(0.77)	(0.86)	(0.90)	(0.95)	1.00	1.00
Central Business	1.00	-	0.77	-	0.90	-	-	1.00
Light Industry	0.90	-	0.72	-	0.85	-	-	1.00
Significant Areas	0.90	-	0.72	-	0.85	-	-	1.00
Urban Residential "A" including Roads	0.6	-	0.52	-	0.61	-	-	0.73
Urban Residential "B" including Roads	0.38	-	0.44	-	0.52	-	-	0.62
Urban Residential "A" excluding Roads	0.55	-	0.49	-	0.58	-	-	0.70
Urban Residential "B" excluding Roads	0.33	-	0.42	-	0.49	-	-	0.59
Rural Residential "A"	0.2	-	0.40	-	0.47	-	-	0.56
Rural Residential "B"	0.1	-	0.38	-	0.45	-	-	0.54
Open Space	0.00	0.35	0.37	-	0.44	-	-	0.53
(Y0)	(0.00)	(0.35)	(0.37)	(0.42)	(0.44)	(0.46)	(0.51)	(0.53)

- References:
1. Australian Rainfall & Runoff 1987 "Probabilistic Model"
  2. 10 year ARI, 1 hour duration rainfall intensity  $^{10}I_1 = 51.7$  mm/hr (Crows Nest Town)
  3. 10 year ARI, 1 hour duration rainfall intensity  $^{10}I_1 = 49.2$  mm/hr (Highfields)

Note:

(Y0) Runoff Coefficient for 0.00 Fraction Impervious

(Y1) Runoff Coefficient for 1.00 Fraction Impervious

[X1] Fraction impervious for 1.00 Runoff Coefficient

- not applicable in context of ARI's adopted for major/minor storms (Table 5.06.1)

Figure C.1: Coefficient of Runoff vs Development Category for Crows Nest Shire  
(DDROC 2000)



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## SECTION 5.22 B: CROW'S NEST SHIRE (Crown Nest Town)

## STORMWATER DRAINAGE

## SUMMARY OF "DEEMED-TO-COMPLY" CRITERIA

## TABLE 5.07.1 RAINFALL/INTENSITY/DURATION

RAINFALL/INTENSITY/DURATION FOR TIMES OF CONCENTRATION OF &lt; 60 MINUTES.

EQUATION TYPE : Bureau of Meteorology, seven coefficient log-normal polynomial.

RAINFALL INTENSITY (mm/hour)

Duration		Average Recurrence Interval (Years)						
(mins)	(hours)	1	2	5	10	20	50	100
5.00	0.083	96	123	155	175	202	240	270
6.00	0.100	89	115	144	163	189	224	252
7.00	0.117	84	108	136	153	178	211	237
8.00	0.133	80	103	129	145	168	199	224
9.00	0.150	76	98	123	139	160	190	213
10.00	0.167	73	94	118	133	153	181	204
11.00	0.183	70	90	113	127	147	174	195
12.00	0.200	68	87	109	122	141	167	188
13.00	0.217	66	84	105	118	136	161	181
14.00	0.233	64	81	102	114	132	156	175
15.00	0.250	62	79	99	111	128	151	169
16.00	0.267	60	77	96	107	124	146	164
17.00	0.283	58	75	93	104	120	142	159
18.00	0.300	57	73	91	102	117	138	154
19.00	0.317	55	71	88	99	114	134	150
20.00	0.333	54	69	86	96	111	131	146
21.00	0.350	53	68	84	94	108	128	143
22.00	0.367	52	66	82	92	106	125	139
23.00	0.383	51	65	80	90	103	122	136
24.00	0.400	49.5	63	79	88	101	119	133
25.00	0.417	48.5	62	77	86	99	117	130
26.00	0.433	47.6	61	75	84	97	114	128
27.00	0.450	46.7	60	74	83	95	112	125
28.00	0.467	45.8	58	73	81	93	110	123
29.00	0.483	44.9	57	71	80	91	108	120
30.00	0.500	44.1	56	70	78	90	106	118
31.00	0.517	43.4	55	69	77	88	104	116
32.00	0.533	42.6	54	67	75	87	102	114
33.00	0.550	41.9	54	66	74	85	100	112
34.00	0.567	41.3	53	65	73	84	98	110
35.00	0.583	40.6	52	64	72	82	97	108
36.00	0.600	40.0	51	63	71	81	95	106
37.00	0.617	39.4	50	62	69	80	94	105
38.00	0.633	38.8	49.5	61	68	78	92	103
39.00	0.650	38.2	48.8	60	67	77	91	101
40.00	0.667	37.7	48.0	59	66	76	89	100
41.00	0.683	37.1	47.4	59	65	75	88	98
42.00	0.700	36.6	46.7	58	64	74	87	97
43.00	0.717	36.1	46.1	57	64	73	86	96
44.00	0.733	35.6	45.4	56	63	72	84	94
45.00	0.750	35.2	44.8	55	62	71	83	93
46.00	0.767	34.7	44.3	55	61	70	82	92
47.00	0.783	34.3	43.7	54	60	69	81	91
48.00	0.800	33.8	43.2	53	59	68	80	89
49.00	0.817	33.4	42.6	53	59	67	79	88
50.00	0.833	33.0	42.1	52	58	67	78	87
51.00	0.850	32.6	41.6	51	57	66	77	86
52.00	0.867	32.2	41.1	51	57	65	76	85
53.00	0.883	31.9	40.6	50	56	64	75	84
54.00	0.900	31.5	40.2	49.6	55	63	74	83
55.00	0.917	31.1	39.7	49.0	55	63	74	82
56.00	0.933	30.8	39.3	48.5	54	62	73	81
57.00	0.950	30.5	38.8	47.9	53	61	72	80
58.00	0.967	30.1	38.4	47.4	53	61	71	79
59.00	0.983	29.8	38.0	46.9	52	60	70	78
60.00	1.000	29.5	37.6	46.4	52	59	70	78

Date: May 2000

Page: B.5-35

Figure C.2: Rainfall Intensity for Time of Concentration &lt; 60 minutes for Highfields (DDROC 2000)

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**SECTION 5.22 B: CROW'S NEST SHIRE (Highfields)**  
**STORMWATER DRAINAGE**  
**SUMMARY OF "DEEMED-TO-COMPLY" CRITERIA**  
**TABLE 5.07.2**  
**RAINFALL/INTENSITY/DURATION**

LOCATION: HIGHFIELDS; 27.450 S, 151.950 E; 7 April 2000  
**RAINFALL/INTENSITY/DURATION FOR TIMES OF CONCENTRATION OF > 60 MINUTES.**

EQUATION TYPE : Bureau of Meteorology, seven coefficient log-normal polynomial.

ARI (years)	REGIONAL CO-EFFICIENTS						
	Regional Co-efficients						
	a	b	c	d	e	f	g
1	3.3573	-0.6320	-0.0642	0.01018	0.002887	-0.0005871	-0.0000228
2	3.5972	-0.6329	-0.0630	0.00914	0.003045	-0.0004353	-0.0000523
5	3.7933	-0.6366	-0.0589	0.00796	0.003120	-0.0002276	-0.0000671
10	3.8958	-0.6389	-0.0567	0.00750	0.003145	-0.0001358	-0.0001019
20	4.0255	-0.6405	-0.0546	0.00688	0.003164	0.0000323	-0.0001185
50	4.1781	-0.6421	-0.0526	0.00590	0.003281	0.0001210	-0.0001468
100	4.2826	-0.6439	-0.0508	0.00566	0.003255	0.0001770	-0.0001544

RAINFALL INTENSITY (mm/hour)								
DURATION		AVERAGE RECURRENCE INTERVAL (YEARS)						
(hours)	(mins)	1	2	5	10	20	50	100
1.0	60	28.7	36.5	44.4	49.2	56	65	72
1.5	90	22.0	27.9	34.0	37.6	42.8	49.9	55
2.0	120	18.0	22.8	27.9	30.8	35.1	40.9	45.4
3.0	180	13.5	17.0	20.9	23.1	26.3	30.6	34.0
4.5	270	10.0	12.6	15.5	17.3	19.6	22.9	25.4
6.0	360	8.13	10.2	12.6	14.1	16.0	18.6	20.7
9.0	540	6.06	7.60	9.45	10.7	12.1	14.0	15.6
12.0	720	4.93	6.19	7.74	8.86	9.96	11.6	12.9
18.0	1080	3.71	4.66	5.90	6.99	7.71	8.96	10.0
24.0	1440	3.03	3.81	4.88	6.01	6.49	7.53	8.46
30.0	1800	2.58	3.26	4.22	5.40	5.70	6.58	7.42
36.0	2160	2.26	2.86	3.73	4.98	5.13	5.90	6.67
48.0	2880	1.82	2.31	3.06	4.43	4.34	4.93	5.60
72.0	4320	1.30	1.65	2.23	3.80	3.34	3.68	4.22

Figure C.3: Rainfall Intensity for Time of Concentration > 60 minutes for Highfields (DDROC 2000)



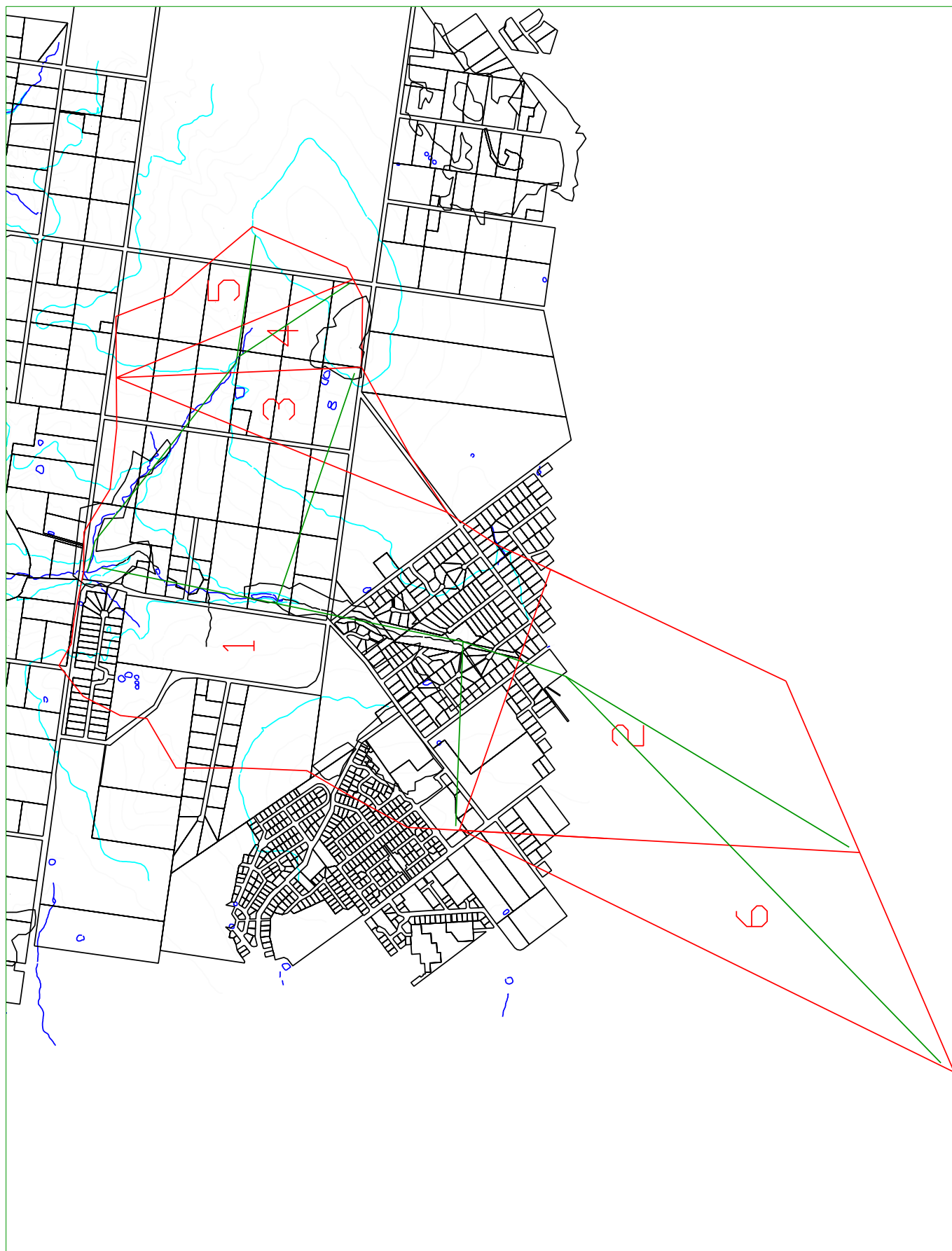


Figure C.4: Catchment Subdivision with Flow Paths

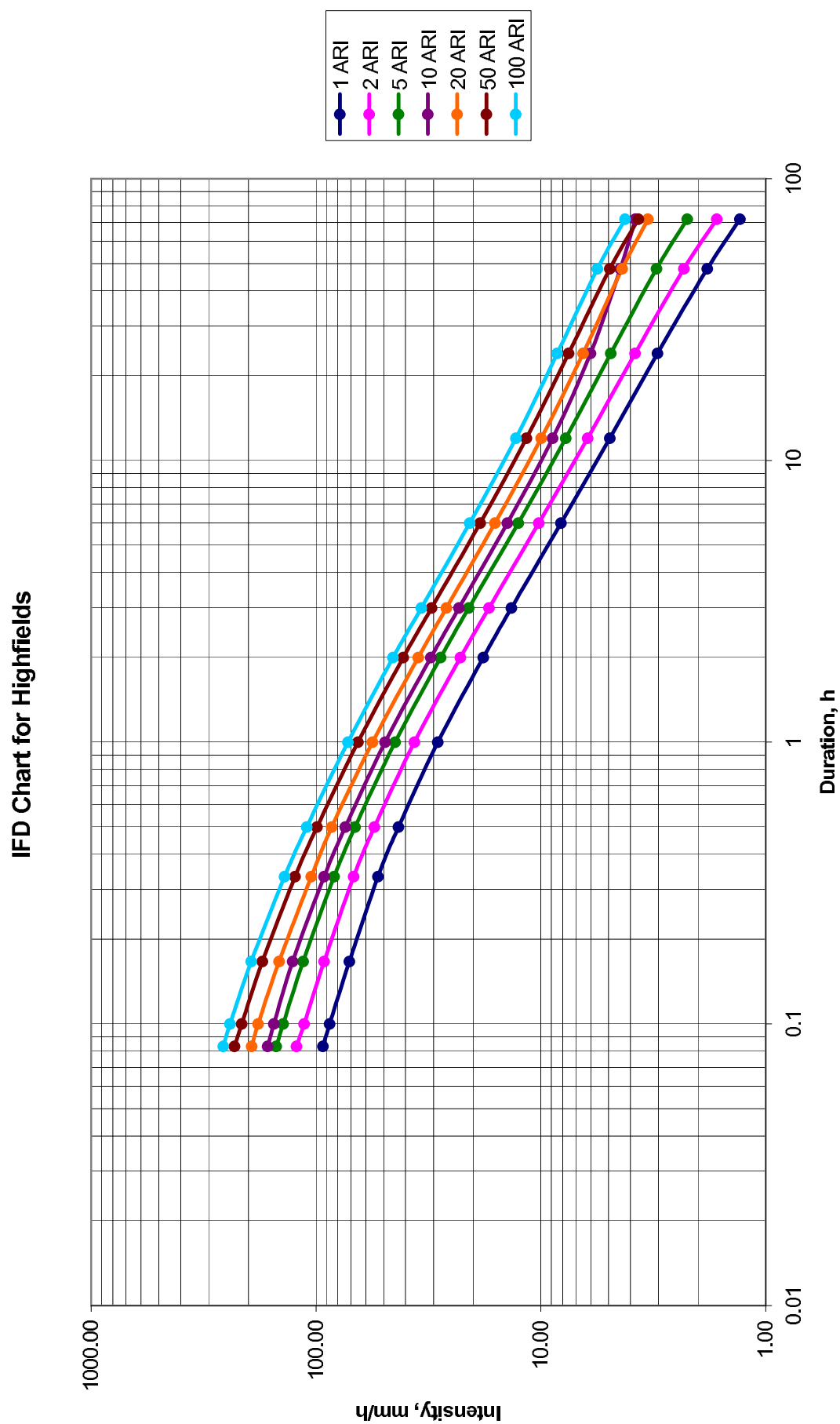


Figure C.5: IFD Curve for Highfields

## Time of Concentration Calculations

Time of Concentration													
Area 1		Upper Catchment			Travel Times (minutes) for 2yr Storm			Travel Times (minutes) for 10yr Storm			Travel Times (minutes) for 100yr Storm		
La = 1530		Sa = 0.028	n* = 0.05		ta(La)	66.32		ta(La)	57.00		ta(La)	46.81	
ta(La)*0.4		273.7945			tbL(a+b)	279.60		tbL(a+b)	232.14		tbL(a+b)	188.23	
					tb(La)	180.05		tb(La)	139.95		tb(La)	116.47	
Lb = 1050		Lower Catchment	n* = 0.15		Total Overland Flow Time (minutes)			Total Overland Flow Time (minutes)			Total Overland Flow Time (minutes)		
		Sb = 0.024			165.88			149.20			118.57		
tbL(a+b)*0.4		758.4425											
tb(La)*0.4		554.3266											
Area 2		Upper Catchment			Travel Times (minutes) for 2yr Storm			Travel Times (minutes) for 10yr Storm			Travel Times (minutes) for 100yr Storm		
La = 2760		Sa = 0.018	n* = 0.05		ta(La)	123.04		ta(La)	105.86		ta(La)	87.72	
ta(La)*0.4		445.3538			tbL(a+b)	388.06		tbL(a+b)	320.89		tbL(a+b)	260.35	
					tb(La)	285.93		tb(La)	245.52		tb(La)	199.10	
Lb = 1050		Lower Catchment	n* = 0.15		Total Overland Flow Time (minutes)			Total Overland Flow Time (minutes)			Total Overland Flow Time (minutes)		
		Sb = 0.024			215.16			181.22			148.97		
tbL(a+b)*0.4		958.3096											
tb(La)*0.4		789.7621											
Area 3		Upper Catchment			Travel Times (minutes) for 2yr Storm			Travel Times (minutes) for 10yr Storm			Travel Times (minutes) for 100yr Storm		
La = 1090		Sa = 0.041	n* = 0.15		ta(La)	102.29		ta(La)	87.98		ta(La)	72.70	
tf*0.4		385.1545			tbL(a+b)	238.87		tbL(a+b)	195.73		tbL(a+b)	151.21	
					tb(La)	125.48		tb(La)	107.96		tb(La)	89.49	
Lb = 1050		Lower Catchment	n* = 0.15		Total Overland Flow Time (minutes)			Total Overland Flow Time (minutes)			Total Overland Flow Time (minutes)		
		Sb = 0.024			215.89			178.75			134.42		
tbL(a+b)*0.4		677.9523											
tb(La)*0.4		452.2793											
Area 4		Upper Catchment			Travel Times (minutes) for 2yr Storm			Travel Times (minutes) for 10yr Storm			Travel Times (minutes) for 100yr Storm		
La = 820		Sa = 0.049	n* = 0.15		ta(La)	76.94		ta(La)	66.14		ta(La)	54.43	
ta(La)*0.4		307.7824			tbL(a+b)	139.21		tbL(a+b)	119.80		tbL(a+b)	99.46	
					tb(La)	79.47		tb(La)	68.32		tb(La)	56.25	
Lb = 890		Lower Catchment	n* = 0.15		Total Overland Flow Time (minutes)			Total Overland Flow Time (minutes)			Total Overland Flow Time (minutes)		
		Sb = 0.045			136.87			117.62			97.64		
tbL(a+b)*0.4		490.7362											
tb(La)*0.4		315.7467											
Area 5		Upper Catchment			Travel Times (minutes) for 2yr Storm			Travel Times (minutes) for 10yr Storm			Travel Times (minutes) for 100yr Storm		
La = 1010		Sa = 0.04	n* = 0.15		ta(La)	97.43		ta(La)	83.79		ta(La)	69.18	
ta(La)*0.4		370.6714			tbL(a+b)	150.87		tbL(a+b)	129.86		tbL(a+b)	107.95	
					tb(La)	93.15		tb(La)	80.10		tb(La)	66.10	
Lb = 890		Lower Catchment	n* = 0.15		Total Overland Flow Time (minutes)			Total Overland Flow Time (minutes)			Total Overland Flow Time (minutes)		
		Sb = 0.045			155.15			133.54			111.03		
tbL(a+b)*0.4		522.7603											
tb(La)*0.4		357.8024											
Area 6		Upper Catchment			Travel Times (minutes) for 2yr Storm			Travel Times (minutes) for 10yr Storm			Travel Times (minutes) for 100yr Storm		
La = 3345		Sa = 0.018	n* = 0.05		ta(La)	142.49		ta(La)	122.63		ta(La)	101.85	
ta(La)*0.4		499.8013			tbL(a+b)	437.48		tbL(a+b)	361.25		tbL(a+b)	293.17	
					tb(La)	347.86		tb(La)	288.03		tb(La)	233.63	
Lb = 1050		Lower Catchment	n* = 0.15		Total Overland Flow Time (minutes)			Total Overland Flow Time (minutes)			Total Overland Flow Time (minutes)		
		Sb = 0.024			232.10			195.85			161.39		
tbL(a+b)*0.4		1044.052											
tb(La)*0.4		886.3159											

## Calculations for 2yr Design Storm

### DESIGN FLOWS FOR 2 YEAR ARI STORM

#### Area 1

<b>Full Area Calculation</b>					
tc =	165.88	round to:	166 minutes		
2t	166min = 21.23				
F =	0.278				
Rural Residential B	1.5015 km <sup>2</sup>	f =	0.1	C2 =	0.38
Urban Residential A	0.2145 km <sup>2</sup>	f =	0.6	C2 =	0.52
Urban Residential B	0.429 km <sup>2</sup>	f =	0.38	C2 =	0.44
				From Regional Standards Manual	
Q1 =	5.141 m <sup>3</sup> /s				
<b>Partial Area Calculation</b>					
tc =	60 (assumed)	In 60 minutes, all the impervious area will contribute and part of the pervious area			
2t	60min = 36.50				
F =	0.278				
Residential:		Impervious:	0.44187	f =	1
		Pervious:	1.70313	f =	0
				C2 =	0.77
				C2 =	0.37
		assume approximately 50% of pervious area will contribute in 60 minutes			
Q1 =	6.650 m <sup>3</sup> /s				
	The part-area calculation produced the largest flow				

**Total Area**  
2.145 km<sup>2</sup>

**Q Adopted**  
6.650 m<sup>3</sup>/s

#### Area 2

<b>Full Area Calculation</b>					
tc =	215.16	round to:	215 minutes		
2t	215min = 14.64				
F =	0.278				
Urban Residential B	1.989 km <sup>2</sup>	f =	0.38	C2 =	0.44
				From Regional Standards Manual	
Q2 =	3.561 m <sup>3</sup> /s				
<b>Partial Area Calculation</b>					
tc =	80 (assumed)	In 80 minutes, all the impervious area will contribute and part of the pervious area			
2t	80min = 31.35				
F =	0.278				
Residential:		Impervious:	0.75582	f =	1
		Pervious:	1.23318	f =	0
				C2 =	0.77
				C2 =	0.37
		assume approximately 50% of pervious area will contribute in 80 minutes			
Q2 =	7.061 m <sup>3</sup> /s				
	The part-area calculation produced the largest flow				

**Total Area**  
1.989 km<sup>2</sup>

**Q Adopted**  
7.061 m<sup>3</sup>/s

#### Area 3

<b>Full Area Calculation</b>					
tc =	215.69	round to:	216 minutes		
2t	216min = 14.59				
F =	0.278				
Open Space	0.1038 km <sup>2</sup>	f =	0	C2 =	0.37
Rural Residential B	0.2422 km <sup>2</sup>	f =	0.1	C2 =	0.38
				From Regional Standards Manual	
Q3 =	0.529 m <sup>3</sup> /s				
<b>Partial Area Calculation</b>					
tc =	80 (assumed)	In 80 minutes, all the impervious area will contribute and part of the pervious area			
2t	80min = 31.35				
F =	0.278				
Residential:		Impervious:	0.02422	f =	1
		Pervious:	0.32178	f =	0
				C2 =	0.77
				C2 =	0.37
		assume approximately 50% of pervious area will contribute in 80 minutes			
Q3 =	0.681 m <sup>3</sup> /s				
	The part-area calculation produced the largest flow				

**Total Area**  
0.346 km<sup>2</sup>

**Q Adopted**  
0.681 m<sup>3</sup>/s

<b>Full Area Calculation</b>				
tc =	136.67	round to:	137 minutes	
21137min =	23.53			
F =	0.278			
<b>Rural Residential B</b>	0.284 km <sup>2</sup>	f = 0.1	<b>C2 = 0.38</b>	From Regional Standards Manual
<b>Q4 =</b>	<b>0.706</b> m <sup>3</sup> /s			
<b>Partial Area Calculation</b>				
tc =	50 (assumed)	In 50 minutes, all the impervious area will contribute and part of the		
2150min =	40.16	pervious area		
F =	0.278			
<b>Residential:</b>	<b>Impervious:</b>	0.0284	f = 1	<b>C2 = 0.77</b>
	<b>Pervious:</b>	0.2556	f = 0	<b>C2 = 0.37</b>
assume approximately 50% of pervious area will contribute in 50 minutes				
<b>Q4 =</b>	<b>0.772</b> m <sup>3</sup> /s			
The part-area calculation produced the largest flow				

**Q Adopted**  
0.772 m3/s

<b>Full Area Calculation</b>					
tc = 155.15	round to:	<b>155 minutes</b>			
21155min = 22.03					
F = 0.278					
<b>Open Space</b>	0.176 km <sup>2</sup>	f = 0	<b>C2 = 0.37</b>		
<b>Rural Residential B</b>	0.704 km <sup>2</sup>	f = 0.1	<b>C2 = 0.38</b>		
From Regional Standards Manual					
<b>Q5 =</b>	<b>2.037</b> m <sup>3</sup> /s				
<b>Partial Area Calculation</b>					
tc = 60 (assumed)	In 60 minutes, all the impervious area will contribute and part of the pervious area				
2160min = 36.50					
F = 0.278					
<b>Residential:</b>	<b>Impervious:</b>	0.0704	f = 1		
	<b>Pervious:</b>	0.8096	f = 0		
			<b>C2 = 0.77</b>		
			<b>C2 = 0.37</b>		
assume approximately 50% of pervious area will contribute in 60 minutes					
<b>Q5 =</b>	<b>2.070</b> m <sup>3</sup> /s				
The part-area calculation produced the largest flow					

**Q Adopted**  
2.070 m3/s

<b>Full Area Calculation</b>				
tc = 232.10	round to:	<b>232 minutes</b>		
21232min = 13.86				
F = 0.278				
Urban Residential B	0.8968 km2	f = 0.38	C2 = 0.44	From Regional Standards Manual
Rural Residential B	0.2242 km2	f = 0.1	C2 = 0.38	
Q6 =	1.848 m3/s			
<b>Partial Area Calculation</b>				
tc = 90 (assumed)	In 90 minutes, all the impervious area will contribute and part of the pervious area			
2190min = 29.45				
F = 0.278				
Residential:	<b>Impervious:</b>	0.363204	f = 1	C2 = 0.77
	<b>Pervious:</b>	0.757796	f = 0	C2 = 0.37
assume approximately 50% of pervious area will contribute in 90 minutes				
Q6 =	3.438 m3/s			
The part-area calculation produced the largest flow				

**Q Adopted**  
3.438 m<sup>3</sup>/s

**Total Area**  
6.765 km<sup>2</sup>

## Calculations for 10yr Design Storm



### DESIGN FLOWS FOR 10 YEAR ARI STORM

#### Area 1

<b>Full Area Calculation</b>				
tc = 149.20	round to:	149 minutes		
10 149min = 30.19				
F = 0.278				
Rural Residential B	1.5015 km2	f = 0.1	C10 = 0.45	From Regional Standards Manual
Urban Residential A	0.2145 km2	f = 0.6	C10 = 0.61	
Urban Residential B	0.429 km2	f = 0.38	C10 = 0.52	
Q1 =	8.640 m3/s			
<b>Partial Area Calculation</b>				
tc = 55 (assumed)	In 55 minutes, all the impervious area will contribute and part of the pervious area			
10 55min = 51.52				
F = 0.278				
Residential:	<b>Impervious:</b>	0.44187	f = 1	C10 = 0.9
	<b>Pervious:</b>	1.70313	f = 0	C10 = 0.44
assume approximately 50% of pervious area will contribute in 55 minutes				
Q1 =	11.063 m3/s			
The part-area calculation produced the largest flow				

**Total Area**  
2.145 km2

**Q Adopted**  
11.063 m3/s

#### Area 2

<b>Full Area Calculation</b>				
tc = 181.22	round to:	181 minutes		
10 181min = 22.94				
F = 0.278				
Urban Residential B	1.989 km2	f = 0.38	C10 = 0.52	From Regional Standards Manual
Q2 =	6.595 m3/s			
<b>Partial Area Calculation</b>				
tc = 70 (assumed)	In 70 minutes, all the impervious area will contribute and part of the pervious area			
10 70min = 45.32				
F = 0.278				
Residential:	Impervious:	0.75582	f = 1	C10 = 0.9
	Pervious:	1.23318	f = 0	C10 = 0.44
assume approximately 50% of pervious area will contribute in 70 minutes				
Q2 =	11.989 m3/s			
The part-area calculation produced the largest flow				

**Total Area**  
1.989 km2

**Q Adopted**  
11.989 m3/s

#### Area 3

<b>Full Area Calculation</b>				
tc = 178.75	round to:	179 minutes		
10 179min = 27.33				
F = 0.278				
Open Space	0.1038 km2	f = 0	C10 = 0.44	From Regional Standards Manual
Rural Residential B	0.2422 km2	f = 0.1	C10 = 0.45	
Q3 =	1.175 m3/s			
<b>Partial Area Calculation</b>				
tc = 70 (assumed)	In 70 minutes, all the impervious area will contribute and part of the pervious area			
10 70min = 45.32				
F = 0.278				
Residential:	Impervious:	0.02422	f = 1	C10 = 0.9
	Pervious:	0.32178	f = 0	C10 = 0.44
assume approximately 50% of pervious area will contribute in 70 minutes				
Q3 =	1.167 m3/s			
The full-area calculation produced the largest flow				

**Total Area**  
0.346 km2

**Q Adopted**  
1.175 m3/s

<b>Area 4</b>					
<div> <div> <div>tc = 117.62</div> <div>round to:</div> <div>118 minutes</div> </div> <div> <div>10 118min = 34.24</div> <div>F = 0.278</div> </div> </div> <div> <div> <div>Rural Residential B</div> <div>0.284 km2</div> <div>f = 0.1</div> <div>C10 = 0.45</div> <div>From Regional Standards Manual</div> </div> <div> <div>Q4 =</div> <div>1.216 m3/s</div> </div> </div> <div> <div> <div>tc = 45 (assumed)</div> <div>10 145min = 57.27</div> <div>F = 0.278</div> </div> <div> <div>Partial Area Calculation</div> <div>In 45 minutes, all the impervious area will contribute and part of the pervious area</div> </div> <div> <div> <div>Residential:</div> <div> <div>Impervious: 0.0284</div> <div>Pervious: 0.2556</div> </div> <div> <div>f = 1</div> <div>f = 0</div> </div> <div> <div>C10 = 0.9</div> <div>C10 = 0.44</div> </div> </div> <div> <div>assume approximately 50% of pervious area will contribute in 45 minutes</div> </div> <div> <div>Q4 =</div> <div>1.302 m3/s</div> </div> <div> <div>The part-area calculation produced the largest flow</div> </div> </div> </div>					<div>Total Area</div> <div>0.284 km2</div>
<b>Area 5</b>					
<div> <div> <div>tc = 133.54</div> <div>round to:</div> <div>134 minutes</div> </div> <div> <div>10 134min = 31.97</div> <div>F = 0.278</div> </div> </div> <div> <div> <div>Open Space</div> <div>0.176 km2</div> <div>f = 0</div> <div>C10 = 0.44</div> </div> <div> <div>Rural Residential B</div> <div>0.704 km2</div> <div>f = 0.1</div> <div>C10 = 0.45</div> <div>From Regional Standards Manual</div> </div> </div> <div> <div>Q5 =</div> <div>3.504 m3/s</div> </div> <div> <div> <div>tc = 55 (assumed)</div> <div>10 155min = 51.52</div> <div>F = 0.278</div> </div> <div> <div>Partial Area Calculation</div> <div>In 55 minutes, all the impervious area will contribute and part of the pervious area</div> </div> <div> <div> <div>Residential:</div> <div> <div>Impervious: 0.0704</div> <div>Pervious: 0.8096</div> </div> <div> <div>f = 1</div> <div>f = 0</div> </div> <div> <div>C10 = 0.9</div> <div>C10 = 0.44</div> </div> </div> <div> <div>assume approximately 50% of pervious area will contribute in 55 minutes</div> </div> <div> <div>Q5 =</div> <div>3.459 m3/s</div> </div> <div> <div>The full-area calculation produced the largest flow</div> </div> </div> </div>					<div>Total Area</div> <div>0.88 km2</div>
<b>Area 6</b>					
<div> <div> <div>tc = 195.85</div> <div>round to:</div> <div>196 minutes</div> </div> <div> <div>10 196min = 21.70</div> <div>F = 0.278</div> </div> </div> <div> <div> <div>Urban Residential B</div> <div>0.8968 km2</div> <div>f = 0.38</div> <div>C10 = 0.52</div> </div> <div> <div>Rural Residential B</div> <div>0.2242 km2</div> <div>f = 0.1</div> <div>C10 = 0.45</div> <div>From Regional Standards Manual</div> </div> </div> <div> <div>Q6 =</div> <div>3.422 m3/s</div> </div> <div> <div> <div>tc = 80 (assumed)</div> <div>10 180min = 42.20</div> <div>F = 0.278</div> </div> <div> <div>Partial Area Calculation</div> <div>In 80 minutes, all the impervious area will contribute and part of the pervious area</div> </div> <div> <div> <div>Residential:</div> <div> <div>Impervious: 0.363204</div> <div>Pervious: 0.757796</div> </div> <div> <div>f = 1</div> <div>f = 0</div> </div> <div> <div>C10 = 0.9</div> <div>C10 = 0.44</div> </div> </div> <div> <div>assume approximately 50% of pervious area will contribute in 80 minutes</div> </div> <div> <div>Q6 =</div> <div>5.791 m3/s</div> </div> <div> <div>The part-area calculation produced the largest flow</div> </div> </div> </div>					<div>Total Area</div> <div>1.121 km2</div>
<div> <div>Total Flow</div> <div>34.824 m3/s</div> </div> <div> <div>Total Area</div> <div>6.765 km2</div> </div>					

## Calculations for 100yr Design Storm

**DESIGN FLOWS FOR 100 YEAR ARI STORM****Area 1**

**Full Area Calculation**  
tc = 118.57 round to: **119 minutes**

100|119min = 48.81

F = 0.278

Rural Residential B 1.5015 km2  
Urban Residential A 0.2145 km2  
Urban Residential B 0.429 km2

f = 0.1  
f = 0.6  
f = 0.38

C100 = 0.54  
C100 = 0.73  
C100 = 0.62

From Regional  
Standards Manual

**Total Area**  
2.145 km2

**Q1 = 16.738 m3/s**

**Partial Area Calculation**

tc = 50 (assumed)  
100|50min = 79.75  
F = 0.278  
In 50 minutes, all the impervious area will contribute and part of the pervious area

Residential: **Impervious:** 0.44187 f = 1 C100 = 1  
**Pervious:** 1.70313 f = 0 C100 = 0.53  
assume approximately 50% of pervious area will contribute in 50 minutes

**Q1 = 19.802 m3/s**

*The part-area calculation produced the largest flow*

**Q Adopted**  
19.802 m3/s

**Area 2**

**Full Area Calculation**  
tc = 148.97 round to: **149 minutes**

100|149min = 42.92

F = 0.278

Urban Residential B 1.989 km2

f = 0.38

C100 = 0.62

From Regional  
Standards Manual

**Total Area**  
1.989 km2

**Q2 = 14.715 m3/s**

**Partial Area Calculation**

tc = 60 (assumed)  
100|60min = 72.00  
F = 0.278  
In 60 minutes, all the impervious area will contribute and part of the pervious area

Residential: **Impervious:** 0.75582 f = 1 C100 = 1  
**Pervious:** 1.23318 f = 0 C100 = 0.53  
assume approximately 50% of pervious area will contribute in 60 minutes

**Q2 = 21.670 m3/s**

*The part-area calculation produced the largest flow*

**Q Adopted**  
21.670 m3/s

**Area 3**

**Full Area Calculation**  
tc = 134.42 round to: **134 minutes**

100|134min = 45.61

F = 0.278

Open Space 0.1038 km2  
Rural Residential B 0.2422 km2

f = 0  
f = 0.1

C100 = 0.53  
C100 = 0.54

From Regional  
Standards Manual

**Total Area**  
0.346 km2

**Q3 = 2.356 m3/s**

**Partial Area Calculation**

tc = 55 (assumed)  
100|55min = 75.60  
F = 0.278  
In 55 minutes, all the impervious area will contribute and part of the pervious area

Residential: **Impervious:** 0.02422 f = 1 C100 = 1  
**Pervious:** 0.32178 f = 0 C100 = 0.53  
assume approximately 50% of pervious area will contribute in 55 minutes

**Q3 = 2.301 m3/s**

*The full-area calculation produced the largest flow*

**Q Adopted**  
2.356 m3/s

<b>Area 4</b>					
tc = 97.64		round to:	<b>Full Area Calculation</b>		
100I98min = 54.53			<b>98 minutes</b>		
F = 0.278					
Rural Residential B	0.284 km2	f = 0.1	C100 = 0.54	From Regional Standards Manual	<b>Total Area</b> 0.284 km2
Q4 =	2.325 m3/s				
tc = 40 (assumed)		<b>Partial Area Calculation</b> In 40 minutes, all the impervious area will contribute and part of the pervious area			
100I40min = 90.28					
F = 0.278					
Residential:	<b>Impervious:</b> 0.0284	f = 1	C100 = 1		
	<b>Pervious:</b> 0.2556	f = 0	C100 = 0.53		
assume approximately 50% of pervious area will contribute in 40 minutes					
Q4 =	2.413 m3/s				
The part-area calculation produced the largest flow					<b>Q Adopted</b> 2.413 m3/s
<b>Area 5</b>					
tc = 111.03		round to:	<b>Full Area Calculation</b>		
100I111min = 50.79			<b>111 minutes</b>		
F = 0.278					
Open Space	0.176 km2	f = 0	C100 = 0.53		<b>Total Area</b>
Rural Residential B	0.704 km2	f = 0.1	C100 = 0.54	From Regional Standards Manual	0.88 km2
Q5 =	6.685 m3/s				
tc = 45 (assumed)		<b>Partial Area Calculation</b> In 45 minutes, all the impervious area will contribute and part of the pervious area			
100I45min = 84.57					
F = 0.278					
Residential:	<b>Impervious:</b> 0.0704	f = 1	C100 = 1		
	<b>Pervious:</b> 0.8096	f = 0	C100 = 0.53		
assume approximately 50% of pervious area will contribute in 60 minutes					
Q5 =	6.699 m3/s				
The part-area calculation produced the largest flow					<b>Q Adopted</b> 6.699 m3/s
<b>Area 6</b>					
tc = 161.39		round to:	<b>Full Area Calculation</b>		
100I161min = 41.06			<b>161 minutes</b>		
F = 0.278					
Urban Residential B	0.8968 km2	f = 0.38	C100 = 0.62	From Regional Standards Manual	<b>Total Area</b>
Rural Residential B	0.2242 km2	f = 0.1	C100 = 0.54		1.121 km2
Q6 =	7.729 m3/s				
tc = 65 (assumed)		<b>Partial Area Calculation</b> In 65 minutes, all the impervious area will contribute and part of the pervious area			
100I65min = 68.83					
F = 0.278					
Residential:	<b>Impervious:</b> 0.363204	f = 1	C100 = 1		
	<b>Pervious:</b> 0.757796	f = 0	C100 = 0.53		
assume approximately 50% of pervious area will contribute in 65 minutes					
Q6 =	10.792 m3/s				
The part-area calculation produced the largest flow					<b>Q Adopted</b> 10.792 m3/s
			<b>Total Flow</b>	<b>Total Area</b>	
			63.731 m3/s	6.765 km2	

## Appendix D

# RORB Data and Results

Included in this Appendix:

1. Catchment Subdivision for RORB Model
2. Catchment File for Natural Catchment Conditions
3. Catchment File for Current Catchment Conditions
4. Plots of Storm Durations used to find Critical Duration
5. Rainfall Hyetographs and Runoff Hydrographs for Natural Conditions
6. Plots of Kuhls Road Detention Basin Inflow and Outflow
7. Plots of Wetland Inflow and Outflow

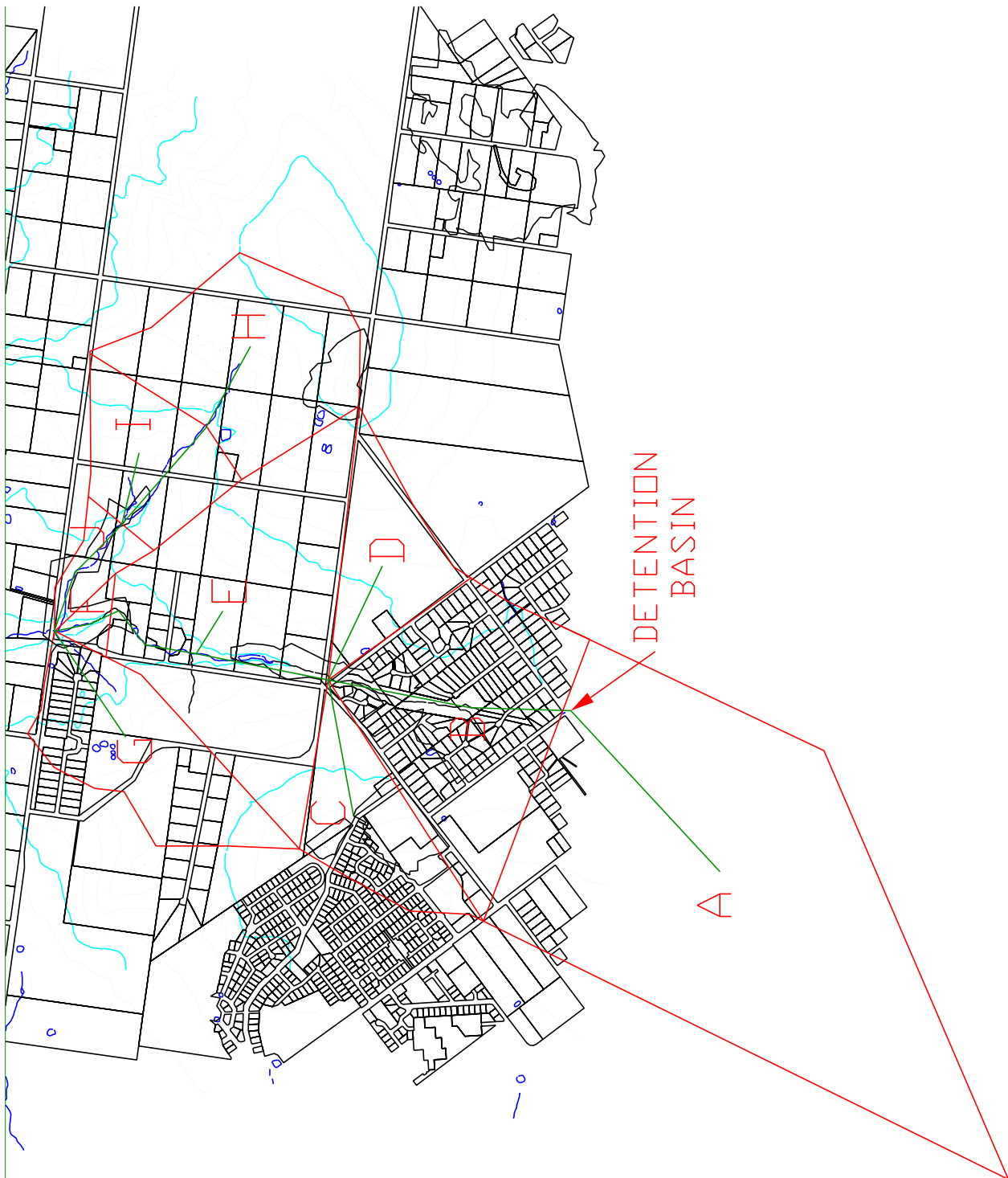


Figure D.1: Catchment Subdivision for RORB Model

### Catchment File for Natural Catchment Conditions

Klein Creek, Highfields

1, all natural

1,0.95,-99, SUB-AREA A (reach length 0.95km)

2,0.6,-99, SUB-AREA B (added to running hydrograph, reach length 0.6km)

3, store hydrograph

1,0.58,-99, SUB-AREA C (reach length 0.58km)

4, add hydrograph from C to running hydrograph

3, store hydrograph

1,0.55,-99, SUB-AREA D (reach length 0.55km)

4, add hydrograph from D to running hydrograph

5,0.58,-99, route to addition of sub-area E

3, store hydrograph

1,0.2,-99, SUB-AREA E (reach length 0.2km)

4, add hydrograph from E to running hydrograph

5,0.47,-99, route to addition of sub-area F

2,0.15,-99, SUB-AREA F (added to running hydrograph, reach length 0.15km)

3, store hydrograph

1,0.57,-99, SUB-AREA G (reach length 0.57km)

4, add hydrograph from G to running hydrograph

3, store hydrograph

1,0.8,-99, SUB-AREA H (reach length 0.8km)

3, store hydrograph

1,0.33,-99, SUB-AREA I (added to running hydrograph, reach length 0.33km)

4, add hydrograph from I to running hydrograph

5,0.2,-99, route to sub-area J input

2,0.37,-99, SUB-AREA J (added to running hydrograph, reach length 0.37km)

4, add hydrograph from J to running hydrograph

7

Wetland Inflow

0, end of control vector

C sub area data

1.42,1,0.5,0.4,1.5,0.1,0.5,0.75,0.4,0.2,-99

0,-99, all sub-areas completely pervious

Figure D.2: Catchment File used in RORB Model for Natural Conditions



### Catchment File for Current Catchment Conditions

Klein Creek, Highfields  
 0, catchment urbanised  
 1,1,0.43,-99, SUB-AREA A (reach length 0.95km)  
 16, existing storage at football grounds on Kuhls Rd  
 Football Grounds  
 2,0,1, spillway data  
 56.5,75,1.9,0.5,0,1, one pipe of diameter 600mm.  
 10,1,55,1,0.6,-99, pipe data  
 2,2840,2.7,55,-99, elevation-storage relation  
 5,1,0.52,-99, route to addition of sub-area B  
 2,1,0.6,-99, SUB-AREA B (added to running hydrograph, reach length 0.6km)  
 3, store hydrograph  
 1,1,0.58,-99, SUB-AREA C (reach length 0.58km)  
 4, add hydrograph from C to running hydrograph  
 3, store hydrograph  
 1,1,0.55,-99, SUB-AREA D (reach length 0.55km)  
 4, add hydrograph from D to running hydrograph  
 5,1,0.58,-99, route to addition of sub-area E  
 3, store hydrograph  
 1,1,0.2,-99, SUB-AREA E (reach length 0.2km)  
 4, add hydrograph from E to running hydrograph  
 5,1,0.47,-99, route to addition of sub-area F  
 2,4,0.15,-99, SUB-AREA F (added to running hydrograph, reach length 0.15km)  
 3, store hydrograph  
 1,1,0.57,-99, SUB-AREA G (reach length 0.57km)  
 4, add hydrograph from G to running hydrograph  
 3, store hydrograph  
 1,1,0.8,-99, SUB-AREA H (reach length 0.8km)  
 3, store hydrograph  
 1,1,0.33,-99, SUB-AREA I (added to running hydrograph, reach length 0.33km)  
 4, add hydrograph from I to running hydrograph  
 5,1,0.2,-99, route to sub-area J input  
 2,1,0.37,-99, SUB-AREA J (added to running hydrograph, reach length 0.37km)  
 4, add hydrograph from J to running hydrograph  
 7  
 Wetland Inflow  
 16.1, Current Wetland (print h/g's)  
 Current Wetland  
 C pipe and weir flag, s'way coeff, pipe ent loss coeff, bend loss coeff  
 2,1.9,0.5,0,-99  
 C flag for storage-elevation formula, formula parameters  
 2,2840,2.7,50,-99  
 0, end of control vector  
 C sub area data  
 1.42,1,0.5,0.4,1.5,0.1,0.5,0.75,0.4,0.2,-99  
 1,0.38,0.38,0.1,0,0.1,0,0.1,0,0,0,-99,

Figure D.3: Catchment File used in RORB Model for Current Conditions

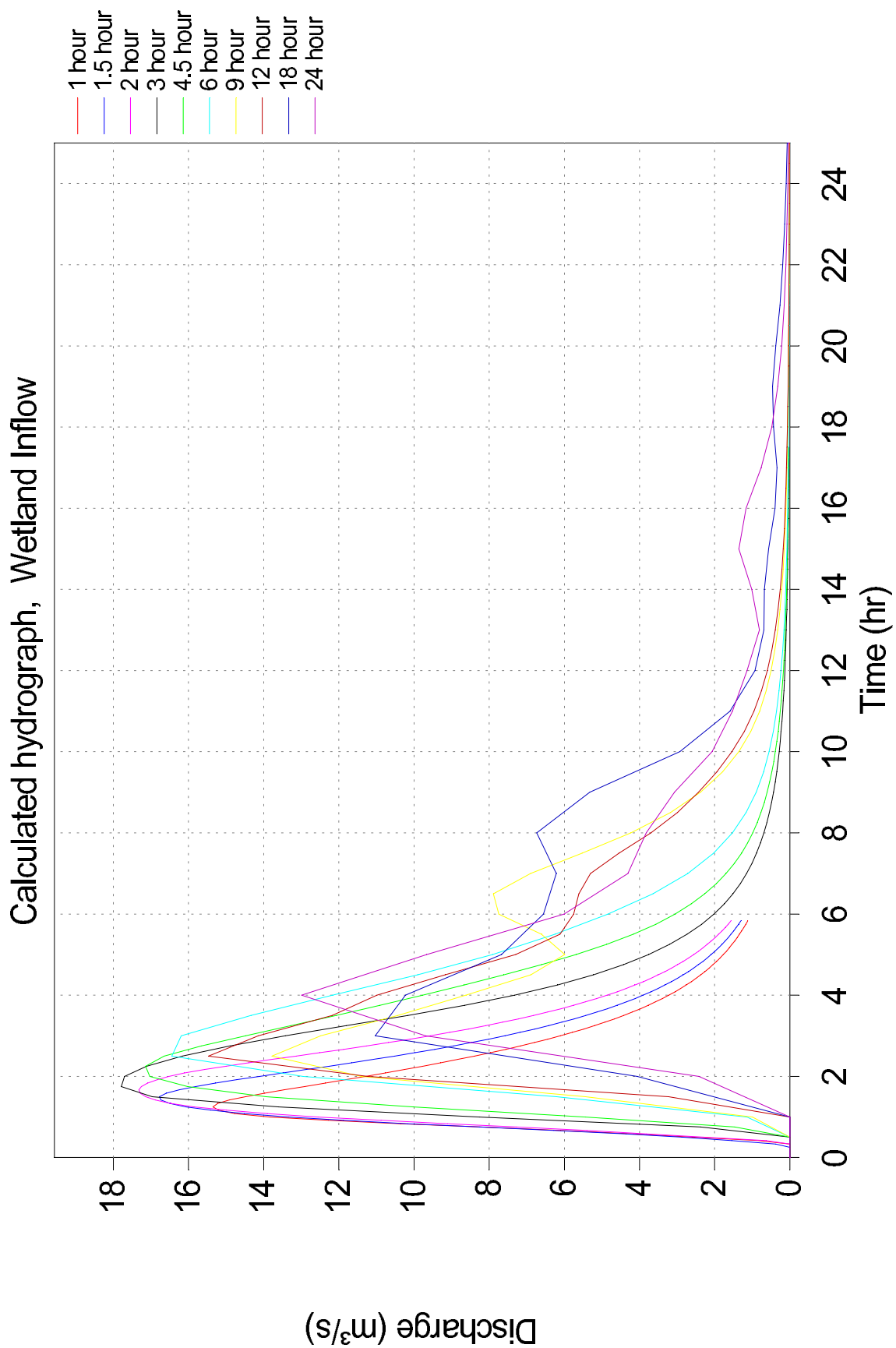


Figure D.4: Storm Duration Plot for 2yr Design Storm

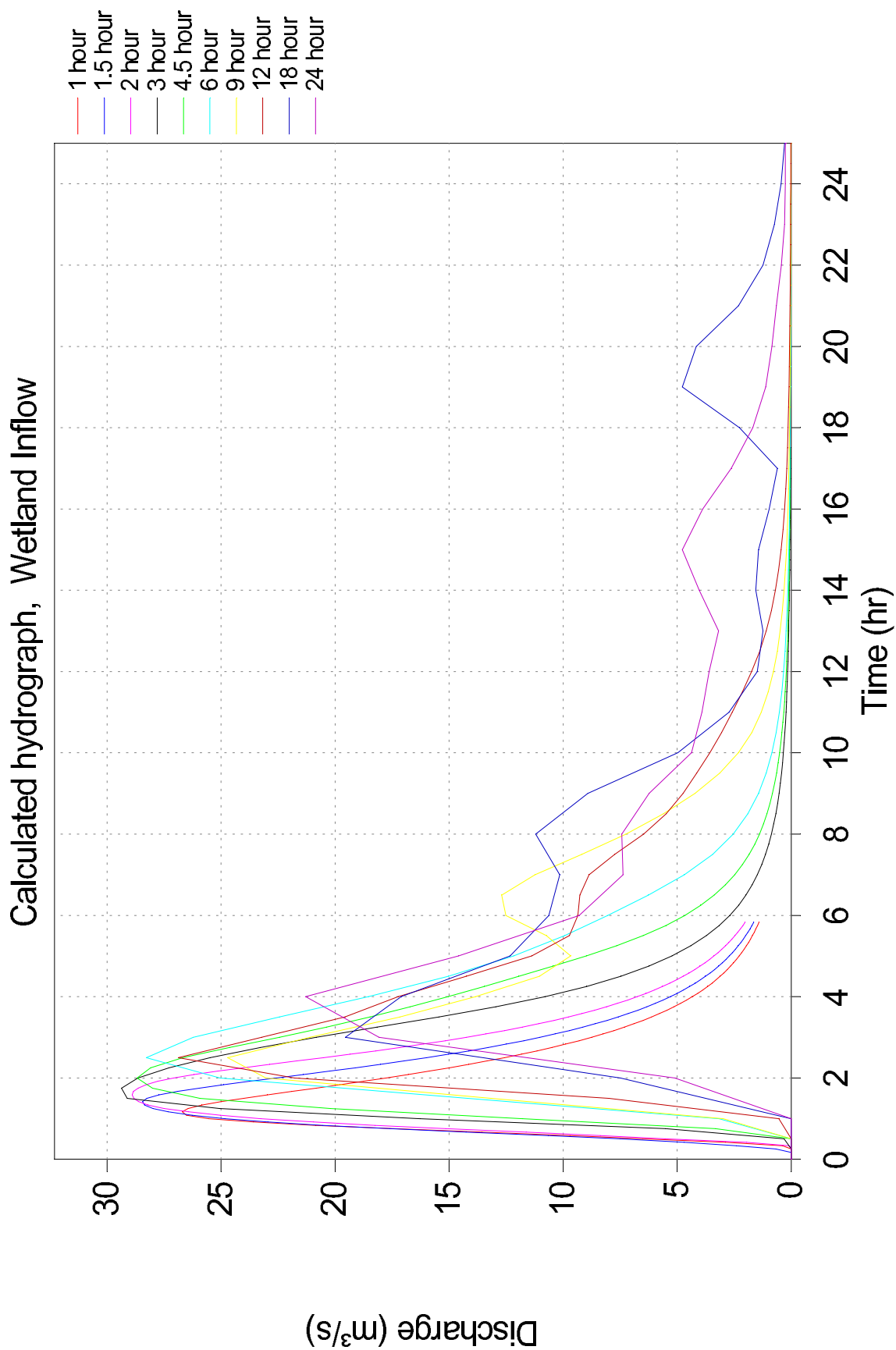


Figure D.5: Storm Duration Plot for 10yr Design Storm

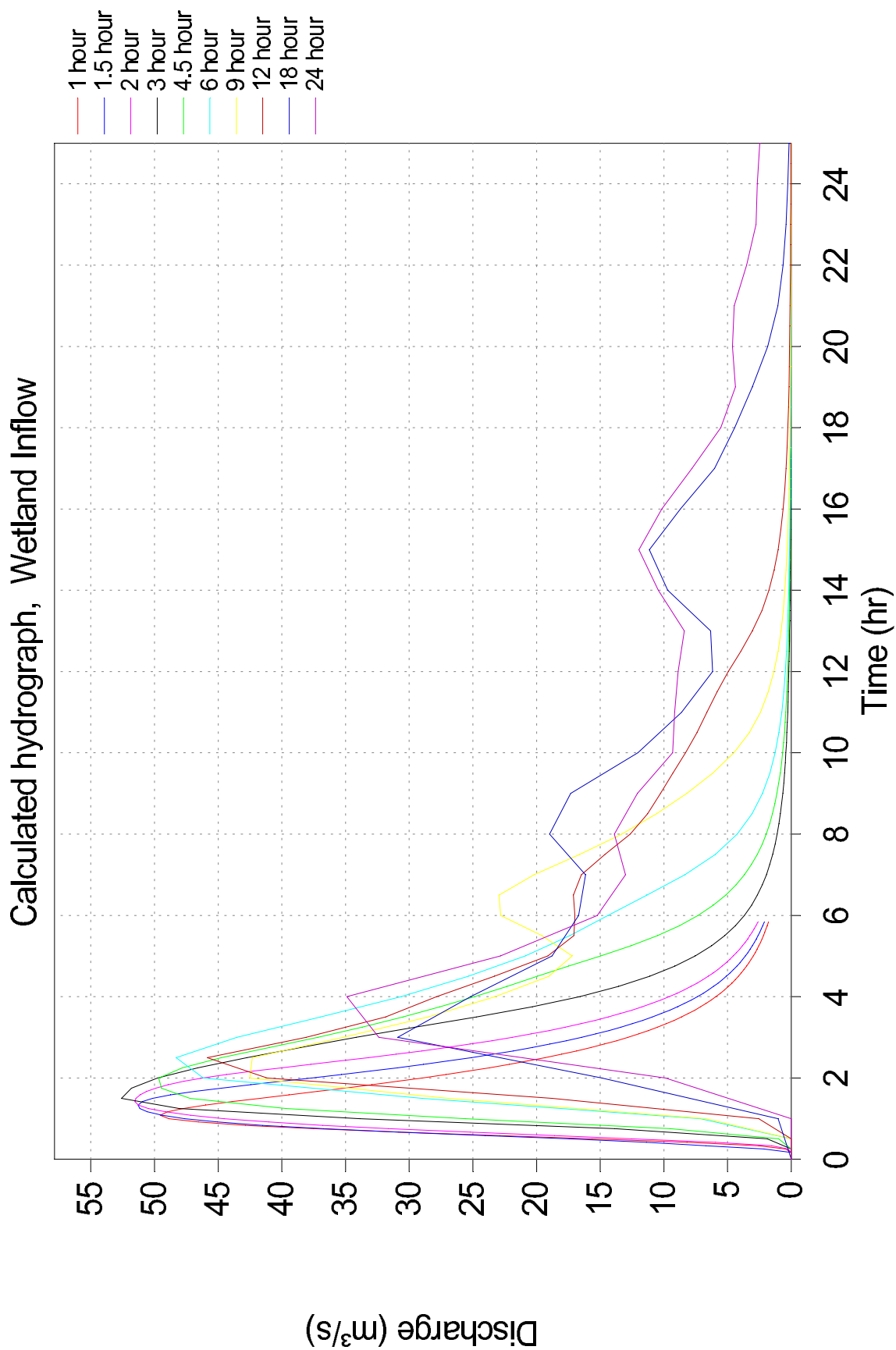


Figure D.6: Storm Duration Plot for 100yr Design Storm

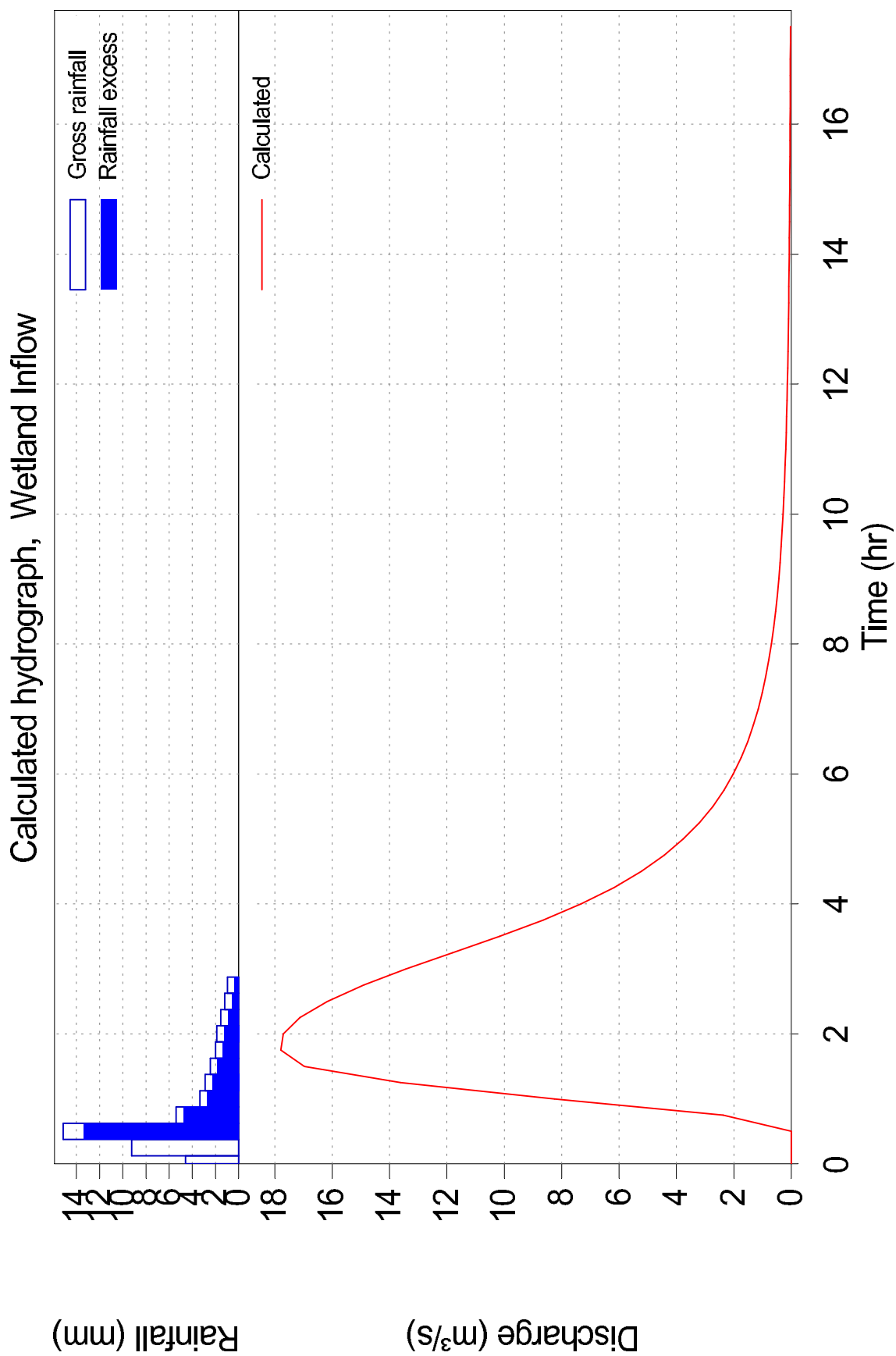


Figure D.7: Runoff Hydrograph for a 2yr Design Storm assuming Natural Conditions

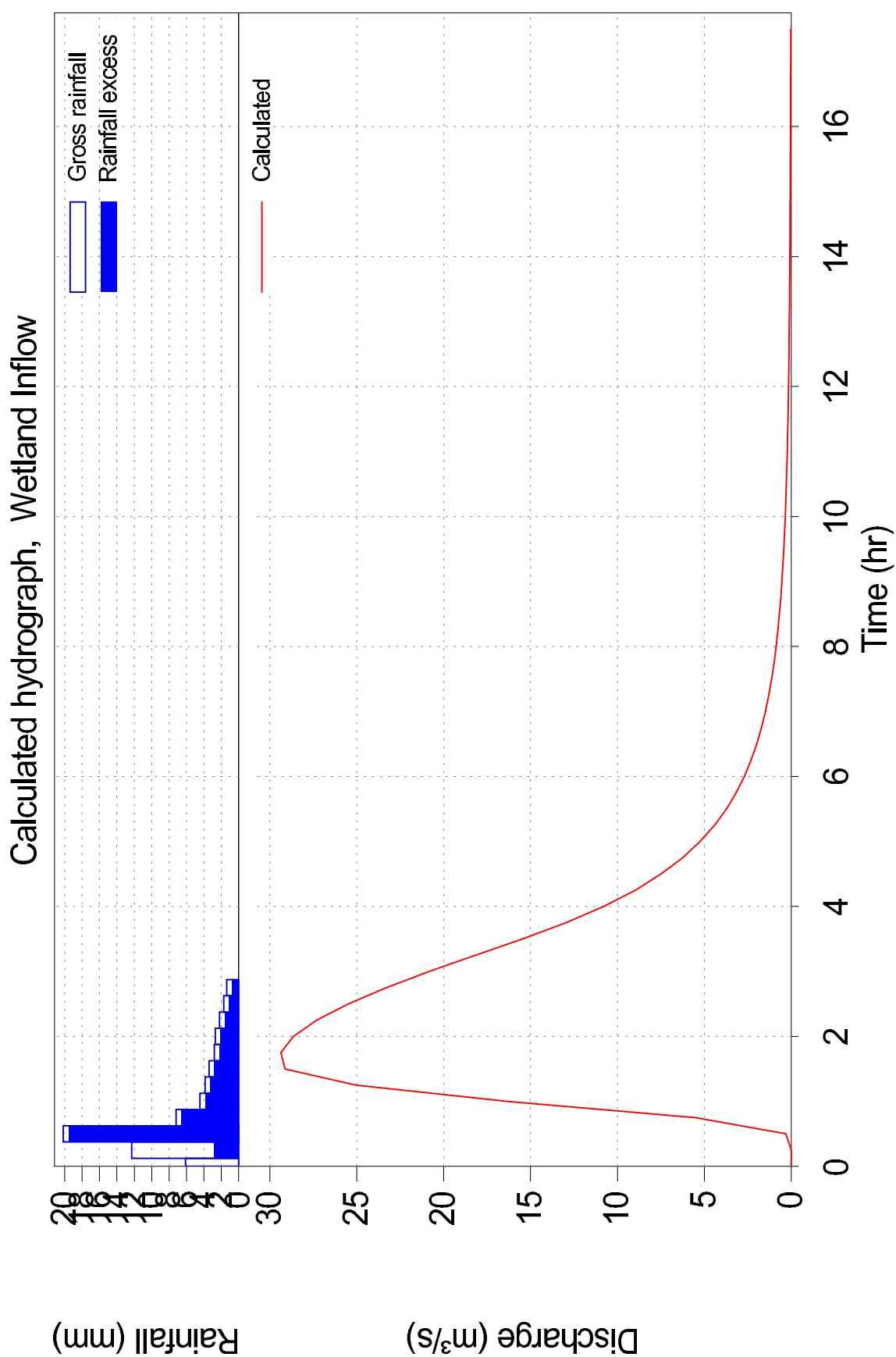


Figure D.8: Runoff Hydrograph for a 10yr Design Storm assuming Natural Conditions

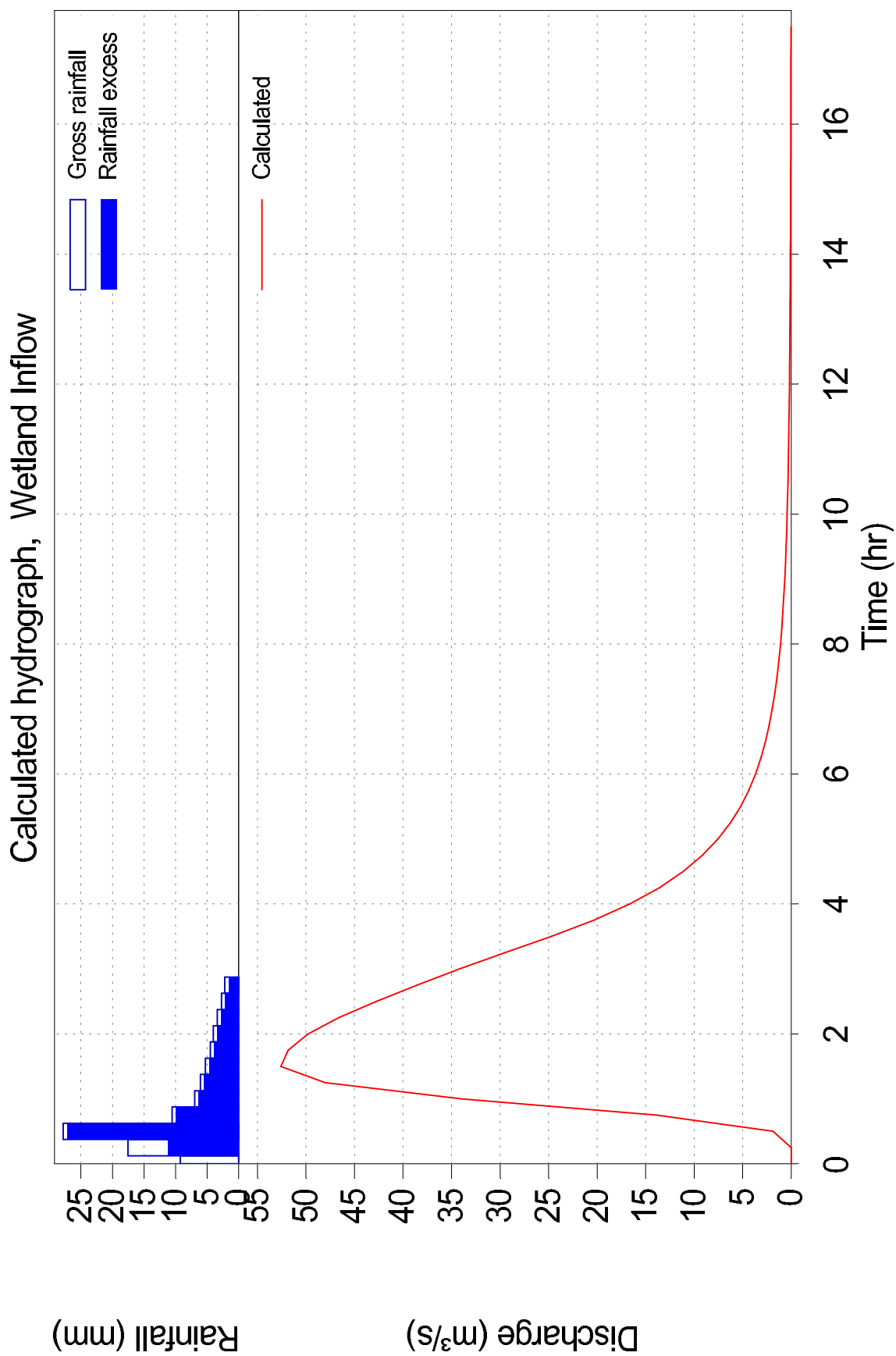


Figure D.9: Runoff Hydrograph for a 100yr Design Storm assuming Natural Conditions

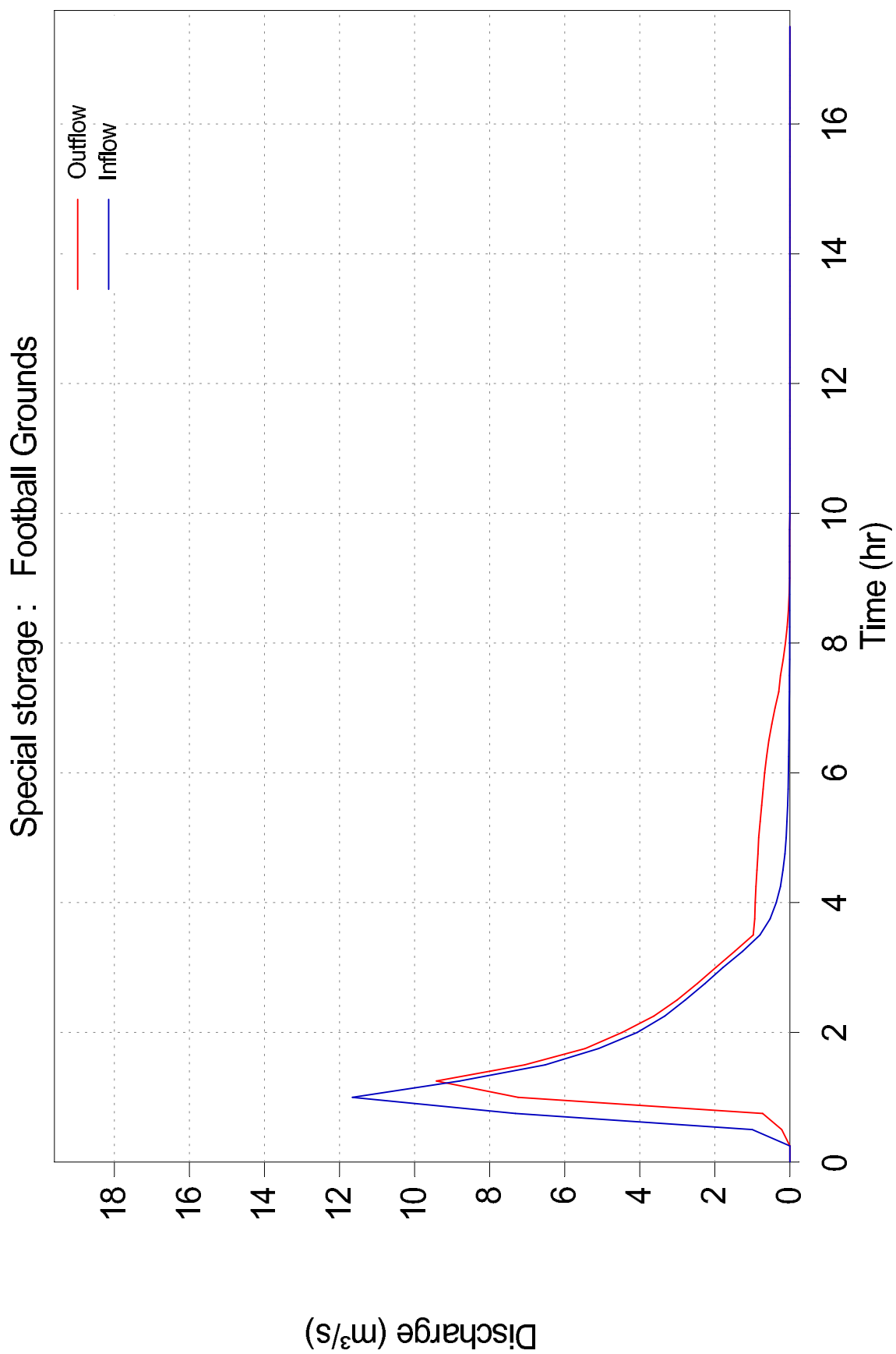


Figure D.10: Inflow and Outflow at Detention Basin for 2yr Design Storm



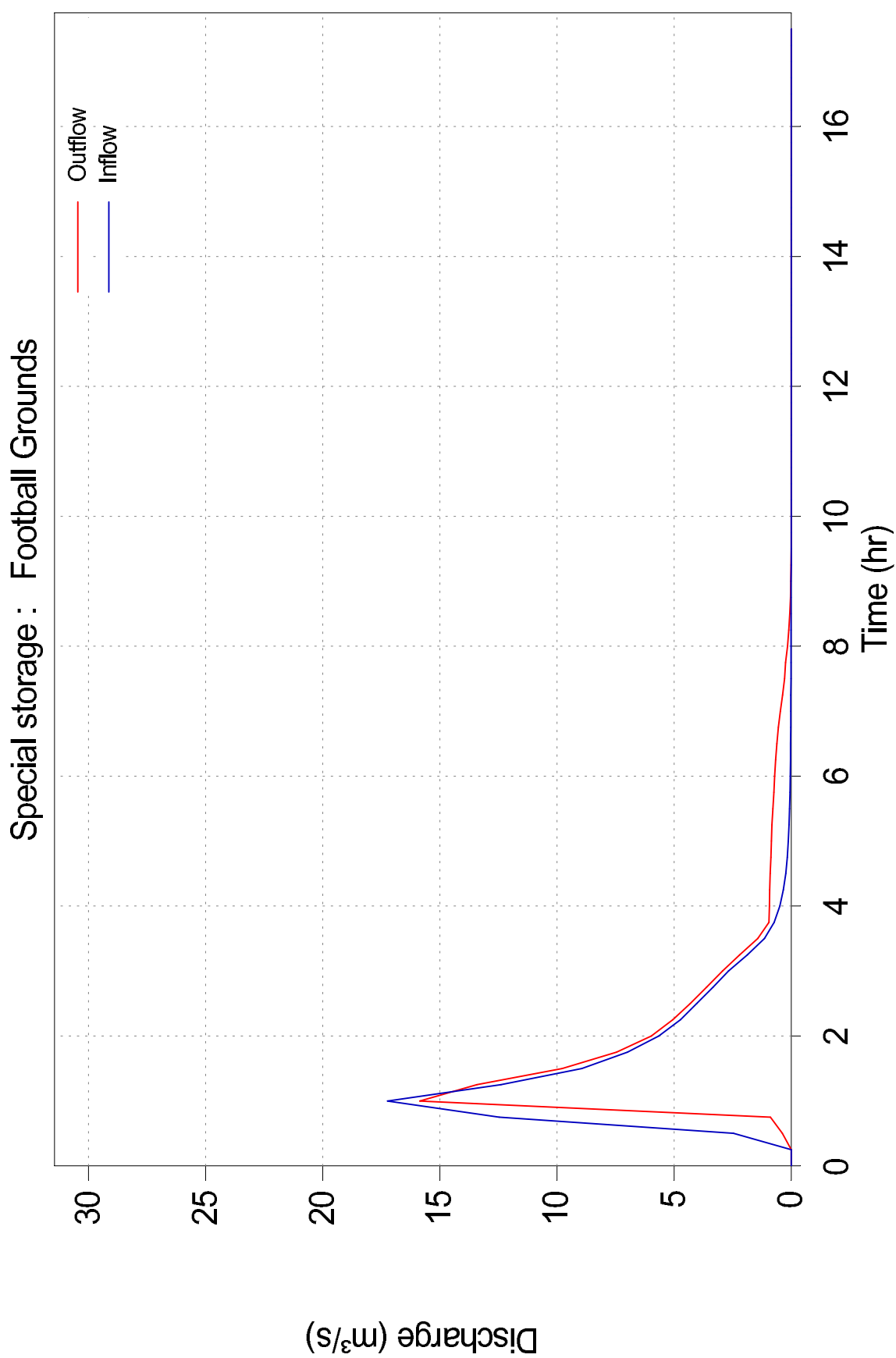


Figure D.11: Inflow and Outflow at Detention Basin for 10yr Design Storm

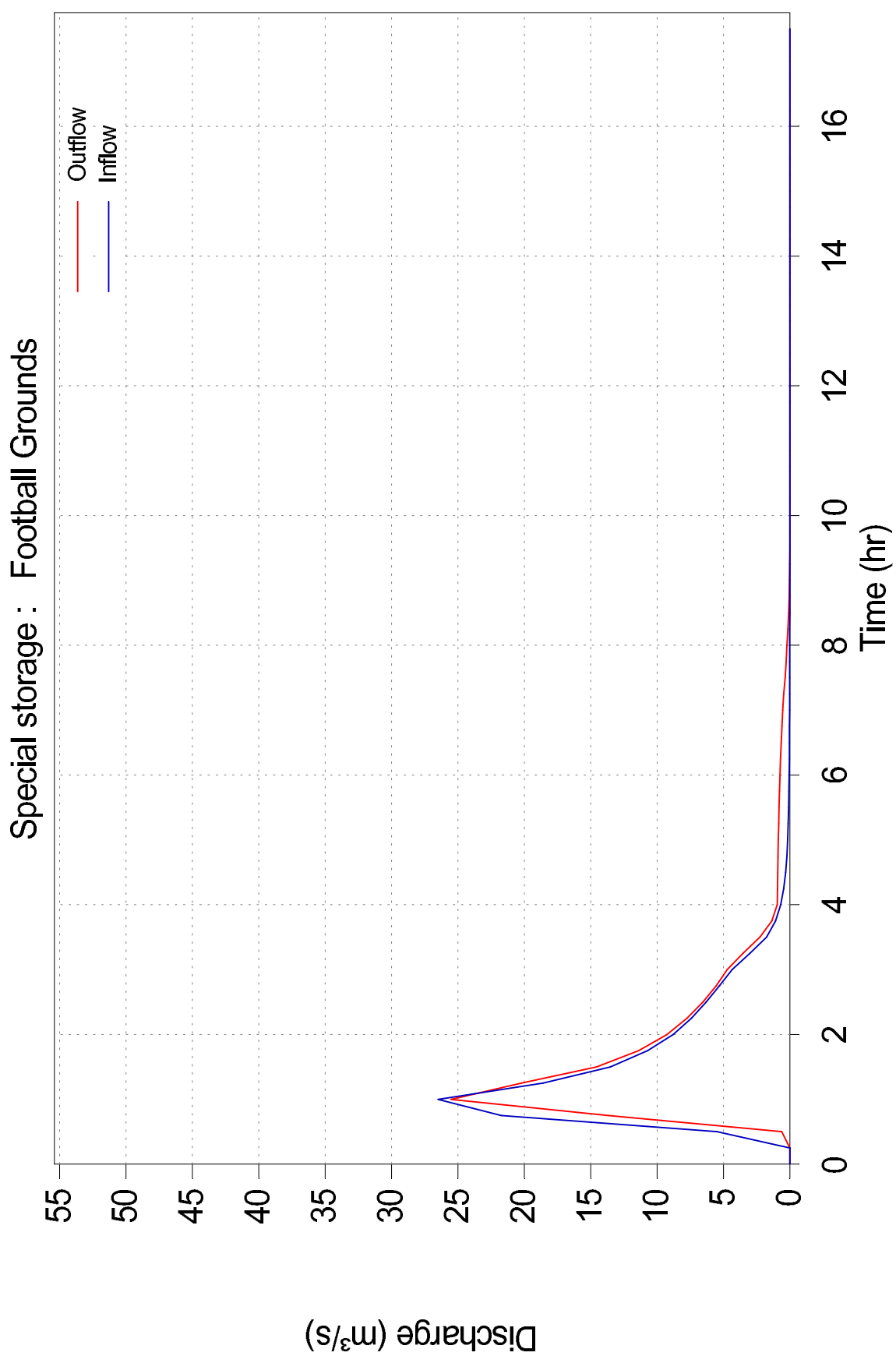


Figure D.12: Inflow and Outflow at Detention Basin for 100yr Design Storm

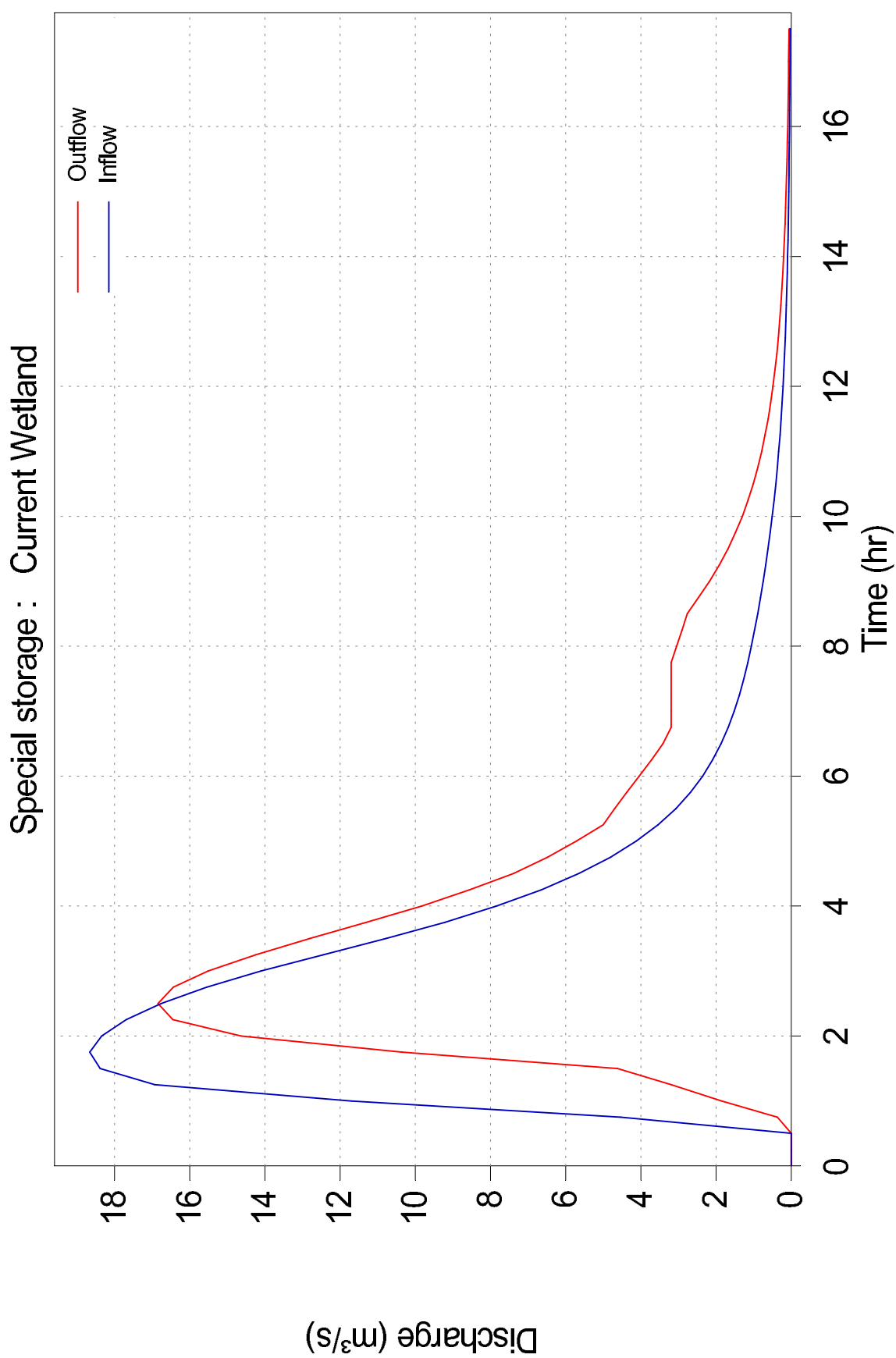


Figure D.13: Current Inflow and Outflow at Wetland for 2yr Design Storm

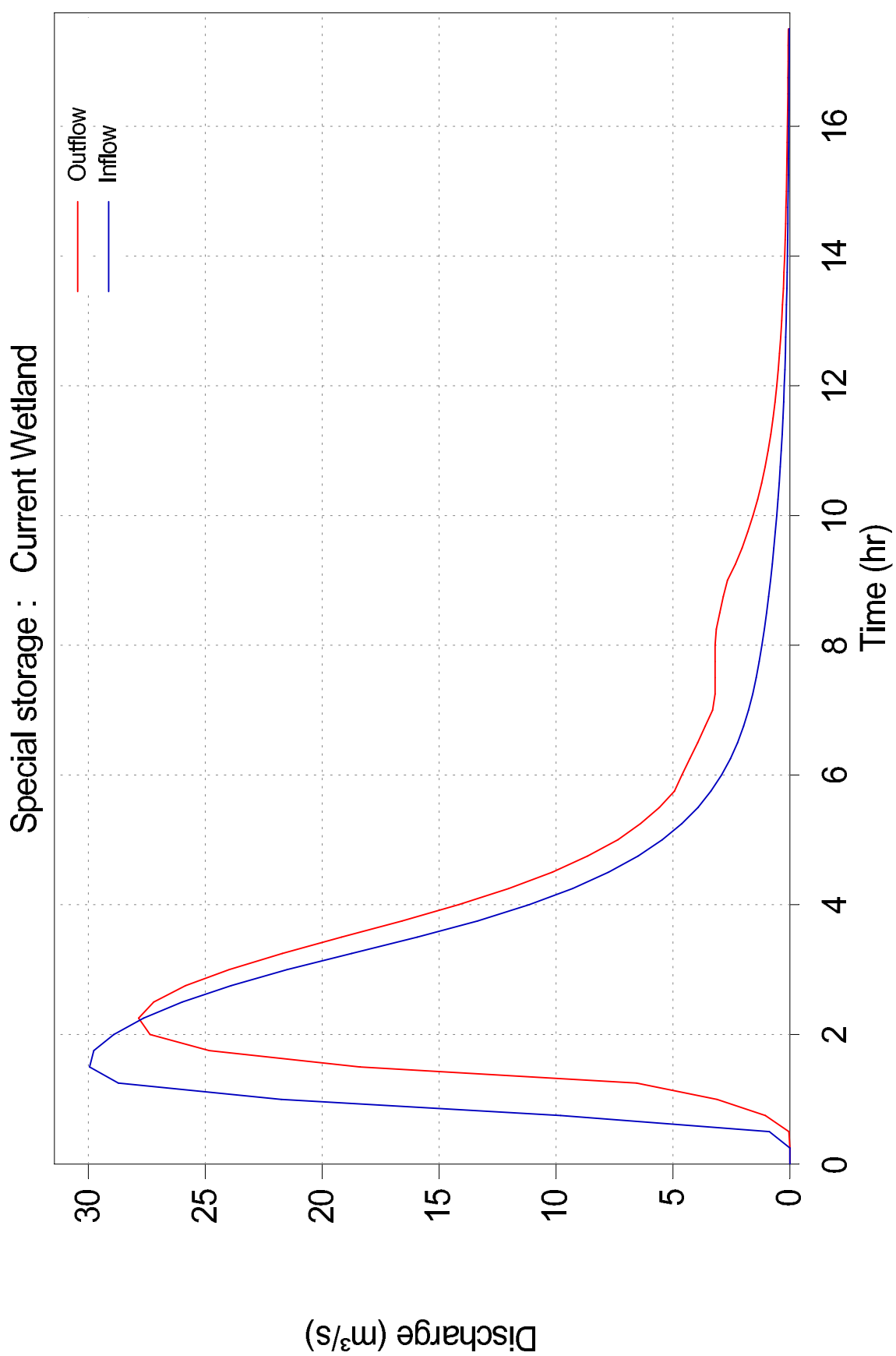


Figure D.14: Current Inflow and Outflow at Wetland for 10yr Design Storm

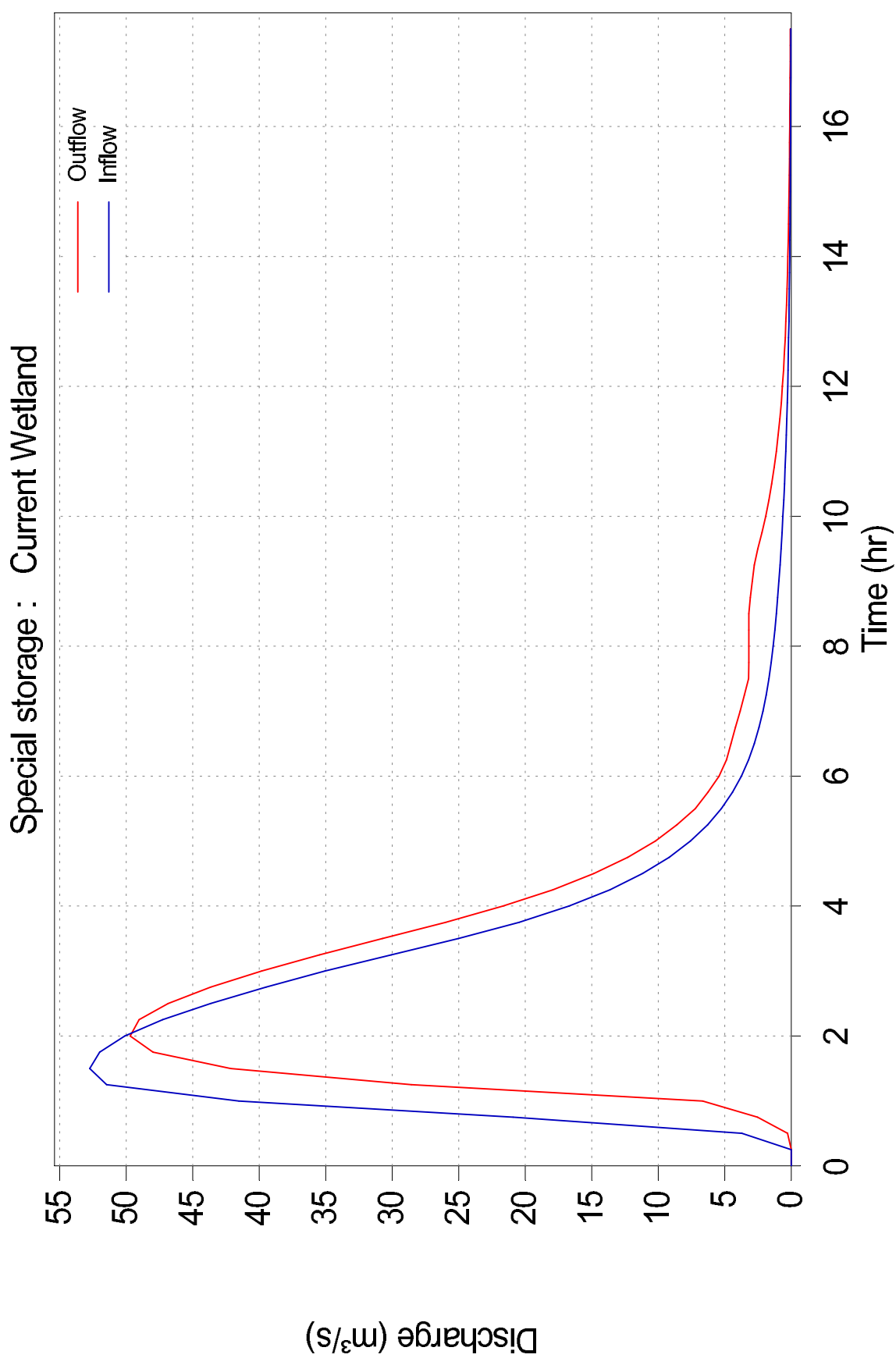


Figure D.15: Current Inflow and Outflow at Wetland for 100yr Design Storm