THE DEVELOPMENT OF A RAINFALL-RUNOFF-ROUTING (RRR) MODEL

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Abstract

Most mathematical models used in Australia to simulate runoff events from catchments were developed in the 1960s and 1970s. Models in use include the ILSAX model for urban catchments, and runoff routing models such as RORB, RAFTS and WBNM for both urban and rural catchments.

Research in the past decades has been generally directed towards the calibration and determination of regional parameters without review of the basic structure of the models. There has been limited success in the development of generalised parameters, with no consistent factors being found which govern catchment response apart from the length of the main stream within the catchment, and average annual rainfall.

This study commences with an investigation into intrinsic links between the runoff routing models. A relationship between RORB and RAFTS is determined but the relationship does not apply to RAFTS models having more than one node or sub-area. It is shown that the cause is the non-linearity of the model storages affecting the total storage and thus storage lag in the model as the number of nodes or sub-areas changes. Examination of other runoff routing models reveals that all the runoff routing models have similar problems. RORB, RAFTS and WBNM are not internally consistent and regional relationships will give appropriate results only if applied to a model having the same number of sub-areas as the model used to determine the relationship.

It is suggested that the limited success in deriving generalised relationships for storage parameters arises because they are capable of modelling only one runoff process. Hydrologists are aware that a continuum of processes occurs, for which different responses are likely. The continuum of processes is however generally dominated by one process for an individual catchment. Present model usage has favoured this type of catchment.

A new model (named the Rainfall Runoff Routing or RRR model) is developed to overcome the limitations of internal consistency and the single runoff process. The application of the new model is verified on a range of catchments in South Australia, New South Wales and the Northern Territory, and the model is applied successfully to two catchments having mixed urban and rural land use. The model is also applied to a group of catchments in the Mount Lofty Ranges, and generalised

parameter values found. The storage lag due to hillside processes appears to be related to the water holding capacity and the depth of the soil within the catchment.

Three identified processes were found to occur during runoff events, namely baseflow, slow and fast runoff. The climatic zone in which the catchment is situated, the initial state of the catchment and the magnitude of the rainfall event can all influence the processes that occur in a catchment.

It is concluded that the RRR model with these three processes being modelled will provide more consistent regional storage parameters than other runoff routing models.

STATEMENT

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

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

DAVID KEMP
29/07/02

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SYMBOLS AND ABBREVIATIONS

α	In WBNM the ratio of interbasin lag to ordered basin lag
A	catchment area (km^2)
A	channel cross section area (m^2)
Ad	area of downstream sub-catchment of a catchment having two sub-catchments (km^2)
A_i	area of sub-catchment i (km^2)
A_r	channel area (m^2)
AMC	Antecedent Moisture Condition
ARBM	Australian Representative Basins Model
Au	area of upstream sub-catchment of a catchment having two sub-catchments (km^2)
b	exponent in the relationship $K=aA^b$
B	storage delay time coefficient
B	width of the catchment element (m)
BFI	baseflow index
Bi	impervious area B value
Bp	pervious area B value
BS	moisture in the baseflow store (mm)
BX	a calibration factor in the RAFTS model
c	a catchment lag parameter, equal to RORB k_c/d_{av}
cd2	number of type 2 conceptual storages in the RORB model
Cg	the sub-surface supply parameter in the KINDOG model
circ	catchment area / perimeter^2
CL	Continuing Loss (mm/hr)
c_p	catchment characteristic lag parameter in the RRR model
Cr	channel conveyence coefficient in the KINDOG model
Cs	the surface supply parameter in the KINDOG model
d	the longest flow path length in a catchment (km)
d_{av}	average flow distance of the channel network (km)
d_g	depth of flow at the gutter face (mm)
d_p	depth of flow at the edge of pavement (mm)
f	soil infiltration capacity (mm/hr)

F	flow correction factor
f_c	final soil infiltration rate (mm/hr)
F_i	A factor depending on the type of reach in the RORB model
f_0	initial soil infiltration rate (mm/hr)
for	fraction of forest
GIS	Geographical Information System
GUT	gutter flow factor used in ILSAX
Hg	depth in the sub-surface store (mm)
HYDSYS	a HYDrological data storage SYStem
I	rainfall intensity (mm/hr)
I	channel inflow (m^3/s)
IBFL	a modifier of the B parameter to account for older sub-catchments
IL	initial loss (mm)
ILSAX	ILLUDAS-SA, with something extra
k	a shape factor
k	a dimensional empirical coefficient
k	sub-catchment storage delay time (hrs)
k	channel storage lag in the RRR model (hrs)
K	catchment lag (hrs)
K	channel conveyence (m^3/s)
K_B	ordered basis lag in the WBNM model
k_c	RORB storage parameter
Kd	storage lag of the downstream sub-area of a catchment having two sub-areas
K_D	dimensionless storage delay time
K_I	interbasin lag in the WBNM model
Ki	impervious area storage lag (hours)
K_i	lag of an individual sub-catchment I
KINDOG	A catchment model incorporating KINematic wave
K_M	average storage delay time
Kp	pervious area storage lag (hours)
k_p	process lag in the RRR model
k_{pi}	urban unconnected area process lag parameter
k_r	relative delay time

k_{ri}	relative delay time of storage i
KS	surface store recession constant
K_{split}	the true lag of a split catchment RAFTS model
Ku	storage lag of the upstream sub-area of a catchment having two sub-areas
k^*	k_c/dav
L	flow path length (m)
L	channel reach length (m)
Lg	gutter flow length (m)
L_i	length of channel reach represented storage i (km)
lmns	lnn / the mainstream length
lnn	length of streams having an order of one less than the outlet
Lo	overland flow length (km)
Lp	pipe flow length (m)
lrat	ratio of the largest RORB sub-catchment to the total area
LRRM	Laurenson Runoff Routing Method
m	a dimensionless exponent
medrn	median annual rainfall
minel	elevation of the catchment outlet
n	Manning's n, a measure of channel or pipe roughness
n	storage non-linearity exponent (used in RAFTS)
n	number of hydrograph ordinates
N	number of reservoirs
n_g	Manning's n of the gutter
ni	Manning's roughness for the impervious area
nn	number of streams of order one less than the outlet
NN	the number of nodes in a RAFTS model
n_p	Manning's n of the pavement
np	Manning's roughness for the pervious area
ns	number of sub-catchments upstream of the point of interest
O	channel outflow (m^3/s)
OF	an objective function used to measure the goodness of fit
P	wetted perimeter (m)
pe	ratio of mean annual rainfall to evaporation

pem	the ratio of median annual rainfall to evaporation
PERN	a modifier of the B parameter to account for catchment roughness
PEST	Parameter ESTimation program
PHI	the objective function used by PEST
PL	Proportional Loss
q	instantaneous runoff rate (m^3/sec)
Q	discharge (m^3/sec)
$Q_c(t)$	calculated hydrograph at time t (m^3/s)
q_m	total mean flow ((m^3/s)
$Q_o(t)$	observed hydrograph at time t (m^3/s)
Q_{op}	peak flow of the observed hydrograph (m^3/s)
Qp	peak flow (m^3/s)
q_{split}	the flow from one part of a split-sub-catchment RAFTS model
RAFTS	Runoff Analysis and Flow Training Simulator
RF	annual rainfall (mm)
r_i	the hydraulic radius of the ith pipe (m)
rla	RORB length over area
rlen	length of the reaches in the RORB model
rlm	RORB length over the mainstream length
rlt	RORB stream length / total stream length
r_m	the mean hydraulic radius (m)
RORB	RunOff Routing developed on a Burroughs computer
rr	relief ratio (maximum elevation - minimum elevation over main stream length)
rrd	number of raindays per year
RSWM	Regional Stormwater Drainage Model
s	storage volume (hrs x m^3/sec), used in RAFTS
s	slope (m/m)
S	slope (m/m)
S	storage (m^3)
sa	the number of sub-catchments in the RORB model
S_c	slope of catchment (%)
S_g	gutter slope (m/m)
Sg	rate of sub-surface supply (mm/hr)

S_i	the slope of the ith pipe (m/m)
So	overland flow slope (m/m)
S_o	Soil sorptivity
S_p	pipe slope (m/m)
ss	surface supply rate in the KINDOG model
SS	moinsture in the surface store (mm)
strm	stream order at the outlet
SWMM	StormWater Management Model
t	time from the start of rainfall (minutes)
t_1	lag of sub-catchment 1 (hrs)
t_2	lag of sub-catchment 2 (hrs)
t_{end}	the end time of calculations (minutes)
$t_{overland}$	overland flow time (minutes)
t_{r2}	translation time between sub-catchments (hrs)
t_{rm}	mean translation time for all sub-catchments
TRRL	Transportation and Road Research Laboratory (UK)
U	fraction of catchment urbanised
v_c	channel characteristic velocity in the RRR model (m/s)
Vd	runoff volume of the downstream sub-area of a catchment having two sub-areas (m^3)
Vu	runoff volume of the upstream sub-area of a catchment having two sub-areas (m^3)
WBNM	Watershed Bounded Network Model
y	channel flow depth (m)
y_o	original channel flow depth (m)
z	reciprocal of channel side slope (m/m)
Z_G	reciprocal of gutter cross-slope (m/m)
Z_p	reciprocal of pavement cross-slope (m/m)
γ	hillslope flow exponenent in KINDOG
ϕ	final infiltration rate (mm/hr)

What is the use of science if all it can do is complicate your view of the world? Every scientist should be trying to see the world in the simplest possible way.

Jack Cohen & Ian Stewart (1994)

1. Introduction

1.1 The Need

Mathematical models that predict flood hydrographs from catchments are extensively used in Australia. For this application a runoff routing model is most often used. The term "runoff routing" indicates that the hydrograph is calculated by some form of routing of rainfall excess (the part of rainfall that appears as stream flow) through a representation of catchment storage. Models in use include the ILSAX and DRAINS model for use on urban catchments, and models such as RORB, RAFTS and WBNM for use on both urban and rural catchments.

The RORB, RAFTS and WBNM models were first developed in Australia in the 1960s and 1970s and although they have been in wide use the basic fundamentals of the models have remained substantially unchanged for the past 20 years. Research effort has been generally directed towards calibration and derivation of regional parameters.

Many features have been added over time, so that the models now have some or most of the following features:

- Built-in design storms, based on Australian Rainfall and Runoff Book II;
- A range of rainfall loss models;
- Ability to model urban and part urban catchments;

- Flood routing in stream channels;
- Flood routing through storage reservoirs;
- Hydraulics of culvert and bridge structures; and
- The ability to divert surcharging flows that exceed the capacity of the channel or structure to other points on the catchment.

However the basis of the model, as represented by the "mathematical engine" or algorithm has not changed. An example of this is the RAFTS model, which if stripped of the user interface and many of the above features has not changed substantially since the mid 1970s.

The widespread use of software with simple data input has led to the situation where users do not have to be aware of the structure and limitations of the underlying "mathematical engine" to enable results to be achieved. This is unlike the modeller of twenty years ago, who needed to understand how the model worked in order to achieve results.

The complexity of some of the models in regard to the number of sub-areas or nodes, and the loss model used may not be warranted. Even for ungauged catchments models such as RORB and WBNM require manual catchment sub-division. A simple model containing no information on catchment layout such as a single node RAFTS model may be able to provide the translation necessary from rainfall to runoff at a particular point of interest.

Grayson and Nathan (1993) support the view that the main justification for a complex model is variability in catchment spatial attributes and/or temporal inputs. It is often doubtful that sufficient data are available to adequately represent this variability, and indeed it may not be necessary for the production of flood hydrographs. As extra information and understanding of catchment and rainfall behaviour becomes available it should be used but for now a lumped conceptual model may be appropriate.

Klemeš (1986) considered the appropriate level of complexity and form of mathematical models. He states:

"For a good mathematical model it is not good enough to work well. It must work for the right reasons. It must reflect, even if only in a simplified form the essential features of the physical prototype"

The best model is the model that captures the essence of the catchment process, with the simplest structure, and the least number of variables or parameters to determine.

Given the other great uncertainties in the hydrological process, predicted flows are usually verified by means other than modelling. It is often the case (and perhaps should always be the case) that predicted flows from hydrological models are compared with historical records, observation, and synthesised data such as regional regressions.

Storage parameter values for runoff routing models have been the subject of much investigation. In many cases regional relationships have been determined. However no clear and consistent relationships have been found between storage parameter values and physical catchment characteristics, apart from a dependence on average annual rainfall in southern and western Australian catchments (Yu, 1990 and Kemp, 1993).

One reason for the lack of success in the derivation of regional parameters may be that different processes are occurring on the catchments examined, leading to a wide range of responses that are not captured by the parameters used for regional regressions. The change in catchment response due to changes in catchment processes is what is shown by the dependence on average annual rainfall. Arid areas have lower storage parameter values because direct surface runoff occurs in preference to other processes that occur in humid catchments.

Another reason is that runoff routing models only model one process, which is assumed to be surface flow. All storage is contained within the channel system. It is generally assumed that a baseflow occurs, and this baseflow flow is separated from the total hydrograph before modelling commences. If baseflow and surface flow are not the two processes occurring in the catchment being modelled a variation in parameter value will occur that cannot be explained by physical catchment characteristics.

To improve the determination of regional relationships for model storage parameters it is necessary to include more than one runoff process in the model, so that there is certainty that the same runoff process is being compared across catchments. This will lead to regional relationships being meaningful.

More data sets are required on which to calibrate runoff routing models. Only in this way will there be a separation between errors caused by inadequacies in data and insufficiencies in models.

The need for good data sets is particularly evident in urban areas, which receive a large proportion of the money spent on drainage works in Australia. Because of technical difficulties in the measurement of flows in pipes there are only a small number of catchments for which data is available for model calibration.

In recognition of this a catchment at Glenelg in suburban Adelaide was chosen for monitoring by a group representing state and local government, universities and private industry. It was the availability of data from this catchment, plus the need to determine local parameters for use with the RAFTS model that was the driving force behind research into the various models.

As the research progressed, the limitations of some models were exposed. This led to the detailed examination of the structure of runoff routing models, and the development of a new model structure to significantly improve the performance of runoff routing models. The new model structure is able to be used without catchment sub-division, and can model several runoff processes. The new model is applied to a range of urban and rural catchments in Australia, to show a wide range of applicability.

In addition the new model is calibrated on further catchments in the Mount Lofty Ranges, to determine relationships between model parameters and catchment characteristics. This has given an insight into the main factors that determine different catchment behaviours and the processes that operate within them.

1.2 Objectives

The objectives of the research are as follows:

- To confirm that the two widely used rainfall runoff models, RAFTS and ILSAX are applicable to South Australian catchments. In the case of the RAFTS model, the confirmation was to be or both urban and rural catchments;
- To derive generalised parameters for the two models suitable for use on South Australian catchments. In the case of the RAFTS model, parameters are required for both urban and rural catchments;
- To examine the model structures of the RORB, RAFTS, WBNM and ILSAX models to determine any limitations caused by the model structure;
- To propose a new model to predict event flows in both urban and rural catchments that can be applied without catchment sub-division, and can model several runoff processes;
- To provide sufficient calibration and verification of the model to provide evidence of its general application on Australian catchments, and particularly South Australian catchments; and
- By examining the relationships between the model's parameters and catchment physical characteristics, determine the main factors that affect catchment response time and rainfall losses.

1.3 Methodology

The methodology adopted was to:

- Undertake literature research on hydrological modelling to determine trends that may help in the development of a new model;
- Examine the main runoff routing models in general use in Australia, namely RORB, RAFTS, WBNM and ILSAX, to confirm their structure and identify any inadequacies;
- Determine the links between the models. Since all the models perform the same basic function, that is, the representation of the runoff process, it could be expected that links between the parameters of the models could be found;
- Model storm events on a range of rural catchments previously modelled by RORB with a simple single node RAFTS model to confirm the relationship between these two models;
- Model storm events on three urban or partly urban catchments in Adelaide using both the ILSAX and single node RAFTS models to both confirm the link between the models and the applicability of the models on South Australian catchments;

- Investigate the effect of the number of sub-areas modelled and the internal consistency of the models;
- Determine the structure for a new model that can be applied without catchment sub-division, and can model several runoff processes;
- Apply the new model to Australian catchments, both urban and rural catchments, to determine the likely parameters for the model;
- Verify that the model functions satisfactorily by applying the model with calibrated parameters to a set of independent storm events on the same catchment;
- Compare the performance of the model to other storm runoff models;
- Determine design parameters for the model for Mount Lofty Ranges catchments, where good rainfall and gauging data exists on which to calibrate the model, and then examine the relationships between parameters values and catchment characteristics, and;
- Apply the model to several complex catchments to ensure that the model is useable, and to get experience in its application.

1.4 Content

Following on from **Chapter 1**, **Chapter 2** is a literature review of the history of storm runoff models, to determine trends in modelling.

The examination of runoff routing models commences in **Chapter 3** with a description of the structure of each of the models examined, and where applicable comment on the parameters to be used in the application of the models. The models chosen (ILSAX, RAFTS, RORB and WBNM) represent the four models most commonly in use in Australia.

Chapter 4 examines the relationship between the models, with special reference to the lag parameters. All the models are using the same series of pipe or channel storages to represent catchment behaviour. It was expected that the storage parameters for one model could be translated to the storage parameters for another. In this way published parameter values for one model could be applied to another model, thus making the most use of available knowledge of parameter values. The emphasis is on the relationships between RORB and ILSAX and RAFTS, as this thesis stemmed from a need to find appropriate parameter values to be applied when using the RAFTS model on South Australian catchments.

It is known that the number of nodes or sub-areas in runoff routing models has an effect on the predicted flows. **Chapter 5** both examines the effect in the RAFTS and WBNM models, and quantifies the effect.. Internal consistency of models is also discussed.

Chapter 6 contains the ILSAX modelling on South Australian urban catchments, and demonstrates the applicability of the model to these catchments. **Chapter 7** undertakes a similar program for the RAFTS model. However the RAFTS model is used only as a single node model, and with a linear response, as urban catchments were shown in Chapter 3 to behave in a linear fashion.

Chapter 8 discusses the limitations of runoff routing models and introduces the RRR model. This innovative model introduces the ability to model more than one process by a series of storages to produce a model that has the potential to be significantly better than other models. **Chapter 9** describes the verification of the RRR model on a range of Australian urban and rural catchments. It also compares the performance of the RRR model with other runoff routing models, including KINDOG.

Chapter 10 outlines the calibration of the RRR model on further catchments in the Mount Lofty Ranges, and examines the relationships between catchment characteristics and model parameter values.

Chapter 11 describes the application of the model in three catchments, two being the catchments of Adelaide urban creeks and the third being an extreme flood event at Olary, in outback South Australia.

The thesis is summarised in **Chapter 12**, and concludes with a statement of the findings. These include a theoretical confirmation of the effect of the number of nodes in the RAFTS model, the identification of three separate runoff processes and the implications of this to large rainfall events. The newly developed RRR model overcomes the identified limitations in existing runoff routing models, and performs better than either the traditional runoff routing models (RAFTS, RORB and WBNM) or a newer model (KINDOG) used as a direct comparison.

Mathematical descriptions of nature are not fundamental truths about the world, but models. There are good models and bad models, and what model you use depends on the purposes for which you use it and the range of phenomena that you want to understand.

Jack Cohen & Ian Stewart (1994)

2. A Review of Storm Runoff Models

2.1 Introduction

When investigating the structure and parameters of runoff routing models it is useful to consider the history of event runoff models in general. This chapter will briefly discuss the history of hydrological models, but will place emphasis on the development of the structure and parameters for the four models to be examined in more detail (ILSAX, RAFTS, RORB and WBNM).

Storm or event runoff models generally do not represent the whole of the hydrological process. It is assumed that some parts can be ignored as they have little effect in the short durations considered, and thus the model can be simplified. Some simplifications include ignoring evapotranspiration, moisture redistribution, and in many cases baseflow. The models are not physically based, rather they are conceptual models.

It is convenient to sub-divide the hydrological model into a number of conceptual components. Following this approach Ball (1992) identified four conceptual components of a catchment modelling system as:

Generation – That component of the system primarily concerned with the estimation of the input to the catchment model. In this respect it would be concerned with the spatial and temporal distribution of the available water quantity, and the available water quality constituents.

Collection – That component of the system primarily concerned with the accurate prediction of the quantity and quality of flow at the downstream point of a catchment, or sub-catchment. This component generally is considered as that component of the system that predicts that inflow to the transportation component of the system and consequently is referred to as the hydrologic component.

Transport – That component of the system where the quantity and quality of water is routed along the channels and pipes of the drainage system. Sometimes this component is referred to as the hydraulic component of the system.

Disposal – That component of the system where the runoff is discharged into the receiving waters with or without treatment to mitigate the impact of components conveyed with the runoff.

Information flow between components is unidirectional. It is possible for more than one combination of input information and information transportation by a combination of process models will produce output information that is similar to recorded information.

Also there is a concurrent flow of errors through the modelling system. For example, errors in rainfall models will propagate and grow as the information flows through the hydrologic and hydraulic process models.

2.2 Early Models – The Unit Hydrograph

The unit hydrograph, a method for estimating storm runoff, was first proposed by L.K. Sherman in 1932 (Chow, 1964), and since then has been used as a key concept. The unit hydrograph is defined as the watershed response to a unit depth of excess rainfall (ie rainfall causing direct runoff), uniformly distributed over the entire watershed and applied at a constant rate for a given period of time. In 1938, after studying watersheds in the Appalachian mountains of the United States, Snyder proposed that there was a relationship between some of the characteristics of the unit hydrograph, being peak flow, lag time, base time, and width (in units of time) at 50% and 75% of the peak flow (Chow, 1964). A significant contribution to the unit hydrograph theory was given by Clark (1945), who proposed a unit hydrograph which was the result of a combination of a pure translation routing process followed by a pure storage routing process.

Although Clark did not develop a spatially distributed analysis, the translation part of the routing is based on the time-area diagram of the watershed. The storage part consisted of routing the response of the translation through a single linear reservoir located at the watershed outlet. The detention time of the reservoir is selected in order to reproduce the falling limb of observed hydrographs.

2.3 Accounting for Spatial Variability

One focus of research in hydrological modelling has been to overcome the unit hydrograph limitation of uniform and constant rainfall, and to account for spatial variability within the catchment.

Two main strands of models were developed, those with the catchment response being modelled by a series of storages (runoff routing), and those where the movement of water through the catchment is modelled in part by a hydrodynamic response of the hillside and/or channel.

The runoff routing model only has two conceptual components in the system proposed by Ball, being the generation and collection components. The second type of model, having a hydrodynamic response introduces a transportation, or hydraulic component to the model.

A third type of model has been developed where the input to the channel or pipe system is by a simple time-area relationship, followed by routing through the channel or pipe to the next inflow point. The routing may be by simple time translation, hydrodynamic response, or storage routing. This form of model is substantially a convoluted unit hydrograph model.

2.4 Runoff Routing Models

Storage has both a delaying and attenuating effect on runoff input. As rainfall occurs across the catchment, and flow is generated both on hillsides and in channels, storage is obviously highly distributed. Runoff routing models were developed in Australia, starting with the Laurenson

Runoff Routing Model (LRRM) described by Laurenson (1964). Runoff routing models deal with the distributed nature of the storage by introducing a series of storages representing catchment sub-areas being based either on isochronal areas (Laurenson, 1964) or geomorphological areas (Laurenson, 1975, Goyen & Aitken, 1976, Boyd et al, 1979).

Runoff Routing models also generally allow for non-linear catchment response, where the response time of the catchment is not constant, but is related to the outflow from the catchment.

Laurenson (1964) specifically states that the runoff routing model proposed should account for the fact that the relationship between stream discharge and catchment storage is non-linear, but gives no evidence for this. However in a paper from the same period (Body, 1962) it can be found that there was awareness that the time base of the unit hydrograph was reduced as the peak flow from a catchment increased, in other words that non-linear behaviour is in evidence. Kulandaiswamy (1964) also presented an investigation of non-linearity of runoff and found that non-linear behaviour was demonstrated in six drainage basins.

More recently however Simas and Hawker (1998) investigated the lag time of small watersheds in the U.S.A. Lag times were evaluated from runoff data in over 50,000 events on 168 small catchments. It was found that only 5 out of the 168 catchments had a coefficient of determination (r^2) of greater than 0.5 for a relationship between lag time and mean total discharge. This finding does not support non-linear catchment behaviour.

Laurenson's model can be considered to be the founding model of runoff routing models in Australia. It divided the catchment into ten sub-areas each with its rainfall excess being routed to the next downstream sub-area. The sub-areas were based for convenience on equal travel time from the outlet. Variations were tried, but were less successful. These included linear storages, five sub-areas instead of ten and using sub-areas bounded by major watershed lines. Laurenson stated that whereas the delineation of sub-areas by major watershed lines was less successful, further trials would be carried out.

The first form of Laurenson's model (ten sub-areas, based on equal travel time) formed the basis of the current RAFTS model. The second (sub-areas bounded by watershed lines) is the basis for the RORB and the WBNM model.

Aitken (1975) adapted the Laurenson model for use on urban catchments, and derived generalised relationships for the storage parameters, which are still used in the RAFTS model.

RAFTS makes use of Laurenson's model which was primarily aimed at rural catchments but modified by Aitken. It uses the same ten sub-areas as Laurenson, but uses these as parts of a total catchment, connecting the sub-catchments by channel or pipe elements to build up a complete catchment model that allows for spatial variability.

The introduction of sub-catchments to the model introduces a transportation, or hydraulic component to the model.

The RAFTS model also allows the user to split the pervious and impervious portions of the sub-catchment, and have different loss and storage parameters for each.

Hood (1991) found that when applying the RAFTS model to ACT catchments the choice of model type (with or without the impervious area split) and the number of sub-catchments made a difference to the predicted flow. Hood & Daniell (1993) found that peak flow could be under-estimated by up to 30% on ACT catchments by using Aitken's relationship. Cupitt (1992) applied RAFTS to the 1910ha catchment of Winding Creek in Newcastle and found that for the three events modelled it was necessary to multiply Aitken's generalised storage parameter by a factor of 7.0, 17.0 and 1.97 respectively.

Goyen et al (1991) described the application of the RAFTS model to three case studies, both urban and rural and reported satisfactory results.

The RORB model was first released as RORT in December 1975, but has not changed in structure since. It makes use of sub-areas bounded by watershed divides, with the storage relationship for each sub-area storage being based on an overall catchment storage parameter (k_c), and a non-linearity exponent (m).

The parameters kc and m are generally considered to be independent of the event being modelled (Laurenson & Mein, 1990, Weeks, 1980, McMahon & Muller, 1983, 1985,1986). This assumption is not supported by Wong (1989) who investigated three catchments and obtained two different values of k_c. Wong related the different values of k_c to in-bank and overbank flow conditions. Bates et al (1993) tested Wong's hypothesis on five catchments but did not find any statistically significant variation in model parameters with event magnitude.

Calibration strategies have been discussed by Weeks (1980), McMahon & Muller (1983), Bates et al (1991), Kuczera (1991) and Hill et al (1993). The main problem with the calibration of the storage parameters is that m and k_c are interdependent and increasing either leads to a decrease in predicted peak flow. Weeks (1980) proposed a method of parameter interaction curves, a method whereby the best pair of m and k_c can be chosen from a range of calibrated storm events. Hill et al (1993) proposed a sensitivity method of determining the best parameter values. The parameters are chosen to minimise the average error in all events examined. Kuczera (1991) developed a Bayesian methodology to evaluate the parameters for non-linear models. The method uses ordinary least squares as the objective function. Stochastic errors of auto-correlation (correlation between a residual and the residual(s) preceding it) and heteroscedasticy (residual variance not being independent of other model inputs, for example flow magnitude) are allowed for in the methodology. The methodology was criticised by Bates et al (1991) because continuity was not retained.

Because of the difficulties of parameter interaction, and the application of parameter interaction curves many users adopt m=0.8 (Dyer, 1994) unless there are strong indications to the contrary. The value of 0.8 is also recommended by Australian Rainfall & Runoff (I.E.Aust., 1987).

There have been many regional relationships derived for k_c for rural catchments, including Weeks & Stewart (1978), Monash University (1979), Morris (1982), Sobinoff et al (1983), Flavell et al (1983), Flavell (1983), Hairsine et al (1983), Hansen et al (1986), Weeks (1986), Maguire et al (1986), Walsh (1993), and Kemp (1993). Most authors find a strong relationship between k_c and catchment area or main stream length.

This is not surprising, given that by the very nature of the structure of the model k_c is dependent on the catchment boundary, in other words the catchment area and main stream length. McMahon & Muller (1983) presented an argument that for comparing or transposing RORB parameters it is necessary that a boundary independent parameter, k_c/d_{av} be used, where d_{av} is the average flow distance in the catchment. Yu & Ford (1989) also discussed the boundary dependence of k_c and determined that k_c/d_{av} was independent of the catchment boundary. Following this Yu (1990) investigated regional relationships based on k_c/d_{av}. Dyer (1994) also produced a set of regional relationships for k_c/d_{av}.

Pearse et al (2002) found that the logarithms of k_c/d_{av} are normally distributed, and recommended that the mean value be used on ungauged catchments.

Crouch & Mein (1978) applied the RORB model to three urban catchments and derived a relationship for k_c in terms of catchment area, fraction impervious, and slope. The relationship chosen was similar to that of Aitken (1975).

Boyd developed another runoff routing model where the sub-catchments are bounded by watershed lines (Boyd et al, 1979). The Watershed Bounded Network Model (WBNM) allows for two different types of sub-catchments, the first having no inflow across the sub-catchment boundary and the second receiving inflow from other sub-catchments.

Boyd (1983) compared the performance of WBNM and RORB by applying them to five catchments in eastern New South Wales and found similar levels of accuracy. Sobinoff et al (1983) determined parameter values for 21 catchments in New South Wales. It was concluded that all runoff routing models provide similar accuracy.

2.5 Hydrodynamic Models

The assumption of linearity in hydrological models implies that water velocities throughout the catchment remain constant for the whole range of flows, in other words steady-state water velocities. In fact, there is a gradual increase in depth of flow at any point in the catchment and the depth of flow increases down the catchment. Hydrodynamic models account for this. The kinematic wave method forms the basis of many hydrological models, and accounts for the factors in a simplified manner. These models are useful where inertial and pressure forces are not important, that is, when the frictional resistance balances the gravitational force of the flow. They are thus useful where channel slopes are steep and backwater effects are negligible.

When pressure forces become important but inertial forces remain unimportant a diffusion wave model is applicable. When both inertial and pressure forces are important, such as mild slope rivers then a dynamic wave routing method is required. More recent models can deal with dynamic wave routing, two dimensional modelling, with the catchment broken into individual elements, and the splitting of the model to represent more than one response regime. The latter feature has been included as it is recognised that catchments generally have a slow response (base or subsurface flow) and a fast response (surface flow). Another approach is to represent the two dimensional nature of the catchment by a series of one-dimensional stream tubes, as developed by Moore & Grayson (1991) and Sun (1996).

Application of the kinematic wave routing procedure to catchment modelling has resulted in a range of hydrological models. Ishihara (1964) used a simple plane rectangle as his model, while Wooding (1965) added a V-shaped channel to two such planes, and Harley et al (1970) combined the overland flow planes and stream modules in a branching network. In Australia Field and Williams (1983, 1985) described a model which routes flows down channels using a kinematic wave procedure. Lateral inflow to the channels is derived by routing excess rainfall through non-linear storages.

Mesa and Mifflin (1986), Naden (1992) and Troch et al. (1994) presented similar methodologies to account for spatial variability when determining the watershed response. The catchment response is calculated as the convolution of a channel network response and a hillside response.

To calculate the network response, Mesa and Mifflin (1986) use the solution of the advection-dispersion equation, weighted according to the normalised width function of the network. In their paper, the normalised width function is defined as the number of channels at a given distance to the outlet, divided by the total length of all channels in the network. For the hillside response, Mesa and Mifflin suggest a double travel time function, related to fast and slow flow, in the form of two isosceles triangles. The two functions are weighted, according to the probability that a water drop would take either path to the channel system, and added to give the final hillside response. From the physical viewpoint, fast and slow hillside responses are related to surface and subsurface flow respectively. Their model was tested in a 1.24 km^2 sub-basin of the Goodwin Creek watershed in Mississippi. For the stream network, an average velocity of 1 m/s and a dispersion coefficient of 9.06m^2/s were found. For the hillside response, the average velocities of the fast and slow components were 0.25 m/s and 0.0046 m/s respectively, and the fraction of the slow flow was estimated to be 90% of the total hillside response.

For the network response, Naden (1992) also suggests the solution of the advection-dispersion equation, but weighted by a standardised width function of the network. In her paper, the standardised width function is defined as the number of channels at a given distance to the outlet, divided by the total number of channels in the network. Naden also recommends an additional weighting of the width function by the excess rainfall spatial distribution. There is however no given specific methodology to determine the hillside response, and the one used in the paper "was selected by eye" as a single peak, reflecting the quick response, followed by an exponentially decaying curve for the slow component. For the case of the River Thames at Cookham in United Kingdom, a stream flow velocity of 0.6 m/s and dispersion parameter of 1 m^2/s were found. Additionally, because of the slow component of the hillside response, which yields about 80% of the flow volume, the rainfall spatial variability is smoothed out resulting in almost identical watershed responses for different rainfall spatial patterns. The ratio of the average velocities of the fast and slow components was found to be around 20.

As part of a case study on a small basin in the Appalachian Mountains Troch et al. (1994) proposed the same stream network response as Mesa and Mifflin (1986). However, for the hillside response they suggested a function given by the solution to the advection-dispersion equation, applied this time to the overland flow, and weighted according to a normalised hillside function. The normalised hillside function is interpreted as the probability density function of runoff generated at a given overland flow distance from the channel network. Contrary to Mesa and Mifflin's and Naden's hillside response functions, Troch et al do not account for the slow component.

Another interesting approach to model the fast and slow responses of a catchment is presented by Littlewood and Jakeman (1992, 1994). In their model, the watershed is idealised as two linear storage systems in parallel, representing the surface and the subsurface water systems. The surface system is faster and affects mainly the rising limb of the resulting hydrograph, while the subsurface system is slow and determines the tail of the response.

An example of a fully two dimensional model is CASC2D (Ogden, 1998). It is a fully unsteady, physically based, distributed parameter, raster (square-grid), two dimensional, infiltration excess hydrological model for simulating the hydrological response of a watershed to an input rainfall field. The model will accept spatially non-uniform rainfall. Rainfall interception and soil infiltration are modelled. However the soil is considered to be infinitely deep, and is thus not recommended for catchments where groundwater (baseflow) plays a large role in runoff production. Overland flow routing is by an explicit, two dimensional diffusive wave scheme, using the Manning equation to calculate overland flow velocities. Two options are available for channel flow routing, being an explicit, one dimensional diffusive wave formulation, or a method that solves the full one dimensional equations of motion using the Priessmann 4 point implicit scheme (Holly et al, 1990, Ogden, 1994).

The model has been applied to a number of catchments, including an analysis of the Fort Collins flood of 1997 (Ogden et al, 2000). The flood was produced by a rainfall of over 200mm, and caused over $100m damage to the Colorado State University alone. Molner and Julien (2000) produced an analysis of the effect of grid size on the CASC2D model. It was found that the model could be used to simulate observed peak discharges and time to peak, provided that the model is calibrated at the same grid size as is used in the prediction.

For application on urban areas the SWMM (Storm Water Management Model) was developed as a US EPA funded project, and was released in 1971 (EPA, 1971). SWMM uses a kinematic wave to model overland flow to the inlet to the pipe or channel system, and hydrodynamic routing methods within the pipe or channels.

2.6 Convoluted Unit Hydrograph Models

These models are an extension of the unit hydrograph model, two early examples being HEC-1 (US Army Corps of Engineers, 1981) and RRL (UK Transport & Road Research Laboratory, 1976).

The HEC-1 model includes a land surface runoff component to represent the movement of water across the land surface and into stream channels. Unit hydrograph options include Snyder's unit hydrograph (Snyder, 1938) and the SCS (Soil Conservation Service) dimensionless unit hydrograph (US SCS, 1972). A kinematic wave model can also be used to find the sub-basin runoff. The stream routing component is used to represent the flood wave movement from individual or combined contributions from sub-area runoff, streamflow and diversions. Routing can be undertaken by the Muskingum method, level pool routing, and the kinematic wave routing.

The HEC-HMS program (US Army Corps of Engineers, 2000) supersedes HEC-1 and provides a similar variety of options for simulating precipitation-runoff processes. In addition to unit hydrograph and hydrologic routing options, capabilities include a linear distributed-runoff transformation that can be applied with gridded (eg. radar) rainfall data, a simple "moisture depletion" option that can be used for simulations over extended time periods, and a versatile parameter optimisation option. Future versions will have capability for continuous moisture accounting and snow accumulation/melt simulation.

Yue and Hashino (2000) have developed a unit hydrograph model that incorporates four runoff components, as it was recognised that no baseflow separation would then be required. The components were defined as surface, rapid and delayed subsurface, and groundwater runoffs. The model uses four tanks, three in series to model the subsurface response and one parallel tank to model the surface flow.

The original RRL method has been developed through ILLUDAS (Terstriep & Stall, 1974), ILLUDAS-SA (Watson, 1981), and ILSAX (O'Loughlin, 1993). The ILSAX model is suited to use on urban areas, with an input hydrograph to each input point on the pipe or channel system by a time-area convolution, and pipe or channel routing or translation.

O'Loughlin et al (1991) applied the ILSAX model to three urban catchments in Sydney, and found that first estimates were reasonable, but the model fit could be improved by calibration. Dayayatne et al (1998) investigated the sensitivity of the ILSAX model loss parameters and concluded that the model is most sensitive to the depression storage on the impervious area. It was also found that there was an effect due to the level of subdivision of the catchment. Dayayatne and Perera (1999) applied the ILSAX model to 24 gauged urban catchments in Melbourne to determine regional parameters. It was found that it was necessary to consider catchment properties other than catchment slope and housing density.

2.7 Future Directions

With the advent of greater computing power and the rise of GIS, the future of hydrological modelling lies in a change from discrete event modelling to continuous simulation (ie physically based), for example Muncaster et al (1997), and the linking of the model to the GIS, for example PCSWMM (James & James, 1998)

In Australia the CRC for Catchment Hydrology has as one of its projects provision of a catchment hydrology prediction 'toolkit', containing a suite of existing and newly developed models, appropriate to a broad range of spatial and temporal scales, which will deliver improved catchment prediction modelling capability to the land and water management industry. The project will include the testing of existing modelling frameworks, examining programs in use to see where they could be made more accessible and simpler to run, and the development of new and existing models.

As an example of a recent and comprehensive model the SHETRAN system was developed by the Water Resources Systems Research Laboratory of the Newcastle University, and is based on the SHE (Systeme Hydrologique Europeen) which was developed by international collaboration between groups in the UK, Denmark and France. SHETRAN is a 3D, coupled surface/subsurface, physically-based, spatially-distributed, finite-difference model for coupled water flow, multi-fraction sediment transport and multiple, reactive solute transport in river basins. It gives a detailed description in time and space of the flow and transport in the basin, which can be visualised using animated graphical computer displays. This makes it a powerful tool for use in studying the environmental impacts of land erosion, pollution, and the effects of changes in land-use and climate, and also in studying surface water and groundwater resources and management. SHETRAN is currently being integrated in a decision-support system to maximise its usefulness in environmental impact management.

With simple models the distinction between the model types is becoming blurred, with models such as Drains (O'Loughlin & Stack, 1998) and UDD-32 providing several methods of inputting flows into the pipe or channel network, including Laurenson's Runoff Routing Model (RAFTS) and simple time-area, as well as kinematic wave. The KINDOG model of Kuczera (2000) is based on the model developed by Field and Williams (1983, 1985) and uses a combination of linear and non-linear reservoirs to model base and surface flow, and kinematic wave to model channel flow.

2.8 Summary

This brief review of the history of storm runoff models gives a good indication of the directions in which modelling is moving. Event runoff models started in a very simple fashion, accounting for only one runoff process, and assuming that the catchment behaved in a linear fashion (that is, the same response for each rainfall input).

The recognition that catchment response differed with the flow out of the catchment resulted in the development of two main streams of models, runoff routing models and hydrodynamic models. Spatial variability within the catchment was accounted for by dividing the catchment into sub-catchments, based mainly on geomorphological considerations.

Whereas the structure of runoff routing models has not changed substantially in the past 20 years, hydrodynamic models now separate channel and hillside elements, and allow for different runoff processes. Hydrodynamic models are also now generally run as continuous models, and can be considered to be an attempt at a full physical representation of the catchment.

It is considered that a runoff routing model can be developed that includes a number of runoff processes. This model can be initially very simple, but may in future be extended to include continuous simulation. This will progress the development of hydrological models down the second main stream that in the past 20 years has not received much attention.

Runoff Routing can be defined as the process of routing rainfall-excess (or surface runoff) through catchment storage to produce an outflow that is an estimate of the surface hydrograph of a catchment

Eric Laurenson (1964)

3. Description of the Models

3.1 Introduction to Modelling

The basis of runoff routing as a method of flood estimation is that the catchment can be represented by a conceptual model reflecting storage effects, and an input representing the rainfall excess. Runoff routing applies only to surface runoff, and does not estimate baseflow. It is thus modelling only part of the total runoff process. It is not to be confused with rainfall-runoff process modelling, which estimates baseflow and evapotranspiration losses from the catchment as well as surface runoff.

According to Laurenson (1964) the model should provide for:

- Temporal variation in rainfall excess;
- Areal variation in rainfall excess;
- The fact that different elements of rainfall excess pass through different amounts of storage;
- The fact that catchment storage is distributed rather than concentrated; and
- The fact that in general the relationship between stream discharge and catchment storage is non-linear.

The ILSAX model was developed for application on urban catchments, and for this reason does not allow for catchment non-linearity. It also in general allows for hydrograph translations within a pipe system rather than for full storage routing. This is permissible because storage effects are small in pipes.

The origin of three of the models described (RORB, WBNM and RAFTS) can be traced to the work of Laurenson in the 1960s (Laurenson, 1964) who developed a simple runoff routing model for application on Australian catchments (the LRRM model). The ILSAX model has its origin in the United Kingdom, where it was originally released in 1963.

These four models can be regarded as having their origin in the same period, where the introduction of computers allowed for the increase in calculation complexity over the more manual methods of the unit hydrograph or the rational method.

Reference has also been made in this study to a more recently released model, KINDOG (Kuczera, 2000). KINDOG uses a different approach to the other models described, by splitting hillside and channel processes, allowing for more than one runoff process, and using a kinematic wave approach to the modelling of surface flows on the hillside, and in the channel system.

This chapter describes the structure and operation of the models.

3.2 ILSAX

3.2.1 Background of the ILSAX Model

The ILSAX model is described in detail in the user manual (O'Loughlin, 1993). It is a hydrograph model designed to be used in the analysis of urban areas.

The ILSAX model has a long history of development, extending back to the Transport and Road Research Laboratory (TRRL) in the United Kingdom (U.K. Transport and Road Research Laboratory, 1976), where the original model was released in 1963. It modelled the pipe system reach by reach, generating hydrographs at each entry point into the system by the time-area method, and routing the combined hydrographs through the pipe system.

The ILSAX model was further developed to the DRAINS model (O'Loughlin & Stack, 1998), which included more detailed hydraulic modelling.

3.2.2 Rainfall Definition

Rainfall can be applied to the model either uniformly across the catchment, or in the standard version of ILSAX with up to three different patterns for the sub-areas. ILSAX also has built-in standard rainfall patterns from Australian Rainfall and Runoff.

3.2.3 Sub-area Definition

The ILSAX Model can be formulated such that every inlet to the pipe system is modelled, or in the case of design an inlet group can be modelled, with the assumption that inlets will be provided to satisfy the required capacity.

All sub-areas in the model are separated into three types:

- The directly connected impervious area, being that paved area and roof from which flow passes directly to the inlet;
- The supplementary paved area, an impervious area from which flow is discharged on to a porous surface before reaching the paved flow path. An example of supplementary paved area is a roof area that discharges to a lawn; and
- The grassed or pervious area. Once the losses on the grass (pervious) area are satisfied, flow will pass to the inlet.

It is possible to have non-contributing areas within the model.

The definition of these three contributing areas is one of the more difficult parts of the formulation of the model. So far little work has been done on the relative percentages that are typical for various types of catchments.

3.2.4 Rainfall Losses

ILSAX has a standard loss model for the grassed areas based on the general equation developed by Horton in the 1930s:

$$f = f_c + (f_o - f_c)e^{-kt} \qquad \text{Equation 3.1}$$

where f is the infiltration capacity (mm/hr)

f_0 and f_c are initial and final rates on the curve (mm/hr)

k is a shape factor, taken as 2 h^{-1}

t is the time from the start of rainfall (minutes)

Equation 3.1 describes the curves shown in Figure 3-1. These only apply where there is sufficient rainfall to completely satisfy the infiltration capacities, and accumulated infiltration is increasing at the maximum rate.

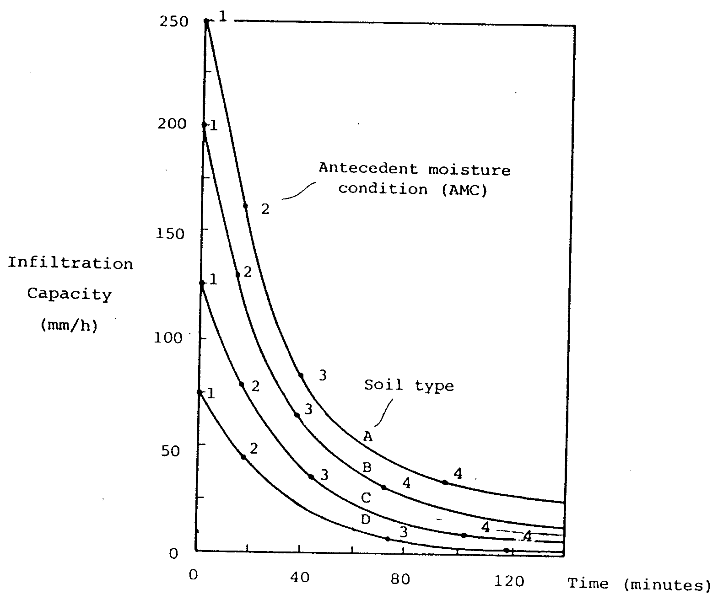


Figure 3-1 ILSAX Infiltration Curves (after O'Loughlin, 1993)

The curves represent soil types which follow the classification used by Terstriep and Stall (1974) that is based on a system developed by the U.S. Department of Agriculture, and described in references such as Chow (1964). There are four main soil classifications, designated A, B, C and D, corresponding to 1, 2, 3 and 4 in the ILSAX input. These are described as:

1 (or A) - low runoff potential, high infiltration rates (consists of sand and gravel);
2 (or B) moderate infiltration rates and moderately well drained;
3 (or C) slow infiltration rates (may have layers that impede the downward movement of water)
4 (or D) high runoff potential, very slow infiltration rates (consist of clays with a permanent high water table and a high swelling potential)

These soil types are used in conjunction with antecedent moisture conditions (AMCs) that fix the points on the infiltration curves at which calculations commence. This is specified, not by an initial infiltration rate in mm/hr but by an antecedent depth of moisture, which corresponds to an area under the curve to the left of the starting point.

The AMCs used in ILSAX are given in Table 3-1:

Table 3-1 Definition of AMC in ILSAX

Number	Description	Total rainfall in 5 days preceding the storm (mm)
1	Completely dry	0
2	Rather dry	0 to 12.5
3	Rather wet	12.5 to 25
4	Saturated	Over 25

The ILSAX program allows the input of any soil type and AMC within the stated range, including fractions. This feature is not consistent with the above definition of AMC. It is also possible for the user to define a curve, and this feature may be used to apply an initial and continuing loss model.

The selection of soil type and AMC, or initial loss, for a design event has a bearing on the design flows predicted by ILSAX. While this parameter is not important in the design of the "minor" system for frequent events it does become important when considering the "major" system, as the predicted flow is sensitive to the adopted losses. Two examples are as follows:

- For the Main North Road catchment near Parafield Airport in Adelaide, BC Tonkin & Associates determined that increasing the initial loss used in the ILSAX model from 30mm to 40mm reduces the predicted 100 year ARI flows throughout the catchment by 20% to 30% (BC Tonkin, 1996); and
- In the old South Western Suburbs Drainage Scheme area of Adelaide, Kinhill (1997) reported increases in the predicted 100 year ARI flows of between 28% and 89% when changing the ILSAX soil type from type 2 to type 3.

3.2.5 Hydrograph Generation

Hydrographs for each sub-area are generated using the time-area method in which the excess rainfall is combined with the time-area diagram, in a similar manner to unit hydrograph calculations.

A time of entry (or time of concentration) must be determined for each sub-area.

The time of entry is generally calculated in the program from data on gutter flow lengths and grades, and overland flow lengths and grades. Gutter flow time is calculated by Manning's formula, using either an assumed hydraulic radius of 60mm and a roughness coefficient of 0.02, or by using an actual hydraulic radius and Manning's n. The application of the latter is not considered warranted, as in any real situation the gutter flow depth, and thus the hydraulic radius is constantly changing with time.

Overland flow time is based on the kinematic wave equation for overland flow (Ragan and Duru, 1972):

$$t_{overland} = 6.94 \frac{(Ln)^{0.6}}{I^{0.4} S^{0.3}} \qquad \text{Equation 3.2}$$

where $t_{overland}$ is the overland flow time in minutes

L is the flow path length in metres

n	is the surface roughness or retardance coefficient
I	is the rainfall intensity in mm/hr
S	is the slope of the flow path in m/m

The surface roughness or retardance coefficient *n* is similar to but not identical to the coefficient n in the Manning's formula for open channel flow. Both are a measure of roughness, but they have different units, flow time being proportional to n in the Manning's formula and $n^{0.6}$ in the kinematic wave equation. They cannot be directly compared.

To determine the total amount of runoff the rainfall falling on the supplementary paved area is added to the grassed area. The losses are then applied to the total depth of rainfall on the grassed area. A depression loss (usually 5mm) is subtracted before the Horton type loss model is applied. For paved areas a depression loss (usually 1mm) is subtracted, and then it is assumed that all further rainfall on paved areas is totally effective.

3.2.6 Pit and Pipe Modelling

The program allows for the modelling of pipes, box culverts and both regular and irregular natural channels. Storage is modelled either by simple time shift, or lagging and routing through the storage in each reach. The results from the two methods show little difference (O'Loughlin, 1993), and simple time shift is recommended for general use.

The modelling of pit and pipe capacity is also catered for. Flow arriving at any pit is compared with a pit capacity, and flows in excess of the capacity can be redirected (with an appropriate lag and allowing for surface storage) to another pit. Similarly, if the pipe or channel capacity is exceeded then overflows can be redirected from the inlet at the upstream end of the reach to any other inlet, with an appropriate lag.

3.2.7 Calibrating the ILSAX Model

In calibrating the ILSAX model there are several main areas in which model parameters can be varied to achieve a reasonable calibration. These are, in order of sensitivity:

- The percentages of contributing areas can be varied. Ideally, as these are physically measurable they should be fixed. In reality however the gathering of data is very time consuming and in most cases is not carried out. Each property must be surveyed for the total impervious and pervious area, and must be visited to determine how much of the impervious area contributes to the street system. The program allows for sensitivity adjustment by transferring a percentage of directly connected paved area to supplementary paved area, and can further reallocate area to the grassed area component.
- The losses can be varied. In doing so it is important if possible to attain a fit that has both peak flow and runoff volume modelled correctly. For smaller magnitude storms no runoff occurs from the grassed area and thus the depression storage on the paved area can be estimated.
- The time of entry for the sub-areas can be changed in the program by a percentage or by changing the retardance coefficient for the grassed areas or Manning's n for the gutters. As storage within the sub-areas is not directly modelled this is the only way of allowing for storage effects before flow reaches the inlet.
- The Manning's n of the modelled pipe or open channels can be changed to reflect the flow time and storage within the pipe or open channel system.

3.3 RAFTS

3.3.1 Background of the RAFTS Model

A detailed description of the RAFTS model is contained in the RAFTS-XP user manual (WP Software 1994).

The model consists of five modules of which two are used to convert rainfall to runoff. Laurenson's non-linear runoff routing model (Laurenson 1964) modified by Aitken (1975) is contained in one (the hydrograph module) and the other (the loss module) uses Phillip's infiltration equations (Phillip 1957) and a modified version of the Australian Representative Basins Model (ARBM) as described by Goyen (1981), or a simple initial loss followed by a continuing or proportional loss.

The model was originally jointly developed by Willing and Partners Pty. Ltd. and the Snowy Mountains Engineering Corporation (Goyen and Aitken 1976) and called the Regional Stormwater Drainage Model.

Since the early 1980s Willing and Partners Pty. Ltd. have carried out significant development of the model, resulting in the RAFTS model, which is marketed by XP Software, a division of Willing and Partners.

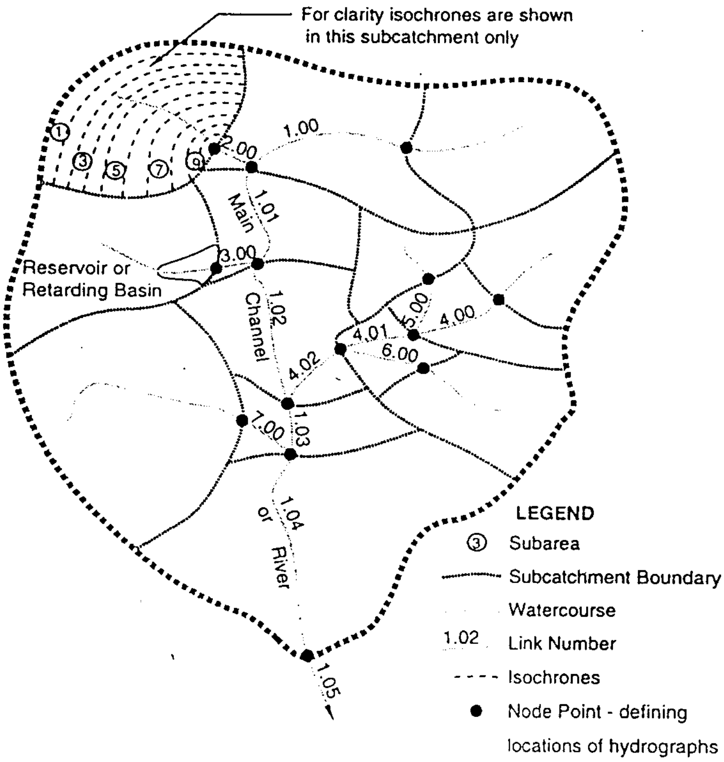


Figure 3-2 RAFTS Model Structure (after WP Software, 1994)

The model is described as suitable for application on all catchments ranging from rural to fully urbanised.

3.3.2 The Runoff Routing Module

The RAFTS program uses the Laurenson Runoff Routing Model (LRRM) as described by Aitken (1975). Each sub-catchment is divided into 10 equally sized sub-areas and the rainfall excess is routed and summed through the ten sub-areas using a non-linear storage. A channel or pipe also incorporating storage connects sub-catchments. Alternatively hydrographs may be translated between sub-catchment outlets.

The storage relationship for each sub-area is of the form:

$$s = Bq^{(n+1)} \qquad \text{Equation 3.3}$$

where s is the storage volume (hrs x m^3/sec)
B is the storage delay time coefficient
q is the instantaneous rate of runoff (m^3/sec)
n is the storage non-linearity exponent

The default value of n in the RAFTS model is -0.285, based on Aitken (1975). Aitken also derived an expression for B, based on data from six urbanised catchments as follows:

$$B = 0.285\, A^{0.52}\, (1+U)^{-1.97}\, S_c^{\;-0.5} \qquad \text{Equation 3.4}$$

Where A is the catchment area (km^2)
U is the fraction of the catchment that is urbanised (varies from 0 to 1)
S_c is the slope of the catchment (%)

Various other modification factors for B are also incorporated in the model. These include an IBFL factor to better model older urban areas where more storage is considered to be available for larger recurrence interval events, PERN to modify catchment roughness and a factor BX to be used as a calibration factor.

Aitken (1975) derived the default relationship for the storage parameter B by fitting the Laurenson model (ie. A single node RAFTS model) on six catchments ranging in size from 0.767km^2 to 56.2 km^2, and ranging from 25% urbanised to 100% urbanised. From three to seven storms were selected for each catchment, and for each a value of B derived that matched the gauged peak flow. The mean B value for each catchment was then used in the derivation of the storage relationship. In the derivation some storms were omitted because they contained runoff from impervious areas only. There is no indication of what losses were used.

Aitken also derived a relationship for catchments that are fully rural (U = 0), but this is not used in the RAFTS model. It should be noted that no fully rural catchments were used in the derivation of the parameter B. This is surprising, given the wide usage of RAFTS for the modelling of rural catchments.

The storms used in the derivation of the B value by Aitken covered a range of recurrence intervals, and percentages of contribution of runoff from pervious areas. The values of B varied widely for each catchment. The explanation for the variation in the B value could lie in the very different responses of the paved and pervious areas within urban catchments, with the overall apparent storage value changing depending on the relative contributions of the two types of areas.

In recognition of this the RAFTS manual recommends the treatment of each sub-area as two sub-catchments (split catchments), one completely impervious (U=2) and the other as rural (U=0). The value U = 2 is an extrapolation of the urbanised percentages for the catchments used by Aitken.

Hood (1991) and Hood and Daniell (1993) have examined the treatment of the storage parameter B in the RAFTS model and raised doubts as to the applicability of the recommended values and approach.

In particular the conclusions were that:

- Different storage parameters for the split and lumped models are needed, and
- The user must decide upon the network size or number of storage nodes as this too influences the estimated outflow from the model.

The basis for the recommended split catchment is of concern as neither of these limiting types of area (U=0 or U=1) were used in the derivation of the expression. The use of the storage parameter B related to $S_c^{-0.5}$ for all catchments must also be questioned. In general when deriving regional storage relationships for other rural runoff models (RORB or WBNM) catchment slope is not found to be a relevant variable. Indeed, if regression is carried out for the five rural catchments examined by Aitken, this is also found to be the case. The relevant relationships are:

$$B = 0.277\,A^{0.57}\,S_c^{-0.08}$$
$$\left(r^2 = 0.98\right)$$
Equation 3.5

Or with one dependent variable:

$$B = 0.36\,A^{0.62}$$
$$\left(r^2 = 0.98\right)$$
Equation 3.6

However, for impervious catchments involving gutter flow the use of a slope term may be reasonable, as flow time and thus storage lag is likely to be related to $L/S^{0.5}$ (from Manning's equation). It may be thus more reasonable to derive separate relationships for these two different types of area within the model.

3.3.3 Rainfall Loss Module

RAFTS uses either an initial loss with a continuing or proportional loss, or infiltration parameters to suit Phillip's infiltration equation, using comprehensive Australian Representative Basin Model (ARBM) algorithms to simulate excess runoff (Goyen, 1981).

The initial and continuing or proportional loss model is the simplest model. It assumes that all rainfall is lost to depression storage, infiltration or evaporation until the rainfall exceeds the value of the initial loss. From then a loss occurs, either as continuing loss (mm/hr) or as proportion of the incipient rainfall (proportional loss).

The use of Phillip’s infiltration equations is more geared to the use of RAFTS as a continuous model, and adds complexity and a number of extra parameters.

3.3.4 Reservoir Routing Module

A reservoir routing module is available in RAFTS to handle ponding basins and other detention and major storage areas. It includes a variety of basin configurations, including those that are hydraulically interconnected. This occurs where the operation of a basin has an effect on the stage/discharge relationship of an upstream basin.

3.3.5 River/Channel Routing Module

RAFTS includes a river or channel routing model that uses the Muskingum-Cunge procedure. Where appropriate, simple time shift routing is also available. The channel routing module contains an option for the inclusion of a baseflow pipe.

The manual gives little guidance as to appropriate values for channel roughness and the sensitivity of the model to the selection of both this and the channel section for use in the routing procedure. The roughness value chosen would not be a normal Manning's n for the section, as it needs to allow for such things as channel irregularity and tortuosity.

3.3.6 Calibrating the RAFTS Model

The RAFTS model can be calibrated in the same fashion as the ILSAX model, by the adjustment of the losses and the storage parameters. The model does not allow for sensitivity adjustments on contributing areas, as ILSAX does, because paved and pervious parts of the catchment are modelled completely separately, with different loss and routing parameters.

The adjustment of losses can be followed much as for ILSAX to obtain reasonable agreement of total runoff volumes, peak flows and the start of the rise of the runoff hydrograph.

The main calibrating factor in the storage routing is the BX parameter, although the non-linearity factor n can be varied from the standard -0.285. The BX parameter applies a uniform multiplication factor over all sub-catchments.

The BX factor cannot be used as a regional parameter, as it has not been shown that the dependence on both the sub-catchment area (exponent 0.52) and slope (exponent -0.5) are applicable to all catchments. For this reason when calibrating the RAFTS model to determine a regional relationship a single node or sub-catchment should be used, with a direct input of the parameter B instead of the BX multiplier. The form of relationship derived can then be examined to determine whether the default equation is reasonable.

For urban catchments, the single node model can be calibrated in two steps. Firstly, those storms with no pervious runoff are examined, to determine appropriate losses and storage parameters for the impervious area. The storms having pervious runoff are then modelled, using those parameters already derived for the impervious area, to determine appropriate storage parameters for pervious runoff.

3.4 RORB

3.4.1 Background of the RORB Model

The RORB model is described in the user manual (Laurenson and Mein, 1990). The first version of the RORB program was released as a program named RORT in 1975. Since that time its use in Australia has become very widespread and it is now probably the most widely used runoff routing program for rural catchments. Extensive work has been carried out on the main storage parameter k_c, which is summarised in Australian Rainfall and Runoff (IE Aust, 1987). In particular, South Australian values have been examined by Maguire et al (1986) and Kemp (1993).

3.4.2 RORB Model Procedure

RORB represents the actual channel network in a catchment by a network of storages arranged similarly to the actual network. Water may enter the network in several ways, but it is generally input as a sub-area inflow, which represents the hydrograph of rainfall excess assumed to enter the stream network at a point near the centroid of the sub-area.

Channel reach storages are assumed to have a storage-discharge relationship of the form:

$$S = 3600\, kQ^m \qquad \text{Equation 3.7}$$

where S is the storage (m^3)

k is a dimensional empirical coefficient (related to the storage delay time)

Q is the outflow discharge (m^3/sec)

m is a dimensionless exponent

The exponent m is a parameter to be fitted, however a value of 0.8 is recommended by the user manual for use on ungauged catchments, and is the exponent value usually used in the derivation of regional relationships.

The coefficient k is formed as the product of two factors:

$$k = k_c k_r$$ Equation 3.8

where k_c is an empirical coefficient applicable to the entire catchment and stream network

k_r is a dimensionless ratio called the relative delay time, applicable to an individual reach storage

For catchment studies each individual storage i is modelled having a coefficient $k_{r\,i}$ defined as follows:

$$k_{ri} = F_i \frac{L_i}{d_{av}}$$ Equation 3.9

where $k_{r\,i}$ is the relative delay time of the storage i

F_i Is a factor depending on the type of reach (eg natural or lined)

L_i is the length of the channel reach represented by storage i (km)

d_{av} is the average flow distance in the channel network of sub-area inflows (km)

Losses on the catchment are modelled by the use of an initial and continuing loss applied to the catchment rainfall.

3.4.3 Calibrating the RORB Model

The RORB model is calibrated by first adjusting the losses to match the start of the rise of the catchment outflow hydrograph, then adjusting the k_c and m for the catchment to match the hydrograph shape.

One problem with the calibration of the RORB model is that there are a number of combinations of k_c and m that can be used to fit the peak flow generated by the RORB model to gauged events. The only variation is in the shape of the hydrograph. Weeks (1980) proposed a method of parameter interaction curves by which a unique pair of k_c and m values can be found that provide the best fit when considering a number of gauged flow events.

There has over the years been a large amount of calibration work carried out on the RORB model, and relationships for the RORB k_c have been derived for most states of Australia. Most of the calibration has been carried out with a standard value of m of 0.8, so that the k_c values can be directly compared over many catchments.

Work was carried out by Dyer et al (1994) on the derivation of a related parameter, k_c/d_{av}, claimed by Dyer to be a more reasonable parameter for regional relationships, on the basis of the strong correlation of k_c with catchment area that is implicit in the RORB model.

3.5 WBNM

3.5.1 Background of the WBNM Model

The WBNM or Watershed Bounded Network Model was first described by Boyd et al (1979). It is based on the Monash or early RORB model, but the intention was to realistically represent the detailed catchment structure and the flow of surface water in the catchment. Later versions are described by Boyd et al (1994) and Boyd (2000). It contains the same basic model structure, but includes many enhancements including the loss model, storage reservoirs, design rainfalls, flow diversions and alternatives for modelling watercourses including full Muskingum-Cunge routing in channels.

The WBNM94 model has much in common with the RAFTS model. Each sub-catchment is complete, with an appropriate storage value such that the flow from the sub-catchment should be reasonable. A storage between sub-catchments is included in the model. The main difference between RAFTS and WBNM is that in WBNM each sub-catchment is composed of only one sub-area, unlike RAFTS with ten, and that storage between sub-catchments can be modelled as a function of the area of the sub-catchment, not just by channel routing or translation of hydrographs.

The WBNM model is also related in some ways to the RORB model in that the catchment is sub-divided into sub-catchments each of which have inflows from excess rainfall. It differs in that two types of storage which correspond to two types of sub-catchments that comprise a catchment subdivided along watershed lines, being ordered basins which receive no inflow across any boundary, and interbasin areas which contain a stream draining upstream areas. Ordered basins can be considered to be geomorphologically similar to complete catchments, for which lag measurements are available.

3.5.2 Catchment Sub-Division and Storage Allocation

The sub-division of the catchment for the WBNM model is the same as the RORB model. The two different types of sub-catchments are then identified.

Whereas the RORB model has an overall storage lag for the catchment, which is allocated to individual storages in proportion to the relative lengths of the modelled storages, in the WBNM model storage lags are individually allocated on the basis of the type of sub-catchment.

The first or ordered basins have a storage lag allocated in the form $K_B = c\ A^{0.57}q^{-0.23}$ on the basis of the catchment lag to mean discharge relationship derived by Askew (1970). The second, or interbasin storages have a storage lag of the form $K_I = 0.6\ c\ A^{0.57}q^{-0.23}$, where K_I is the interbasin lag and the factor of 0.6 was derived empirically by the examination of the performance of the model on ten catchments. It is supported by the tracing studies of Pilgrim (1982). Excess rainfall on interbasin sub-catchments is routed through a storage equivalent to the ordered basin and added to the flow at the outlet of the sub-catchment.

3.5.3 Loss Model

The WBNM model is similar to RORB in that there is a choice of an initial loss followed by a continuing or proportional loss.

3.6 KINDOG

3.6.1 Background of the KINDOG model

KINDOG is based on the work of Field (1982), Field and Williams (1983), Williams and Yeh (1983) and Field and Williams (1987). The reason for developing KINDOG is that it has a conceptually more sound basis as it is based on open channel hydraulics and explicitly distinguishes between hillside runoff and channel flow.

Surface flow is conceptualised as Hortonian overland flow routed through a non-linear storage into the channel. Infiltration recharges a subsurface linear store that simulates the baseflow or subsurface stormflow process, supplying water to the channel at a rate slower than hillside surface runoff. Though this is an oversimplification of a very complex process, it does provide a simple and adequate description of the hillside runoff process. Flow in channels is modelled using a kinematic wave. Based on the Manning formula, the conveyance of channels is specified as a simple power function of the cross sectional area, allowing the parameters for this process to be determined by conventional measurement of cross-sections and estimation of Manning's n. Overbank flows down channels are modelled using appropriately increased conveyances which is claimed to allow more reliable extrapolation to extreme events.

3.6.2 KINDOG Model Structure

The catchment is subdivided into subcatchments or elements that are numbered sequentially. It is preferable but not essential to commence the numbering in the upper reaches, so that the number of a downstream subcatchment is higher than the one upstream. An element can be designated as either an actual reservoir or an ordinary catchment element.

Rain falling on an actual reservoir element is routed with other inflows through level-pool storage (KINDOG assumes all rainfall falling on the maximum reservoir area enters the reservoir element). In contrast, rain falling onto an ordinary catchment element is conceptualised as entering a nonlinear storage, before flowing laterally into the mainstream.

In an ordinary element only the mainstream channel needs to be identified. The stream and rill network draining into this main channel is ignored. Rainfall is assumed spatially uniform within the element.

There are three parameters related to the catchment response time, being Cr, Cs and Cg.

Cg is the sub-surface supply parameter. The rate of subsurface supply per unit area sg is related to the depth in the subsurface store hg by

$$hg = Cg \cdot B \cdot sg \qquad \text{Equation 3.10}$$

Where B is the width of the catchment element.

Cs is the surface supply parameter. The depth of water stored on a hillside hs is related to the surface supply rate ss by

$$hs = Cs \cdot B^{g} \cdot (ss)^{g} \qquad \text{Equation 3.11}$$

where Cs is the surface supply parameter with units metres$^{(1-2\ \gamma)}$ sec$^{\gamma}$, B is the width of the catchment and γ is the hillside flow exponent.

Rough values for Cs and γ can be derived by considering the hillside as a level pool drained by a rectangular broad-crested weir. It can be shown that Cs and γ equal to 0.44 metres $^{-1/3}$ sec $^{2/3}$ and $^{2}/_{3}$ respectively. However, this analysis can be quite misleading.

Separate values of C_s can be specified for the developed (impervious) and undeveloped (pervious) portions of the catchment.

Cr is the Channel Conveyance Coefficient. The kinematic wave approximation sets the friction slope equal to the bed slope and Manning's equation is used for the relationship between discharge and flow area

$$Q = K\,S^{0.5} \qquad \text{Equation 3.12}$$

Where K is the channel conveyance (m^3/s) defined as

$$K = \frac{A_r^{2/3}}{nP^{2/3}} \qquad \text{Equation 3.13}$$

With n being Manning's channel roughness and P the wetted perimeter.

For a composite channel such as a stream with berms, the total conveyance is simply equal to the sum of the conveyances of the various subsections.

Because the wetted perimeter P is strongly correlated with flow area A_r, an exponential relationship between K and A_r is used;

$$K = Cr\,A_r^{\,m} \qquad \text{Equation 3.14}$$

Where Cr is the channel conveyance coefficient and m is the channel conveyance exponent. For composite channel sections different values of Cr and m may be ascribed to different intervals of A_r.

3.6.3 Loss Model

The loss model includes an initial and continuing loss.

The initial loss is the initial infiltration that is required to saturate the soil in order that recharge to the subsurface store can commence. The initial loss is the amount of water "lost" to the soil. In KINDOG this water does not infiltrate into the groundwater store and does not contribute to either subsurface or surface runoff.

The continuing loss is the amount of water that infiltrates into the soil once the soil is saturated and infiltration reaches steady state. Once a volume of infiltration equal to the initial loss has occurred, recharge to the groundwater store occurs at the minimum of the precipitation rate and the continuing loss rate. Rainfall in excess of the continuing loss becomes surface runoff

3.6.4 Calibration

KINDOG has built into it two methods of automatic calibration. These calibrate selected parameter multipliers to an observed storm at a user specified node using either the Shuffled Complex Evolution (SCE) (Sorooshian et al, 1993) or the Nelder-Mead simplex non-linear regression technique (Nelder and Mead, 1965). If this option is selected, the user is prompted to specify which parameter multipliers should be calibrated and asked to provide reasonable bounds for the multipliers. If the user wishes, a report on the fitting results can be viewed.

The SCE algorithm should be employed in cases where little information is available on parameter values. It is a robust global search method but is computationally intensive. The Nelder-Mead algorithm is much faster than the SCE method but can easily get "stuck" near a local optimum. It is best suited for cases where good initial guesses for the parameters are available.

Many types of models have been developed and used in runoff routing applications. While some have more physically realistic structures than others, all models are only approximations of reality and require care and expertise in their application and interpretation.

David Pilgrim (1987)

4. Relationships Between the Models

Since all of the models examined have the same intent of representing real catchments, and three of them (RORB, RAFTS and WBNM) use runoff routing through storages it could be expected that there are relationships between the parameters of the models. Several studies have investigated the relative performance of the RORB, RAFTS and WBNM models (Boyd (1983), Sorbinoff et al (1983)), but these studies did not examine if theoretical relationships exist between the models.

The investigation of the relationships between the parameters of the models gives an insight into the differences between the models, and enables conclusions to be made.

4.1 Relationship of the Storage Parameters in RORB and RAFTS

Because of the extensive amount of calibration of the RORB model it would be beneficial to find a relationship between the fitted storage parameters of RORB and RAFTS, such that the regional relationships of one model could be used in the other model. This would overcome the limited amount of calibration of the RAFTS model.

A single node of the RAFTS model, which is subdivided into ten sub-areas has a storage

$$s = B\,Q^{n+1} \qquad \text{Equation 4.1}$$

Where s is in hrs.m^3/sec and an equivalent RORB model has the storage relationship:

$$S = 3600\,k\,Q^{m} \qquad \text{Equation 4.2}$$

Where S is in m^3 , thus with allowance for the different units:

$$n = m - 1 \qquad \text{Equation 4.3}$$

And:

$$B = k = k_c\,k_{ri} \qquad \text{Equation 4.4}$$

for the individual reach storage. In a single node RAFTS model representing a catchment with a total length L, the area is divided into ten equal sub-areas, with the length of the main channel within each sub-area L_i being approximately L/10 and d_{av} (the average flow distance on the catchment) being approximately L/2.

Thus:

$$\begin{aligned} B &= k_c\,k_{ri} \\ &= k_c \frac{L_i}{d_{av}} \\ &= k_c \frac{L}{10}\frac{2}{L} \\ &= 0.2\,k_c \end{aligned} \qquad \text{Equation 4.5}$$

The relationship between L_i and d_{av} can be found for natural catchments if d_{av} and the total catchment length L is known. Two data sets were examined, that of Hansen for Victoria (Hansen, 1986) and Flavell for Western Australia (Flavell 1983), with 30 and 51 catchments respectively, ranging in area from 5.46 km^2 to 6526 km^2. For each data set the average L_i (assumed to be L/10) divided by d_{av} was calculated. This was found to be 0.194 for Victoria and 0.200 for Western Australia. Thus it would seem that for natural catchments in Australia the relationship is reasonable.

This relationship will be tested by examining the performance of fitted RORB k_c values in a simple one node RAFTS model, deriving the B value as above, and using the same losses as in the RORB model for consistency. The results of this testing are given in Chapter 7.

4.2 Relationship Between the Storage Lags in RAFTS and ILSAX

4.2.1 The basis of the RAFTS Lag parameter B

The lag of urban catchments has been studied by Bufill and Boyd (Bufill and Boyd, 1992), as part of an investigation to produce a simple flood hydrograph model for urban areas. The major findings were:

- The storage discharge relationship for impervious areas can be considered to be linear, that is catchment lag does not change with peak discharge;
- The storage lag Ki for impervious areas was only weakly related to total catchment area, total impervious and impervious connected area;
- When a regression equation was fitted linking Ki to impervious connected area it was found that there was a minimum Ki of 300 seconds (5 minutes); and
- The introduction of catchment average slope as a second independent variable did not improve the prediction results.

To compare this Ki with the RAFTS Bi, the catchment can be considered as a single storage with a lag Ki, which is represented by 10 sub-areas each with a storage lag Bi in the RAFTS model. Ki represents the mean storage delay time, thus:

$$Bi = \frac{Ki}{5} \qquad \text{Equation 4.6}$$

where Ki is the lag parameter for the impervious area of a catchment (hours)

Bi is the B parameter for the impervious sub-catchment in a split catchment model

Bufill and Boyd also assumed that the storage parameter for the pervious area was simply related to the impervious area by the relationship;

$$\frac{Kp}{Ki} = \left(\frac{np}{ni}\right)^{0.6}$$ Equation 4.7

where Kp is the lag parameter for the pervious area
np is the Manning's n roughness of the pervious area
ni is the Manning's n roughness of the impervious area

This relationship was based on the kinematic wave equation for overland flow (Ragan and Duru, 1972)

Thus, as before:

$$Bp = \left(\frac{ki}{5}\right)\left(\frac{np}{ni}\right)^{0.6}$$ Equation 4.8

Where Bp is the B parameter for the pervious area in a split catchment model.

However, Ragan and Duru (1972) state also that the kinematic wave equation holds only for flow lengths where the product of the flow length and rainfall intensity is less than 500 (imperial units), which is equivalent to 3870 in metric units (metres and mm/hr). For a rainfall intensity of 81 mm/hr, the five year, five minute rainfall intensity for a typical catchment, being the Glenelg catchment the greatest flow length for which the kinematic wave equation holds is approximately 50m. The use of the relationship on a catchment wide basis must therefore be questioned, both for this reason and because normal urban development does not allow simple overland flow to occur due to development on the block.

Bufill and Boyd indicate that the relationship for Ki and Kp is not simple, and propose that an urban catchment has a number of "clusters" of similar lag time.

The findings of Bufill and Boyd show that a simple relationship relating catchment storage parameter to catchment area and slope may not be appropriate. The storage or lag parameter may be more closely related to the time of concentration of the catchment, a concept used in the rational method.

Aitken (1975) undertook a derivation of catchment lag related to catchment time of concentration. Aitken derived a dimensionless storage delay time K_D for three catchments from the recession curve and converted this to a dimensional storage delay time K_M by the use of an empirical relationship, $K_M = K_D t_c$, where t_c is the time of concentration of the catchment. This K_M was then used in the Clark unit hydrograph model (a linear storage model) to predict flows for the catchments studied.

The value of K_D for the three catchments studied by Aitken was found to be of the order of 0.3, to which a correction was applied related to catchment size.

The time of concentration t_c is made up of the following factors for impervious areas:

- A fixed time (time of entry), that allows for areas remote from the gutter to enter the gutter;
- A gutter flow time; and
- A time of flow in the pipe or open channel system.

And for pervious areas:

- An overland flow time, that allows flow to enter the gutter;
- The gutter flow time; and
- The flow time in the pipe or open channel system.

Some conclusions can be drawn if catchment lag (for impervious and pervious areas) can be determined from the time of concentration of the catchment:

- For impervious areas there will be a minimum lag, equivalent to the time of entry to the gutter;
- The lag for the rest of the impervious area will be related to both the gutter and pipe or open channel flow lengths;
- The catchment lag will not change with the proportion of impervious area within the catchment, provided it is evenly distributed within the catchment; and
- The lag time for pervious area will be related to the impervious area lag time, differing only by the difference between the overland flow time and the time of entry for the impervious area.

The data presented in Bufill and Boyd may be used to confirm the first conclusion, that there should be a minimum value of Bi. Values of Ki are presented for 16 catchments in several countries of areas from 0.06 km^2 to 31.75 km^2 and slopes from 0.1% to 10%. Table 4-1 gives these values. From these Ki values Bi values can be derived using Equation 4.6, and these are shown on Figure 4-1.

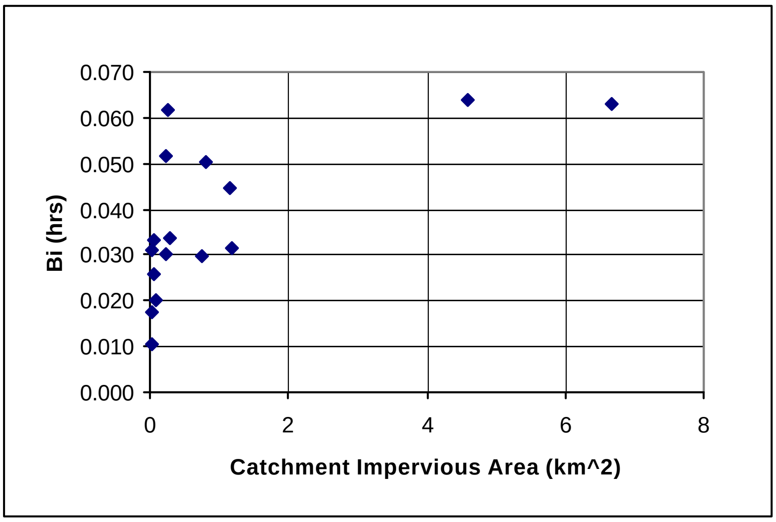


Figure 4-1 Measured Bi Parameter for Urban Areas

Table 4-1 Lag Parameters for Urban Catchments, from Bufill and Boyd (1992)

Name	Location	Country	Area(Km^2)		Slope(%)	Ki(sec)	Bi(hr)
			Total	Impervious			
Maroubra	Sydney	Australia	0.57	0.3	10.0	607.7	0.034
Strathfield	Sydney	Australia	2.34	1.17	1.0	567.6	0.032
Jamison Pk	Sydney	Australia	0.20	0.07	2.0	464.4	0.026
Fisher's Gh.	Sydney	Australia	2.26	0.81	7.8	905.6	0.050
Giralang	Canberra	Australia	0.96	0.24	4.5	542.8	0.030
Long Gully	Canberra	Australia	5.02	0.24	9.0	931.2	0.052
Mawson	Canberra	Australia	4.45	1.15	5.5	805.8	0.045
Curtin	Canberra	Australia	26.9	4.60	6.8	1152.0	0.064
Vine Street	Melbourne	Australia	0.70	0.26	0.4	1108.5	0.062
Elster Ck.	Melbourne	Australia	31.75	6.67	1.4	1132.8	0.063
King's Ck.	Florida	USA	0.06	0.04	1.2	557.6	0.031
St. Marks	Derby	UK	0.08	0.05	0.3	599.1	0.033
Clifton Gr.	Nottingham	UK	0.11	0.04	5.0	190.0	0.011
Munkeris	Copenhagen	Denmark	0.06	0.02	1.0	311.8	0.017
East York	Toronto	Canada	1.55	0.76	1.1	536.8	0.030
Malvern	Burlington	Canada	0.23	0.08	2.0	360.0	0.020

As expected Bi has a minimum value, being 0.011. There is also not a simple relationship evident between the impervious area and Bi.

Although the RAFTS model contains non-linear storage routing the mean storage delay time as measured by Bufill and Boyd can be compared with the RAFTS default relationship for Bi, as it is expected that the RAFTS Bi would represent the mean storage delay time in the catchment.

A regression of Bi versus slope and impervious area of the Bufill and Boyd data resulted in the following relationship:

$$Bi = 0.0472\, Ai^{0.22}\, s^{-0.06} \qquad \text{Equation 4.9}$$
$$(r^2 = 0.55)$$

Which is very different to the default relationship used in RAFTS. Based on equation 2.5, and with U = 2.0 (fully urban area), as recommended by WP Software (1994) the relationship for Bi is:

$$Bi = 0.0327\,Ai^{0.52}\,s^{-0.5}$$ Equation 4.10

Figure 4-2 shows the comparison between the measured catchment Bi values of Bufill and Boyd and the default value for the RAFTS model.

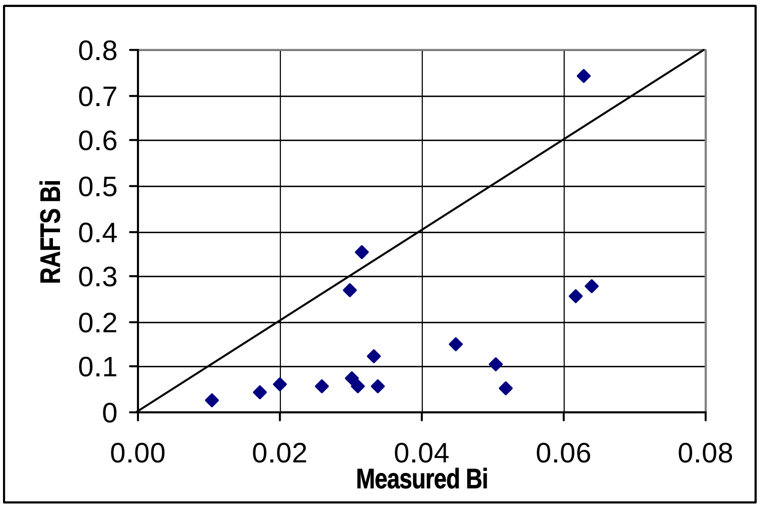


Figure 4-2 Comparison of RAFTS Bi and Bufill and Boyd Bi

The three catchments where there is some agreement are the East York, Strathfield and Elster Creek catchments. There are no obvious similarities between these. The investigation shows that the RAFTS model default value is not reflected by measurements on actual catchments. A value based on catchment time of concentration may be a much better measure. A theoretical value of storage lag Bi will be developed based on the time of concentration of urban catchments as is implicit in the ILSAX model.

4.2.2 Derivation of the RAFTS Lag Parameter B, Based on ILSAX

4.2.2.1 Impervious Area Lag Parameter

If piped urban catchments are linear as proposed by Bufill and Boyd (1992) the derivation of a theoretical storage lag is quite simple. Laurenson (1964) proposed that the storage lag of a catchment (defined as the time between the centroid of the rainfall excess and the centroid of the resulting surface runoff) was equal to the average storage delay time or lag for all elements of the rainfall excess throughout the storm and over the entire catchment. If rainfall is areally uniform then the catchment lag is equal to the storage delay time of points on the catchment corresponding to the centroid of the time-area diagram.

The basic catchment in an urban area is that area contributing to a pit forming the inlet to the pipe system. If the contributing area is considered to be of constant width the storage lag is equal to the average delay time, which would be half the total delay time:

$$K = \frac{total\ delay\ time}{2} \qquad \text{Equation 4.11}$$

This basic catchment can then be considered in two parts, the impervious and pervious areas, having different storage lags, which will be Ki for the impervious portion and Kp for the pervious. This is the method the RAFTS model represents the catchment.

For the impervious area of this basic catchment the total storage delay time in accordance with the ILSAX model (O'Loughlin, 1993) is given as the sum of the time of entry (t_e) and the gutter flow time, related to the length and slope of the gutter. By using the default ILSAX values for gutter Manning's n and hydraulic radius:

$$
\begin{aligned}
Ki &= \text{gutter flow time} + \text{time of entry} \\
&= \frac{L_g}{2 \times 7.66 \times 60 \times 60 \times \sqrt{s_g}} + \frac{5}{2 \times 60} \\
&= \left[18.15 \times 10^{-3} \frac{L_g}{\sqrt{s_g}} + 41.4 \right] \times 10^{-3}
\end{aligned}
$$

Equation 4.12

Where Ki is the lag of the impervious area of the catchment (hours)

L_g is the gutter flow length (m)

s_g is the gutter slope (m/m)

From the lag parameter for this basic catchment the lag parameter for a group of catchments, each draining to a separate inlet to the pipe system must be found. The pipe connection between the inlets can be considered to have little storage in comparison to the storage in the gutters. For this reason with the ILSAX model it is usual to apply translation of hydrographs only in the pipe system.

Using the principle of superposition of moments of flood hydrographs, as described by Boyd (1985), and if the pipe translation time is negligible the overall lag K of two areas A_1 and A_2 with lags K_1 and K_2 can be determined as follows:

$$K = \frac{[A_1 K_1 + A_2 K_2]}{A_1 + A_2}$$

Equation 4.13

If there is a time lag or translation time due to pipe flow of t_2 say between the inlets with A_2 being upstream of A_1 then the overall lag is given by:

$$
\begin{aligned}
K &= \frac{[A_1 K_1 + A_2 (K_2 + t_2)]}{A_1 + A_2} \\
&= \frac{[A_1 K_1 + A_2 K_2 + A_2 t_2]}{A_1 + A_2}
\end{aligned}
$$

Equation 4.14

Or in the general case, with n inlets:

$$K = \frac{[A_1 K_1 + A_2(K_2 + t_2) + \ldots + A_n(K_n + t_n)]}{A_{total}} = \frac{[A_1 K_1 + A_2 K_2 + \ldots + A_n K_n]}{A_{total}} + \frac{[A_2 t_2 + \ldots + A_n t_n]}{A_{total}} \qquad \text{Equation 4.15}$$

The first part of the equation indicates that the overall lag is equal to the area weighted mean lag for all inlets.

The second part represents a term accounting for the translation times of the pipes in the system. It represents what could be called the "time centroid" of all inlets in the system. If the pipe flow velocity is constant throughout the catchment the translation time would be approximately equal to the pipe flow time from the point closest to the centroid of the catchment on the pipe network to the outlet. However, in general, it is expected that pipe sizes and thus velocity will increase with contributing catchment area. The rate of increase of the pipe size is directly related to the flow, which then represents a problem in the case of the prediction of the storage lag. In the ILSAX model, pipe sizes for design runs are determined on the basis of predicted peak flow at the upstream end of the pipe reach and the pipe slope, which when combined with the assumption of full pipe flow gives a time for flow in the reach.

Varying the translation time with varying flows is not however consistent with the linear nature of urban catchments. The method used in the ILSAX model where pipe flow times are determined by calculating the velocity based on part full flow for evaluation of existing systems will lead to a non-linear catchment representation, as translation times vary with flow, ie t is a function of the catchment outflow Q. It would be more correct to assume a constant hydraulic radius, even for part full flow.

In the case where $n_p = 0.012$ and the pipe hydraulic radius is r, a relationship for the pipe translation time to the centroid of the catchment (K), assuming the time to the centroid is half the longest flow time is as follows:

$$K = \frac{n_p}{60\,x\,60\,x\,2} \sum_{i=1}^{n} \left[\frac{L_{pi}}{r_i^{0.667} \sqrt{s_i}} \right] = \left(1.67x10^{-3} \sum_{i=1}^{n} \left[\frac{L_{pi}}{r_i^{0.667} \sqrt{s_i}} \right] \right) x10^{-3} \qquad \text{Equation 4.16}$$

Where	K	is the lag or translation time due to the pipe system (hours)
	n_p	is the Manning's n of the pipe system
	n	is the number of inlets within the catchment
	L_{pi}	is the length of the ith pipe (m)
	s_i	is the pipe slope for the ith pipe (m/m)
	r_i	is the pipe hydraulic radius (m)

Using the principles of superposition of moments the complete equation for the impervious area lag Ki of a piped urban catchment is then:

$$K_i = \left(\left(1.67x10^{-3}\right) \sum_{i=1}^{n} \frac{L_{pi}}{r_i^{0.667}\sqrt{s_i}} + \left(18.15x10^{-3}\right) \frac{L_g}{\sqrt{s_g}} + 41.4 \right) x\ 10^{-3}$$

Equation 4.17

Where Ki is the impervious area lag (hours)

To determine the impervious area storage lag parameter Bi for the RAFTS model it can be assumed that the relative delay time to the centroid of the catchment is 0.5 as per Equation 4.12. Since ten equal storages are assumed in the RAFTS model it follows that:

$$Bi = \frac{Ki}{10\ x\ 0.5} = \frac{Ki}{5}$$

Equation 4.18

And therefore from Equation 4.8:

$$B_i = \left(\left(0.333x10^{-3}\right) \sum_{i=1}^{n} \frac{L_{pi}}{r_i^{0.667}\sqrt{s_i}} + \left(3.63x10^{-3}\right) \frac{L_g}{\sqrt{s_g}} + 8.3 \right) x\ 10^{-3}$$

Equation 4.19

This relationship can be tested by the application of the RAFTS model to catchments modelled with ILSAX models and more particularly to gauged catchments. The results of this testing are given in Chapter 7.

4.2.2.2 Pervious Area Lag Parameter

The expected form of an equation for pervious area lag parameter Bp would contain the same terms for pipe and gutter flow, and differ only in the addition of a term for the storage lag on the pervious area. If it is assumed that the pervious surfaces of most urban areas have approximately the same roughness then a term $Lo/So^{0.5}$ could be introduced, the Lo and So representing overland flow length and slope respectively.

Unfortunately, runoff from the pervious portion of urban areas has been proven to be rare in Adelaide, with only two gauged events having runoff from pervious areas during the two years and for the two catchments used for calibration of the ILSAX model as detailed in Chapter 6. These occurred in the Frederick Street, Glenelg catchment. The average Bp value for the pervious areas in these two events was 0.05, with the impervious Bi value being 0.04. It can be seen from this that there is little difference between Bp and Bi, possibly of the order of 0.01 hours. If it is assumed that the storage relating to the overland flow is reasonably constant in urban areas (due to flow lengths and roughness being similar) then a preliminary equation for Bp could be:

$$Bp = Bi + 0.01 \text{ hours} \qquad \text{Equation 4.20}$$

In most cases for the design of urban drainage systems in Adelaide the value of Bp is of little significance compared with Bi, as the greater proportion of flow comes from the impervious portion of the catchment.

4.2.3 Flows In Excess of the Pipe System Capacity

Equations 4.19 and 4.20 apply only to the case in urban areas where the flows do not exceed the capacity of the pipe system. If inlet capacity or pipe capacities are exceeded more flow can be expected on the road surface, and it can be expected that the apparent lag for the catchment will increase.

In many urban catchments design flows are required for the case of overland and pipe flow, for example where a major-minor system is being designed. It may be expected that the lag of that part of the flow in excess of the pipe system capacity would be related to the total surface flow path length and slope. In Equations 4.19 and 4.20 for Bi and Bp, the L_g term would represent the total flow path length taken by overflow within the catchment.

4.3 Relationship Between RORB and WBNM

The RORB Manual (Laurenson and Mein, 1990) gives guidance as to how the RORB model structure can be altered to that of the WBNM model, by the following method:

- Catchment subdivision is performed as is usual for the RORB model;
- Two nodes are placed in each sub-area, one at the outlet and the other at any point not on the sub-area's main stream. The latter node is the sub-area entry point, and is joined to the node at the sub-area outlet by a notional stream. No other nodes are used;
- Model storages are placed between all pairs of adjacent nodes. Relative delay time indicators are used in place of reach lengths. For model storages between a sub-area entry point and the sub-area outlet, $A^{0.57}$, where A is the area of the sub-area. For model storages between the upstream and downstream ends of a sub-area $0.6A^{0.57}$ is used;
- The control vector is formulated noting that for all sub-areas having an inflow at the upstream end as well as an rainfall excess input, the downstream end is a confluence of the sub-area's main stream and the notional steams from the sub-area entry point, and;
- The storage exponent m in RORB must be set at 0.77, so that an equivalent non-linearity is achieved.

If it is desired to calculate the WBNM c parameter, this can be done using the equation:

$$c = \frac{k_c}{d_{av}} \qquad \text{Equation 4.21}$$

4.4 Summary

This chapter has examined the theoretical relationships between the model examined. It has been concluded that theoretical relationships exist between the storage parameters of RORB and RAFTS, and that a theoretical value of storage lag B for the RAFTS model can be derived for urban areas from the lag implicit in the ILSAX model. A relationship between RORB and WBNM has also been discussed.

One common feature of the three Australian runoff-routing models is that although the division is based on the physical structure of the catchment, the actual number of sub-areas into which the catchment is divided is a subjective decision of the modeller. Since the model response is affected by the number of reservoirs used, this aspect of modelling requires consideration if consistent results are to be obtained.

Michael Boyd (1985)

5. Effect of Model Structure on Predicted Flows

5.1 Introduction

One subjective procedure in the application of runoff routing models is the division of the catchment into sub-catchments. The number of sub-catchments and the layout of the sub-catchments need to be chosen. Studies such as those of Boyd (1985), Boyd et al (1979a), State Rivers and Water Supply Commission, Victoria (1979) and Weeks (1980) demonstrate that the degree of subdivision of the catchment model affects the computed outflow and also the appropriate value of the storage parameter.

The RORB user manual (Laurenson and Mein, 1990) recommends that no sub-catchment should be greater than 25% of the total catchment area and that no reach should be greater than one third the length of the main stream. No evidence supporting this recommendation is given.

This chapter examines and quantifies the effect of the number of nodes in two of the models, RAFTS and WBNM. For these two models it is relatively simple to undertake this assessment, as sub-catchment storage is related to the area of the sub-catchment. As the RORB model relates individual channel storages to a measure of total catchment storage the assessment would be more difficult, and was not undertaken.

5.2 Previous Investigations

The basic component of all linear runoff routing models is the linear reservoir which transforms a time varying inflow i(t) to a time varying outflow q(t). The linear reservoir can be represented by a first order linear equation, in which the parameter K is equal to the lag time between the centroids of the inflow and outflow. The model equation is:

$$K \cdot \frac{d}{dt} q(t) + q(t) = i(t) \qquad \text{Equation 5.1}$$

The first moment of this model is K, the catchment lag. The lag of a model having a number of linear storages in series or parallel can then be found using the principles of superposition of moments.

Although the lag of a model containing non-linear reservoirs cannot be determined in the same way, the effect of the reservoirs is similar, as shown by Boyd (1985)

Boyd set up WBNM models with differing number of sub-catchments, and standard rainfall excess. Five catchments were used. The studies showed that poor results were obtained if the number of sub-catchments was too low, principally because the hydrograph peak occurred too early. The minimum number of sub-catchments for which hydrograph properties became stable depended on the size of the catchment modelled. His conclusions were;

- For the branched network models, as the number of reservoirs N increases and the catchment division becomes finer, the size of all sub-areas decrease, and the lag parameter K applying to each reservoir becomes smaller;

- The variation of hydrograph properties with N depends on the rate of decrease of K relative to the rate of increase of N; and
- The minimum value of N required for hydrograph properties to become stable in both the linear branched network models and the non-linear model WBNM (Boyd et al, 1979) depends on the size of the catchment being modelled and has values of approximately 4, 7 and 15 for catchments areas of 0.1, 10 and 1000 km^2 respectively.

Dyer (1994) also investigated the effect of the number of sub-catchments, this time using the RORB model. The trial involved five catchments, each with two events. The events were automatically calibrated for a differing number of sub-catchments. The number of sub-catchments was reduced by logically combining the original sub-catchments such that the resultant model was a realistic representation of the catchment for the given number of sub-catchments.

Dyer reached the conclusion that Boyd's recommended number of sub-catchments for the WBNM model appears to be high with respect to application to the RORB model.

Dyer also made the point that:

"One often neglected point regarding the number of sub-catchments is that there needs to be sufficient sub-catchments upstream of any point of interest such that the catchment is adequately modelled up to that point. Thus if Boyd's recommendations are to be used, in a catchment of 100km^2 (recommended minimum number of sub-catchments is ten) containing a point of interest, e.g. the site for a retardation basin, with 50km^2 upstream of it, there should be approximately 9 sub-catchments upstream of the point of interest and sufficient sub-catchments downstream of the point of interest to maintain a consistent approach to the subdivision of the catchment, thus the model would have approximately 18 sub-catchments, not 10 as from first indications."

An effect due to the number of nodes in a RAFTS model has also been noticed, and investigated by Hood (1991) and Hood & Daniell (1993). The developer of the RAFTS model, Alan Goyen has been aware of the effect for many years (pers. com 1998). Hood (1991) states that:

The results show that the model size does have a significant impact on the results. This is extremely important when considering that most users will only create one model of their catchment and will not be able to test the sensitivity of the catchment to sub-catchment breakdown.

When a model is broken down into a finer model, this is done so linearly. That is to say that the area of the node in the coarser model will equate to the algebraic sum of the areas of the contributory nodes in the finer model. This is logical, as it ensures that the total catchment area is the same for all models.

Yet, as area is treated non-linearly in the storage equation, (it is raised to the exponent 0.52) and hence a new catchment breakdown will affect the overall storage of the catchment, as defined by Aitken's equation. The result is that the flow in the catchment will be affected by the catchment breakdown and one would expect the outflow to vary between models.

5.3 Theoretical Investigation of the Effect of the Number of Sub-areas in a WBNM Model

5.3.1 Introduction

The Watershed Bounded Network Model (WBNM) has two different types of storages that correspond to the two different types of sub-catchments comprising a catchment subdivided along watershed lines. These two types of sub-catchments are:

Ordered basins. These are complete sub-catchments and no water flows into them across any boundary. The lag applied to these basins is the same as that applied to whole catchments. This lag is termed K_B.

Interbasin areas. These are complete sub-catchments with a stream draining upstream areas flowing through them. Outflow from each interbasin area consists of runoff from both the upstream areas transmitted through the interbasin by its main stream and the runoff from the local sub-catchment. The storage effects and thus lag is assumed to be different for these two types of runoff. The lag for the runoff transmitted through the interbasin area is termed K_I .

The general form of the predictive equation for K_B and K_I is:

$$K = g\,A^{x} \qquad \text{Equation 5.2}$$

Where K is the lag (K_B or K_I), A is the sub-catchment area (km^2), and g and x are determined by comparing calculated model lags with recorded catchment lags.

The value of the ratio K_I/K_B was found by calibration on ten catchments to be 0.6. This ratio will be termed α.

5.3.2 The Ratio α

The value selected for α has an effect on the overall catchment lag. Consider a catchment that is made up of a number of sub-catchments. As each sub-catchment is added to the model the modelled total catchment lag will remain the same as the lag of the total catchment only if a value of α is selected for each sub-catchment added to maintain the total catchment lag at the correct value. If this value of α depends on the ratio of the area of the sub-catchment added (which can be termed the downstream catchment) to the total upstream catchment then it is of no consequence how many sub-catchments are upstream of the point being considered. The problem reduces to a consideration of two sub-catchments.

A theoretical derivation of α to retain correct total catchment lag can be undertaken, based on the principles of superposition of moments of the outflow hydrographs from the individual sub-catchments. Boyd (1985) explains these principles. The lag of a hydrograph is the volume weighted mean lags of any number of hydrographs that have been summed together. This is true only if the catchment response is linear, that is the catchment lag remains constant for all flows.

Consider then a linear catchment. The catchment is divided into two sub-catchments, having area Au and Ad (upstream and downstream sub-catchments) with the hydrograph being routed through a mainstream storage in the downstream sub-catchment. If rainfall and losses are the same on both sub-catchments, the runoff volume is proportional to the area of the catchment or sub-catchment. The hydrograph lags of these sub-catchments are Ku and Kd, and the flow volumes Vu and Vd. For WBNM the hydrograph lag ratio for the mainstream storage is α, giving a lag of αKd.

$$Total\ Catchment\ Lag = \frac{Kd\,Vd + (Ku\,Vu + \alpha\ Kd\,Vu)}{Vd + Vu} = \frac{Kd\,Ad + (Ku\,Au + \alpha\ Kd\,Au)}{Ad + Au}$$ Equation 5.3

Therefore if $K = gA^x$:

$$g\,(Ad + Au)^x = \frac{g\ Ad^x\ Ad + g\ Au^x\ Au + \alpha\ g\ Ad^x\ Au}{Ad + Au}$$ Equation 5.4

And solving for α:

$$\alpha = \frac{(Ad + Au)^{1+x} - (Ad^{1+x} + Au^{1+x})}{Ad^x\ Au}$$ Equation 5.5

The value of α will thus depend only on the ratio Au/Ad, as x is a constant. This indicates that the number of sub-catchments making up Au is not a significant factor in the value of α required to maintain the value of the total catchment lag at the value expected for the total catchment.

A complication arises from non-linearity in the catchment, in that the storage lag is not constant for all flows, but is a function of the catchment outflow. The sub-catchment lag is then related to the outflow from the sub-catchment.

Thus sub-catchment hydrograph lag $K = g\ A^x\ (q_m)^n$, where q_m is the mean storm event outflow from the sub-catchment under consideration. Regional regression analysis has found that in general catchment outflow can be related to area for a rainfall input, with a relationship $q_m = cA^d$. Stewart and Ashkanasy (1982) concluded that in general that d was in the range of 0.7 to 0.8. Eusuff (1995) found a range of d between 0.73 and 0.82, also for the Mount Lofty Ranges

If this extra term is included in the relationship then:

$$\begin{aligned} K &= g\,A^x\ (c\,A^d)^n \\ &= g\,c^n\ A^{(x+dn)} \end{aligned}$$ Equation 5.6

Where g c^n is a constant. This can be substituted in Equation 5.3 and the ratio α then becomes:

$$\alpha = \frac{(Ad+Au)^{1+x+dn} - (Ad^{1+x+dn} + Au^{1+x+dn})}{Ad^{x+dn}\,Au} \qquad \text{Equation 5.7}$$

To confirm this relationship values of x and n can be substituted from the WBNM model. The storage lag is by definition proportional to $A^{0.57}$ and $q^{-0.23}$ thus x = 0.57 and n= - 0.23. An average value for d of 0.7 is used. Table 5-1 gives the value of α for the range of relative sizes of sub-catchments.

Table 5-1 Expected Values of the Ratio α For Two Sub-Catchments

Ad	Au	α
0	1	Not Applicable
0.1	0.9	0.28
0.2	0.8	0.40
0.3	0.7	0.49
0.4	0.6	0.58
0.5	0.5	0.66
0.6	0.4	0.73
0.7	0.3	0.82
0.8	0.2	0.91
0.9	0.1	1.03
1	0	Not Applicable

A WBNM model is made up of many sub-catchments. If all sub-catchments are of the same size a relationship for α can be derived in terms of the number of sub-catchments upstream of the sub-catchment of interest.

Assume that the number of sub-catchments upstream of the sub-catchment of interest is n_s. Then in this case:

$$Au/Ad \approx n_s \qquad \text{Equation 5.8}$$

Or:

$$Au \approx n_s Ad \qquad \text{Equation 5.9}$$

Substituting in Equation 5.7 for A_u and simplifying gives:

$$a \approx \frac{1}{n_s}\left[(1+n_s)^{1+x+dn} - n_s^{\,1+x+dn} - 1\right] \qquad \text{Equation 5.10}$$

where n_S is the number of sub-catchments upstream of the sub-catchment of interest

A typical catchment was also examined to quantify the value of α. The catchment to the Aroona Dam had previously been modelled using the RORB model (Kemp, 1989), and thus had sub-catchment information available. Figure 5-2 shows the layout of sub-catchments. The values of α were calculated using Equation 5.7 with a value of d of 0.71, based on a derived regional flood frequency analysis by Kemp (1989) which included the catchment in the derivation. Table 5-2 gives the derived values for α.

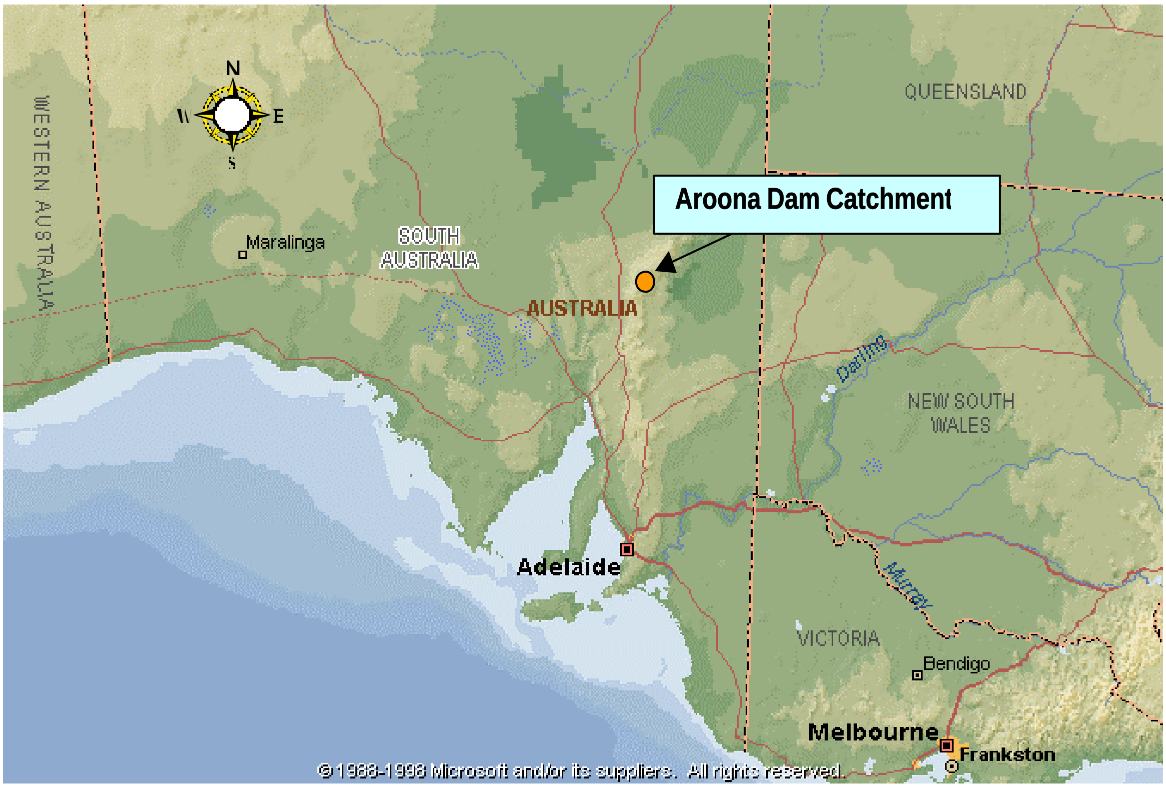


Figure 5-1 Location of the Aroona Dam Catchment

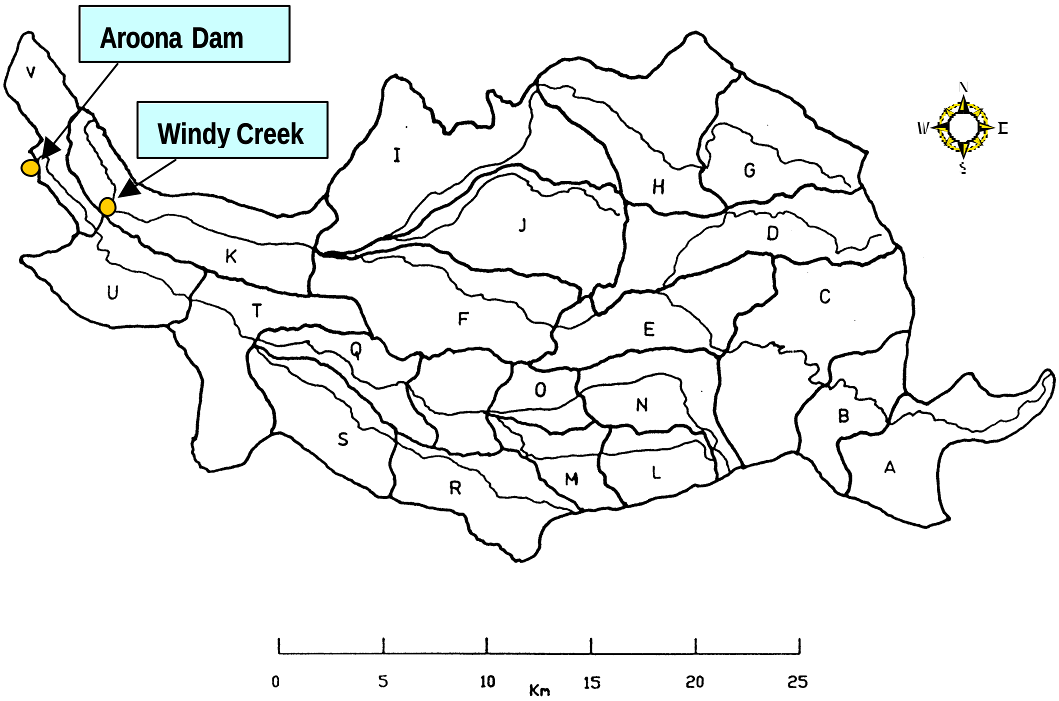


Figure 5-2 RORB Model Layout for the Aroona Dam Catchment

These values of α can be compared with the default value of 0.6, derived by catchment calibration. Figure 5-3 is a plot of the values of α versus the number of upstream sub-catchments and confirms the strong relationship revealed by Equation 5.10.

Table 5-2 Aroona Dam Catchment α

Sub-catchment	Number of Sub-catchments upstream	α from Equation 5.7	α from Equation 5.10
A	0		
B	1	0.57	0.65
C	2	0.61	0.52
D	0		
E	4	0.48	0.40
F	5	0.40	0.36
G	0		
H	1	0.70	0.65
I	2	0.60	0.52
J	0		
K	10	0.25	0.27
L	0		
M	1	0.68	0.65
N	0		
O	1	0.65	0.65
P	4	0.40	0.40
Q	5	0.37	0.36
R	0		
S	1	0.65	0.65
T	8	0.39	0.29
U	9	0.31	0.28
V	19	0.16	0.20
mean		0.48	0.46

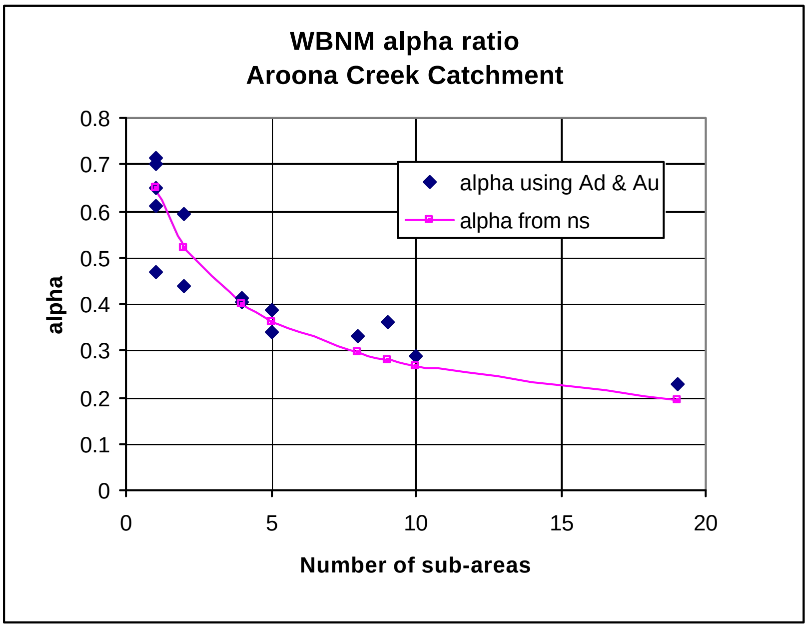


Figure 5-3 Aroona Creek Catchment α Values

As the number of sub-catchments increases the mean value of α to be used with the model should decrease to maintain the same total catchment lag. Using a constant value of α as in WBNM will overestimate sub-catchment inflow lag and thus predict greater total catchment lag time with an increasing number of sub-catchments.

Boyd (1985) has confirmed this effect in his investigation into the effect of catchment sub-division on runoff routing models. Boyd concluded for all five catchments investigated with WBNM that as the number of sub-catchments increased the predicted peak flow was found to decrease and the time to peak increased towards a stable value.

It can be seen also that if the rainfalls or losses on the two sub-catchments considered in Section 5.3.2 is different the ratio α cannot be determined theoretically, as the runoff volume is now a function of both sub-catchment rainfall and area.

5.3.3 Summary

It has been shown that the required storage lag ratio K_I/K_P (designated α) to maintain total catchment lag can be theoretically determined by the superposition of the moments of sub-catchment hydrographs. The required value is a function of the ratio of the sub-catchment area to the upstream area, and following from this the total number of sub-catchments in the model, if the sub-catchments are of a similar size. As the WBNM model uses a constant value the number of sub-catchments will affect the catchment lag, and thus the predicted flows.

5.4 RAFTS

5.4.1 Introduction

As part of an investigation of the relationships between the models a direct relationship between the RORB delay time coefficient k_c and the RAFTS storage delay time coefficient B for a single node RAFTS model was found. The relationship is:

$$B = 0.2k_c \quad \text{Equation 5.11}$$

If

$$n = m - 1 \quad \text{Equation 5.12}$$

The relationship was tested for a flood event on Aldgate Creek (AW503509). Aldgate Creek lies within the Adelaide Hills, and has a catchment area of 7.96 km^2. Figure 5-4 shows the good comparison between the RORB model and the one node (single sub-catchment) RAFTS model, using Equations 5.11 and 5.12.

However when the derived B value was used in a 10 node RAFTS model it became clear that the storage delay time in the model was clearly different to that of the 1 node model, and did not give a result similar to the RORB model. The predicted hydrograph from a 10 node model is also shown on Figure 5-4.

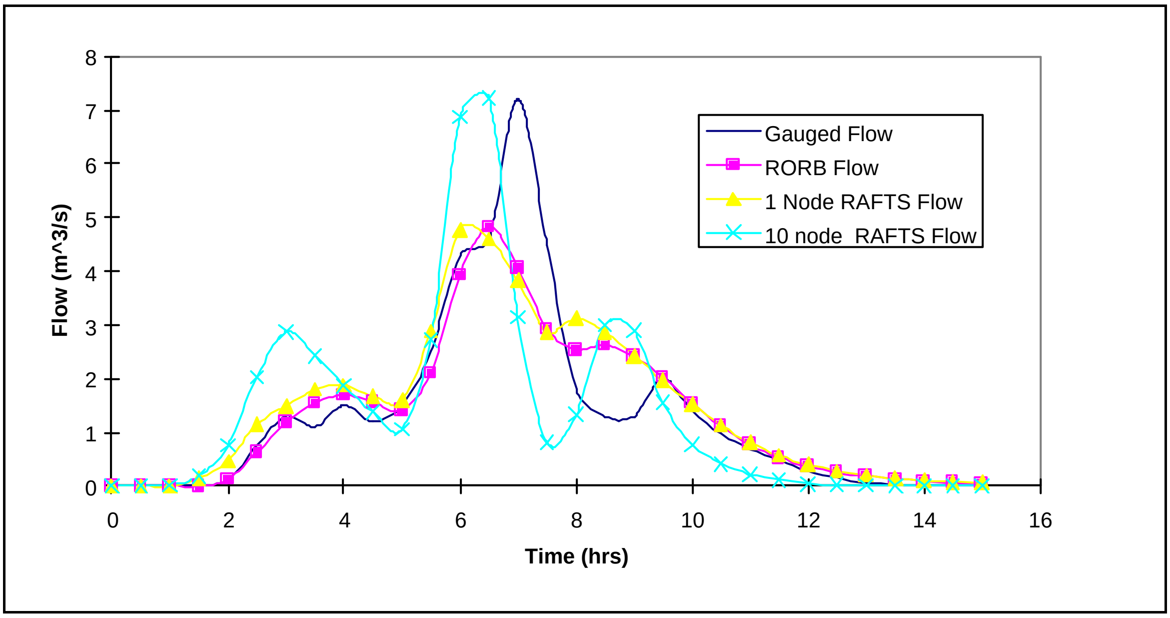


Figure 5-4 Aldgate Creek 17/6/77 Showing the Effect of Number of Nodes in the RAFTS Model

The 10 node RAFTS model has a lower storage delay time. Channel storage was used in the 10 node model, with the channel sections and Manning's n values from a flood study of Aldgate Creek by Kinhill Engineers, (1993). The storage delay time coefficient for the catchments contributing to individual nodes was calculated assuming that:

$$B \propto A^{0.67} \qquad \text{Equation 5.13}$$

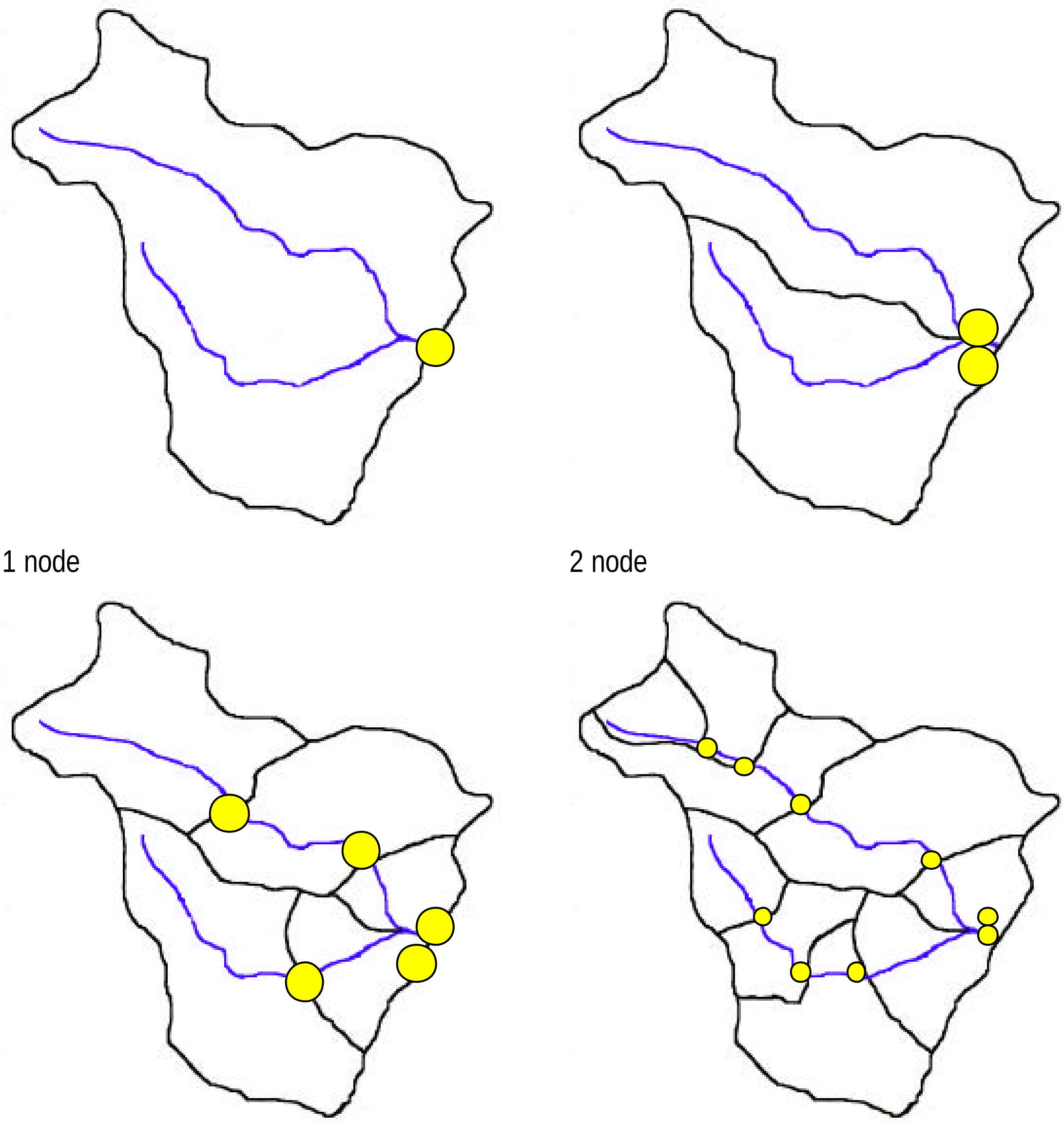
This relationship is evaluated from the RORB k_c coefficient previously derived for South Australia by the Engineering and Water Supply Department (1986).

It was considered that the difference between the one and the 10 node model may be due to the use of equation 5.13 in place of the default equation in RAFTS, or the poor definition of the channel translation and storage. For this reason further investigation was carried out, this time using the RAFTS default equations, to confirm the reason for the effect.

5.4.2 Confirming the Effect

A series of RAFTS models were set up for the Aldgate Creek catchment with 1, 2, 5 and 10 nodes. The Aldgate Creek catchment was chosen because data were available on channel sections within the catchment. (Kinhill Engineers, 1983)

The models were set up with both translations of hydrographs between nodes and full channel routing using the known channel properties. The default equation for the storage delay time coefficient B was used.



1 node 2 node

5 node 10 node

Figure 5-5 Aldgate Creek RAFTS Sub-division

A standard rainfall storm of 2 hours duration with an Average Recurrence Interval of 100 years was used. Rainfall intensities and temporal patterns were derived from Australian Rainfall and Runoff (Institution of Engineers, Australia, 1987). The standard storm was chosen as a typical storm that would be used with the RAFTS model to determine design flows for the catchment.

Channel translation velocities of 1, 1.5 and 2.5m/sec were examined, as well as Muskingum-Cunge channel routing using the known channel properties (shape and Manning's n values)

Figure 5-6 and Figure 5-7 compare the peak flows and time to peak for the three channel translation velocities and channel routing. The peak flow and time to peak ratios are plotted, being the ratio of the predicted peak flow and the time to peak relative to a one node model. Compared with a single node model they show a increase of up to 32% in predicted peak flow and a range of +17% to –12% in the time to peak. Figure 5-8 shows the hydrographs predicted for a translation velocity between nodes of 1m/sec.

The figures clearly indicate that both peak flow and time to peak are sensitive to the number of nodes used in the model, with links modelled both by Muskingum-Cunge channel routing and hydrograph translation. There is no channel translation velocity that can be chosen that will maintain constant peak flow and time to peak. Although the translation velocity of 1.5m/sec retains a reasonable time to peak, the magnitude of the predicted peak flow is not maintained.

The above investigation confirms the findings of Hood (1991), that the flow predicted by the RAFTS model depends on the number of nodes or sub-catchments in the model.

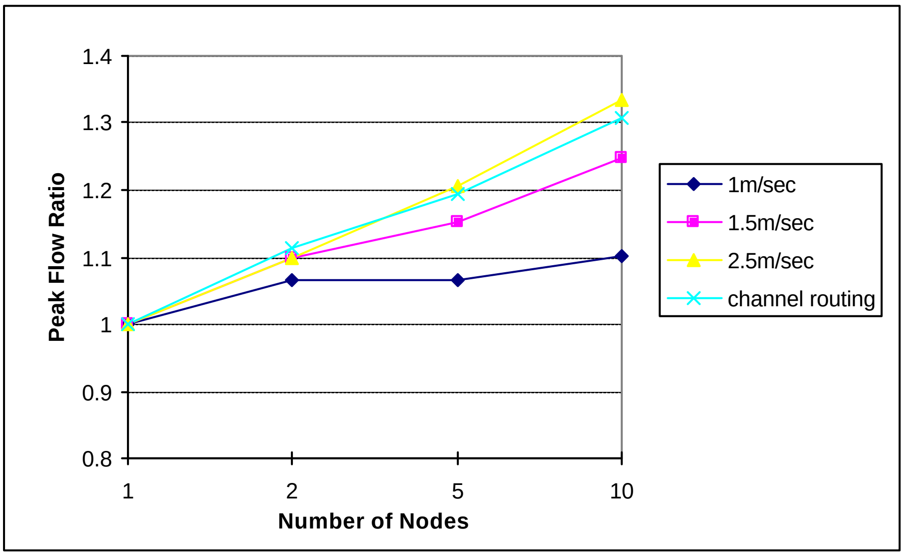


Figure 5-6 Aldgate Creek RAFTS Model Ratio of Peak Flow to Peak Flow for One Node Model

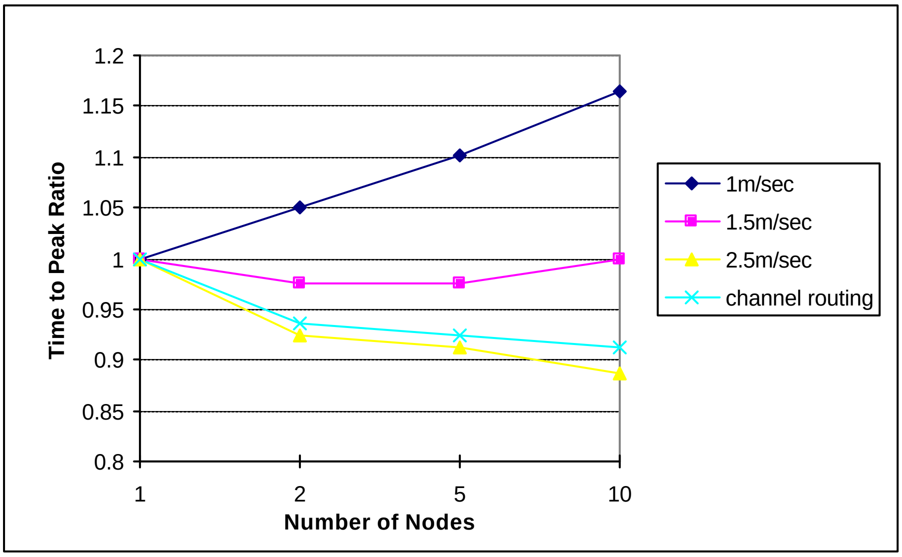


Figure 5-7 Aldgate Creek RAFTS Model Ratio of Time to Peak with Time to Peak for One Node Model

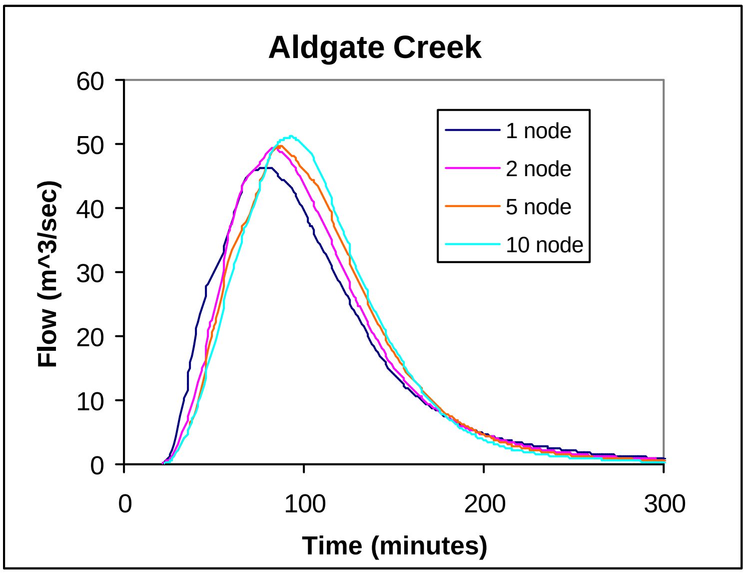


Figure 5-8 Aldgate Creek - RAFTS Model Results Showing the Effect of the Number of Nodes

5.4.3 The Reasons for the Effect

5.4.3.1 Total Catchment Lag

Having confirmed that the number of nodes of the model does affect the predicted outflow the reasons for the effect are now examined.

To simplify the analysis it will be assumed that hydrograph translations are used for the links, in other words the contribution of each node is simply translated to the catchment outlet. There are no storages in series.

It must be first assumed that the runoff volumes from sub-catchments are proportional to the sub-catchment area. This will be the case when uniform rainfall and losses are applied to the catchment, as was the case in the Aldgate Creek catchment. The overall lag time is defined as the time between centroid of rainfall excess and the centroid of the resultant surface runoff. The lag time of two sub-catchments contributing hydrographs of volume V_1 and V_2 to a common node, and having lags t_1 and t_2 can be determined because of the proportionality between volume and area:

$$K = \frac{[V_1 K_1 + V_2 K_2]}{V_1 + V_2}$$ Equation 5.14

Since the runoff volume is proportional to the catchment area the overall catchment lag can be related to catchment areas A_1 and A_2.

$$K = \frac{[A_1 K_1 + A_2 K_2]}{A_1 + A_2} \qquad \text{Equation 5.15}$$

If there is a translation time in the link of t_2 between the nodes of the individual sub-catchments with A_2 being upstream of A_1 then the overall catchment lag is given by:

$$K = \frac{[A_1 K_1 + A_2 (K_2 + t_2)]}{A_1 + A_2} = \frac{[A_1 K_1 + A_2 K_2 + A_2 t_2]}{A_1 + A_2} \qquad \text{Equation 5.16}$$

Or in the general case, with sub-catchments 1 to n with translation times to the catchment outlet of t_1 to t_n:

$$\begin{aligned} K &= \frac{[A_1 K_1 + A_2 (K_2 + t_2) + \ldots\ldots + A_n (K_n + t_n)]}{A_{total}} \\ &= \frac{[A_1 K_1 + A_2 K_2 + \ldots. + A_n K_n]}{A_{total}} + \frac{[A_2 t_2 + \ldots. + A_n t_n]}{A_{total}} \\ &= \frac{[A_1 K_1 + A_2 K_2 + \ldots. + A_n K_n]}{A_{total}} + tr_m \end{aligned} \qquad \text{Equation 5.17}$$

Where tr_m is the mean translation time within the catchment. The first part of Equation 5.17 indicates that the overall lag time due to the hydrographs at each node is equal to the area weighted mean lag time of all contributing node hydrographs.

The second part represents a term accounting for the time translation in the links within the catchment. It represents the area weighted mean translation time to the outlet for all nodes within the catchment.

In a RAFTS model the mean translation time to the outlet for all nodes will remain relatively constant, no matter what number of nodes are used. However for the area weighted mean lag time of all node hydrographs to remain constant the time lag of the hydrographs contributing to each node would have to be constant, and not vary with the contributing area.

This is not the case with the RAFTS model, because the hydrograph time lag at each node varies with catchment area, slope and in most cases outflow, due to non-linearity in catchment response.

5.4.3.2 The Magnitude of the Effect

It is possible to quantify the effect of the number of nodes in a model, by considering the mean node hydrograph lag time for the model. Catchment slope will be neglected, on the assumption that this is a simplified catchment having uniform slope.

The modelled storage delay time for each of ten storages in series contributing to each node in a RAFTS model is given as a non-linear relationship with the area contributing to the node with the form:

$$\begin{aligned} k &= aA^b s_c^{\,c} q^n \\ &= Bq^n \end{aligned} \qquad \text{Equation 5.18}$$

Where	k	is the sub-catchment storage delay time (hrs)
	A	is the area (km^2)
	s_c	is the main drainage slope (%)
	q	is the instantaneous flow (m^3/sec)
	a,b,c,n	are constants

The hydrograph lag time resulting from the ten storages in series contributing to each node can be related to the mean flow through the ten storages. The mean storage delay time is 5k (being half the total storage delay time of the ten storages in series), and the resultant hydrograph lag time is:

$$\begin{aligned} t &= 5Bq_m^n \\ &= 5aA^b s_c^c q_m^n \end{aligned} \qquad \text{Equation 5.19}$$

Where t is the node hydrograph lag time

q_m is the mean flow through the storages contributing to the node

The division of the catchment into sub-catchments that contribute to the nodes has an effect both on the area contributing and the mean flow within the storages contributing to each node. The mean flow through the storages contributing to the node will be proportional to the area contributing to the node, ie. $q_m \propto A^d$, where A is the area contributing to the node and d is a constant. If the total catchment area is A_t, the number of nodes is NN, catchment slope is constant, and all sub-catchments contributing are of equal size then from Equation 5.19 the node hydrograph lag time proportional to the product of the area and the mean node flow as given in equation 5.20:

$$t \propto (A_t / NN)^b ((A_t / NN)^d)^n = \frac{A_t^{b+dn}}{NN^{b+dn}} \qquad \text{Equation 5.20}$$

The node lag time will be inversely proportional to NN^{b+dn}. Equation 5.17 indicates that the total catchment lag time is equal to the area weighted mean node lag time plus the area weighted mean translation time to the outlet for all nodes. As the node lag time varies with the number of sub-catchments (NN) it can be seen that the catchment lag time will also vary with the number of nodes.

A factor BX is included in the RAFTS model to globally multiply the storage parameter B at each node by the same amount. The factor NN^{b+dn} can be incorporated into the model as BX, which will retain the same hydrograph lag for varying numbers of nodes. It could be expected however that for a small number of nodes, variation will occur due to changes in the mean node translation time to the outlet. The factor cannot be used as a correction for the RAFTS model, it merely demonstrates the magnitude of the effect of the number of nodes.

Also this storage delay time coefficient multiplier BX is correct only for equal sized sub-catchments which is not usually the case with catchment models.

5.4.4 The Implications

5.4.4.1 Generalised Storage Relationships

The fact that the number of nodes or sub-catchments in a model has an effect on the predicted hydrograph leads to the conclusion that any generalised storage relationship should be applied only to models having the same number of nodes or sub-catchments. In the case of the relationship found between the RORB k_c and m and RAFTS B and n it can be stated that it is only applicable with a RAFTS model having only one node.

Aitken (1975) derived the default equation for the storage parameter B in RAFTS. Aitken's relationship for storage lag was derived for a LRRM model, which is equivalent to a single node RAFTS model. The relationship therefore should not be applied to a RAFTS model with more than one node without consideration of the effects of the number of nodes.

5.4.4.2 Self-Consistency

The concept of self-consistency needs explanation. Yu and Ford (1989) indicate that if a model is self-consistent then:

- *Locally specified storage relationships should be independent of the dimension of the entire catchment; and*
- *The subjectivity in the layout of the network itself should have no effect on the output hydrographs for sub-catchments within the catchment.*

The RAFTS model is clearly not self-consistent on the basis of the above statements. Although locally specified storage relationships are not affected by the size of the entire catchment the number of nodes and thus the layout of the model clearly has an effect on the output hydrograph for sub-catchments within the total catchment. The output hydrograph will clearly depend on the number of nodes upstream of the point being considered.

If a RAFTS model is calibrated to known data at the outlet the flows predicted at internal nodes will be incorrect, due to the differing number of nodes contributing to the point of interest.

The normal method of calibration of the RAFTS model is by the use of the default equation for B for each sub-catchment, and the application of the global multiplier BX to the model to adjust the total storage in the model to match the gauged hydrograph. However storage will then be overestimated at each node within the model, resulting in the predicted peak flow being less than would be gauged at the node.

This effect can be illustrated with a gauged storm event on the Aroona Creek catchment. The Aroona Creek catchment in the Northern Flinders Ranges has two gauging stations, one at the Aroona Dam (contributing area = 696km^2) and one upstream within the catchment, on Windy Creek (contributing area = 442km^2).

A RORB model was calibrated on the Aroona Creek catchment for an event on 24 December 1988, with the default RAFTS storage parameters.

When the RAFTS model was calibrated with the BX multiplier at the Aroona Dam gauge, a best fit BX of 0.46 was obtained. As expected this model however did not correctly predict flows at the upstream Windy Creek gauge which is modelled with 5 nodes. A different (and lower) storage multiplier had to be used at Windy Creek to provide a good fit.

Figures 5.13, 5.14 and 5.15 give the best fit at the Aroona Dam (BX = 0.46), the fit at Windy Creek for BX = 0.46 and the best fit for the Windy Creek catchment alone (BX = 0.35).

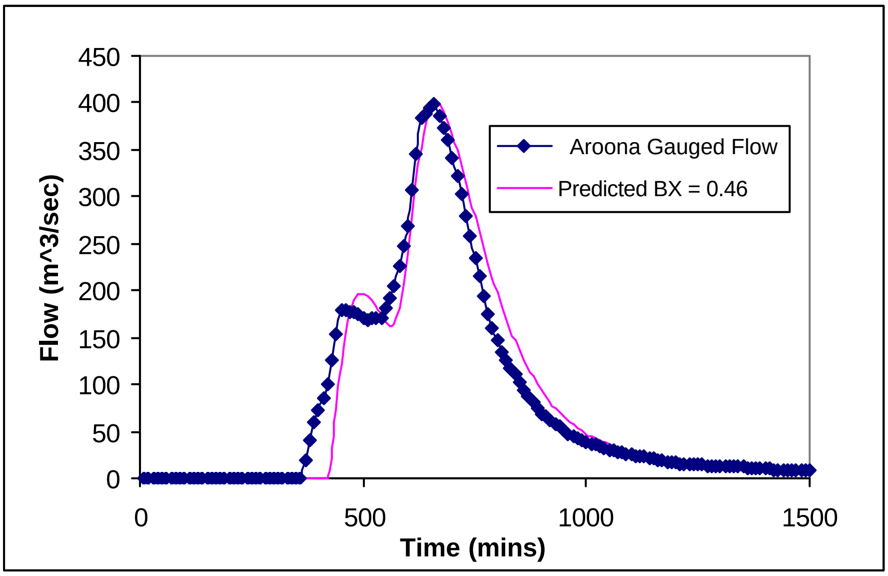


Figure 5-9 Aroona Dam 24/12/88, Best Fit BX = 0.46

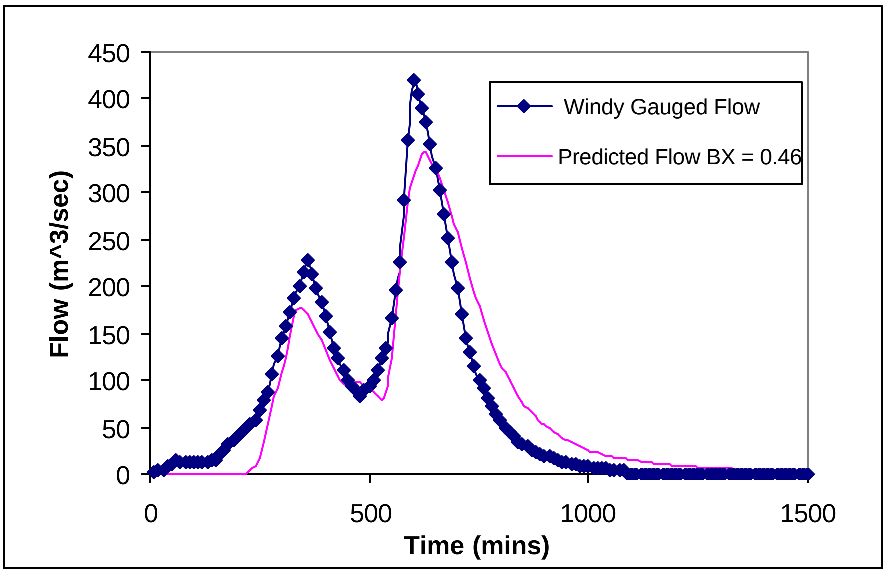


Figure 5-10 Windy Creek 24/12/88, BX = 0.46

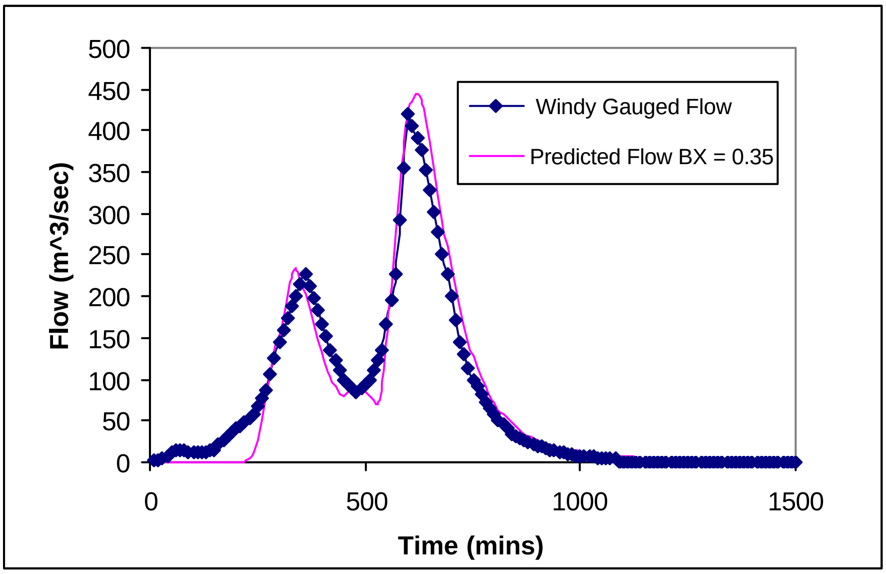


Figure 5-11 Windy Creek 24/12/88, Best Fit BX = 0.35

5.5 Summary

It has been confirmed by this investigation that the number of nodes in a RAFTS model does indeed have an effect on the predicted hydrograph.

A simple relationship has been derived for the RAFTS model that relates hydrograph lag to the number of sub-catchments or nodes.

A storage lag relationship derived for a catchment to a single node (such as Aitken's) should not be applied to a RAFTS model with a different number of nodes. The relationship found between the RORB k_c and m and RAFTS B and n is only applicable with a one node model.

Yu and Ford (1989) showed that self-consistency does not exist in the RORB model, and it has been shown that it does not exist in the WBNM model (or the number of upstream sub-catchments would not affect the output from a sub-catchment).

Thus none of the runoff routing models examined maintains self-consistency, and care must be exercised in the application of all the models with regard to the number of nodes or sub-catchments. In particular predicted flows at any internal nodes or sub-catchments within a calibrated model will not be correct, due to the differing number of sub-catchments above the point of interest.

> *Models should not be taken on trust. They need to be tested against recorded results, evaluated, and altered or refined as necessary. More urban catchment data are becoming available now, and users should expect that models are evaluated against this information.*
>
> *Geoffrey O'Loughlin (1993)*

6. ILSAX Modelling of Adelaide Urban Catchments

6.1 Introduction

Two gauged catchments were established in the Adelaide metropolitan area in the 1990s. These catchments give the opportunity to verify that the ILSAX model is applicable to Adelaide. It is desirable for the purpose of verification that the ILSAX model be detailed, but still as standard as possible in the selection of parameters such as the gutter flow factor, GUT and the pipe Manning's n value. In this way it can be expected that the model will be applicable to ungauged catchments using these same standard parameters.

For each catchment two years of data were examined and the largest storms chosen for analysis. A total of twelve storms were chosen for the Frederick Street catchment and eighteen for the Paddocks catchment. It was considered that this number would give a reasonable indication of the catchment response.

Figure 6-1 Location of the Glenelg and Paddocks Catchments

6.2 Glenelg Catchment

The Glenelg catchment is a fully urbanised catchment. Most of the development within the catchment area occurred during the late 1940s and 1950s, and it can be considered to be stable in terms of the runoff relationships. The greater part of the catchment is residential, with a smaller commercial component. Soils are sandy to silty clays containing some lime.

Slopes are low, with an average gutter gradient being 0.2% to 0.5%. The catchment is also such that it is reasonably isolated from external inflows during major events.

Instrumentation is via six pluviometers and three gauging stations within the catchment, having a total area of 191ha. Each pluviometer is elevated 3 metres above ground level on a 25cm diameter tower. The aim is to improve exposure and also to minimise the possibility of damage by vandals. Instrumentation at each gauging station includes an in-pipe flow measuring device (Detectronics IS 32 Surveylogger) and an additional depth transducer (Mindata). At one station (Frederick Street) a Montedoro Whitney, System Q flow and depth recorder is also installed. At the time ILSAX calibration was carried out only one station data was available for modelling, being the Frederick Street station.

The monitoring project is described by Argue et al (1994).

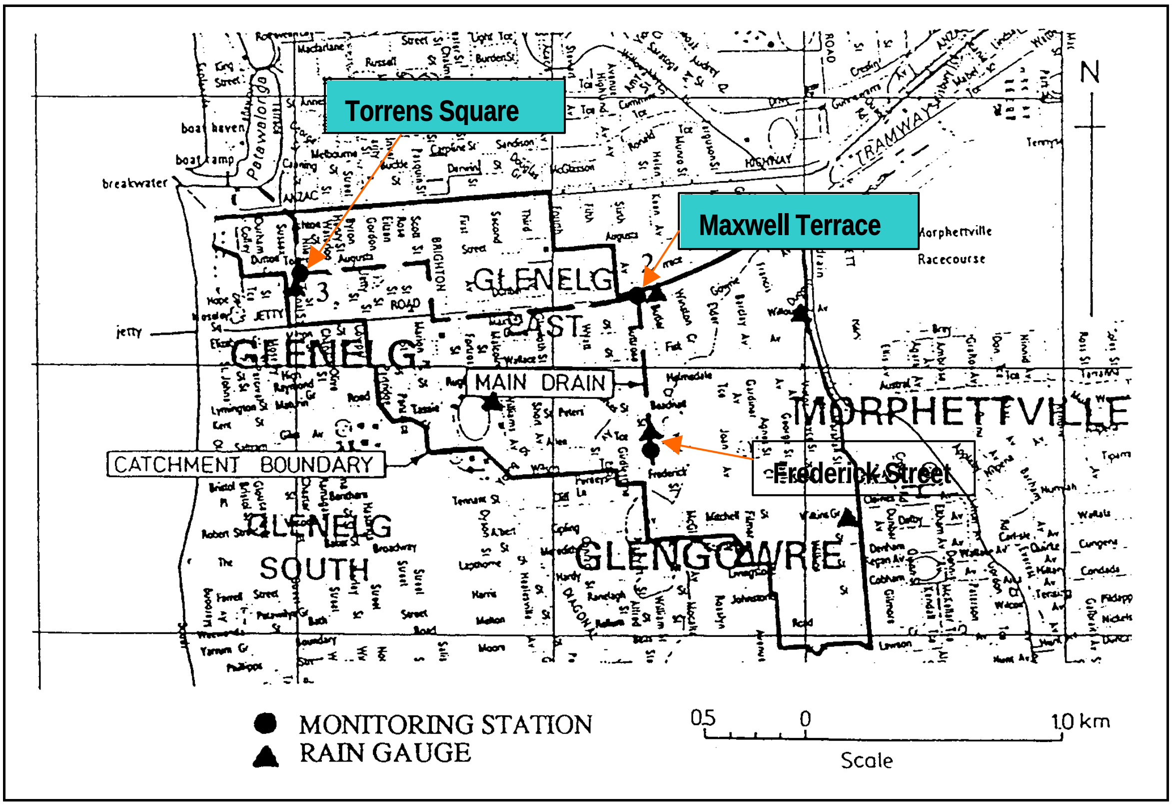


Figure 6-2 The Glenelg Catchment (after Argue et al, 1994)

Figure 6-3 View of the Glenelg Catchment

The catchment stations are given in Table 6-1:

Table 6-1 Glenelg Catchment, Monitoring Stations

Station	Number	Variable
Frederick Street	AW504561	Water level, velocity, rainfall
Maxwell Terrace	AW504554	Water level, velocity, rainfall
Torrens Square	AW504562	Water level, velocity
Coles car park	AW504565	Rainfall
Willoughby Park	AW504555	Rainfall
Morphett Arms hotel	AW504556	Rainfall
Bowling Club	AW504557	Rainfall

Bruce et al (1994) describes the methodology.of determining the contributing areas. Students from the University of South Australia surveyed a substantial part (31.36ha) of the catchment contributing to the first gauging station at Frederick Street (catchment area 48.7ha). Plans were produced for each property by digitising aerial photography. The properties were then visited to determine which of the areas could be considered as directly connected and supplementary paved. The results were summed for each sub-area connected to a pit, for use in the ILSAX model.

The following contributing areas were found by Bruce et al:

Total surveyed area	31.36ha
Impervious street & footpath	4.99ha
Directly connected roof area	4.40ha
Supplementary paved area	5.32ha
Pervious area	16.65ha

Leading to the following overall percentages of the surveyed area for input to the ILSAX model:

Directly connected impervious	30%
Supplementary paved	17%
Pervious	53%

The ILSAX model for the Glenelg catchment was developed such that it was in a form normally used with the default calculation of gutter flow and overland flow times, and inlet capacity. Contributing areas were based on the surveyed areas as above, either directly from the survey (for those areas surveyed) or with similar percentages contributing, depending on land use.

Every pit, pipe and overflow path within the catchment is modelled. This resulted in a complex model having around 350 sub-areas. The pipe file is included in the files described in appendix 1, along with the rainfall files for the events modelled. An electronic copy of the catchment plan is included on the CD.

Data regarding the drainage system of the area was obtained from design plans produced as part of the South West Suburbs Drainage Scheme in the 1960s and other plans from the City of Marion and the City of Glenelg. Field inspection supplemented these.

Gutter grades and road cross slopes were obtained by digital level capable of reading grade to the nearest 0.2%. Comments on individual input items are described below.

6.2.1 Gutter Flow Time

As stated previously, the gutter flow time in the ILSAX program is calculated using Manning's formula, with the default hydraulic radius of 60mm and roughness coefficient of 0.02. The program however allows for the use of differing mean hydraulic radius and roughness by the use of a factor GUT defined as follows:

$$GUT = \frac{Gutter\ Length}{Flow\ Time\ x\ s_g^{0.5}\ x\ 60.0} \qquad \text{Equation 6.1}$$

where s_g is the gutter slope in m/m
Gutter Length is in metres
Flow Time is in minutes

The GUT factor was calculated as recommended in the ILSAX manual using a formula for gutter flow recommended by the US Bureau of Public Roads (Searcy, 1969), as given in Equation 6.2.

$$GUT = \frac{0.375F\left[\left(\frac{Z_G}{n_g}\right)\left(d_g^{8/3} - d_P^{8/3}\right) + \left(\frac{Z_p}{n_p}\right)d_p^{8/3}\right]}{\left[\left(d_G^2 - d_p^2\right)Z_G + d_p^2 Z_p\right]/2} \qquad \text{Equation 6.2}$$

where Z_G is the reciprocal of the gutter cross-slope (m/m)
Z_p is the reciprocal of the pavement cross-slope (m/m)
n_g is the Manning's n of the gutter
n_p is the Manning's n of the pavement
d_g is the depth of flow at the gutter face (mm)
d_p is the depth of flow at the edge of the pavement (mm)
F is a flow correction factor, estimated by Clarke et al (1981) to be 0.8

The equivalent GUT factor for the default hydraulic radius and roughness is 7.66. To determine whether this default GUT factor is appropriate for the Glenelg catchment a survey was undertaken at six locations in the catchment and the GUT factor calculated for various flow depths. The factors calculated are shown in Table 6-2.

It can be seen from Table 6-2 that the GUT factor will vary from site to site, but the biggest variation is with flow depth. The time of concentration formula assumes constant flow depth which is a very simplistic assumption, as the actual GUT varies both in space (along the gutter as the flow depth increases) and with time during the storm.

The default GUT factor of 7.66 was used in the ILSAX runs as it is the recommended value, and is in the range of expected values as shown in the above table. The GUT factor is one of the factors that could be varied during the fit run to match the time of rise of the catchment, thus confirming whether the default value is appropriate.

Table 6-2 GUT factors determined for the Glenelg catchment.

Location	Z_G	Zp	n_g	n_p	d_G(mm)	GUT From Eqn. 6.2
1	13	22	0.012	0.014	50	6.37
	13	22	0.012	0.014	75	7.95
	13	22	0.012	0.014	100	9.45
	13	22	0.012	0.014	150	12.21
2	13	15	0.012	0.014	50	6.55
	13	15	0.012	0.014	75	8.33
	13	15	0.012	0.014	100	9.9
	13	15	0.012	0.014	150	12.7
3	13	22	0.012	0.014	50	6.37
	13	22	0.012	0.014	75	7.95
	13	22	0.012	0.014	100	9.45
	13	22	0.012	0.014	150	12.21
4	13	17	0.012	0.014	50	6.55
	13	17	0.012	0.014	75	8.26
	13	17	0.012	0.014	100	9.81
	13	17	0.012	0.014	150	12.58
5	13	20	0.012	0.014	50	6.44
	13	20	0.012	0.014	75	8.07
	13	20	0.012	0.014	100	9.58
	13	20	0.012	0.014	150	12.34
6	9	26	0.012	0.014	50	5.98
	9	26	0.012	0.014	75	7.25
	9	26	0.012	0.014	100	8.69
	9	26	0.012	0.014	150	11.48

6.2.2 Overland Flow Time

The surface roughness or retardance coefficient was first set in the model at a value of 0.3, the value for lawns being usually 0.17 to 0.48 (Woolhiser, 1975). Again this is a factor that can be varied during fit runs to match the time of rise of grassed or pervious area runoff.

6.2.3 Modelling the 1992 and 1993 Storms at Frederick Street

Data from the seven largest storms recorded in 1992 and the five largest storms of 1993 were fitted to the ILSAX model at the Federick Street gauging station (AW504561).

The fitting procedure was as follows:

- Storms with runoff from the impervious area only were identified, by examining the percentage runoff (runoff volume/rainfall volume);
- The 1992 storms having only an impervious area runoff component were fitted first, by the use of the sensitivity adjustment available within the ILSAX model to transfer directly connected impervious area to supplementary paved area. For example a –10% sensitivity adjustment transfers 10% of the directly connected impervious area to supplementary paved area, without affecting the total catchment area. A paved area depression loss of 1mm was used, as recommended by the ILSAX manual;
- The other storms were then modelled, using the best fit for the directly connected impervious area sensitivity adjustment. The initial loss for the impervious area was set to model the start of the rise of the gauged flow, and the initial loss for the pervious area was set to start the contribution from the pervious area where the fitted flow deviated from the gauged flow, assuming no pervious area runoff. Continuing loss on the pervious area was used to best model the total runoff volume. The apparent lag of the pervious area runoff was adjusted by altering the grassed area roughness value 'n'.

Table 6-3 summarises the storms fitted:

Table 6-3 Frederick Street Catchment Storms Modelled for 1992 and 1993

DATE	DURATION (mins)	PEAK FLOW (m^3/sec)	RAINFALL		RAINFALL VOLUME (m^3)	RUNOFF VOLUME (m^3)	VOLUMETRIC RUNOFF COEFFICIENT
			AW504561 (mm)	AW504556 (mm)			
3/7/92	200	0.336	10.8	11.6	5542	1383	0.250
11/7/92	295	0.128	9	8	4030	981	0.243
19/7/92	190	0.316	5.6	6.2	2939	784	0.267
1/8/92	230	0.306	9	8.6	4242	909	0.214
30/8/92	275	1.078	24.4	22.2	11106	3461	0.312
31/8/92	110	0.394	5.8	5	2542	647	0.255
18/12/92	450	1.242	39.6	39.2	19144	5837	0.305
24/05/93	150	0.322	7.6	6.4	3332	762	0.229
30/08/93	145	0.534	11.2	11.4	5515	1161	0.211
19/09/93	105	0.652	8.2	8.6	4116	970	0.236
30/09/93	170	0.312	5.8	5.6	2763	643	0.233
17/10/93	160	0.548	10.6	7.6	4241	989	0.233

The runoff volumes were first plotted against the rainfall volumes (derived by weighting the rainfalls between the two stations) to determine which storms had runoff from pervious areas. Figure 6-4 indicates that all storms but the two largest storms had a consistent volumetric runoff. The solid line indicating the predicted runoff volume based on the ten smallest storms is shown. This line represents a runoff coefficient of 0.24. The storms of 30/08/92 and 18/12/92 were above the normal value, indicating pervious area runoff.

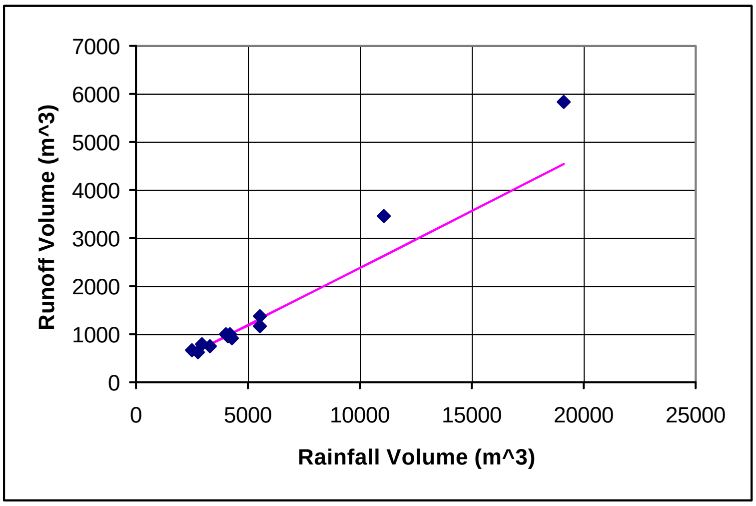


Figure 6-4 Frederick Street, Glenelg Catchment Storms Runoff Ratio

Table 6-4 summarises the sensitivity runs:

Table 6-4 Frederick Street Catchment - Summary of Sensitivity Runs.

Storm	Peak Flow (m^3/sec)	Volume (m^3)	Sensitivity Adjustment							
			0		-5%		-10%		-15%	
			Q(m^3/s)	V(m^3)	Q(m^3/s)	V(m^3)	Q(m^3/s)	V(m^3)	Q(m^3/s)	V(m^3)
03/07/92	0.343	1383	0.349	1472	0.331	1400	0.313	1327	0.296	1255
11/07/92	0.128	981	0.156	1079	0.149	1025	0.142	971	0.134	917
19/07/92	0.316	784	0.323	729	0.305	693	0.288	656	0.271	620
01/08/92	0.306	909	0.349	1129	0.332	1075	0.314	1019	0.295	962
31/08/92	0.349	647	0.425	770	0.404	732	0.368	563	0.361	658

It can be seen from Table 6-4 that there is no one directly connected impervious area sensitivity adjustment factor that can be applied to all storms to give a good match between predicted and observed flows and volumes. The effect of the constant initial loss was first investigated, but this was considered not to have a major effect. The sensitivity adjustment to the directly connected impervious area of -10% was chosen to model the storms with pervious area runoff on the basis that this adjustment was in the mid range of the best fits for the above storms, and by inspection produced the best overall fit of the shape of the hydrographs.

The other storm, including those having pervious area runoff were then modelled, using the predetermined sensitivity adjustment of -10%. Pervious area losses were chosen to best model the runoff volume and shape of the recorded hydrograph. It was discovered at this stage that there was too much lag on the pervious area runoff, and the roughness value was changed to n = 0.03. This matched the shape of the recorded hydrograph well.

The pervious area depression storage was set at 5mm, again as recommended by the ILSAX manual, and the soil type and antecedent moisture condition (AMC) adjusted to give the best fit.

Soil type 3 was found to be best, with an AMC of 2.5 for storm 5 and 2.0 for storm 7. A summary of the fit runs, using the standard sensitivity adjustment of -10% is as shown in Table 6-5:

Table 6-5 Frederick Street Catchment - Summary of ILSAX Fitting

Storm	Recorded		Predicted		Ratio predicted / Recorded		Grassed runoff
	Qp(m^3/s)	Volume(m^3)	Qp (m^3/s)	Volume(m^3)	Qp (m^3/s)	Volume(m^3)	
03/07/92	0.336	1383	0.287	1357	0.85	0.98	0%
01/08/92	0.306	909	0.314	1019	1.03	1.12	0%
11/07/92	0.128	981	0.142	971	1.11	0.99	0%
19/07/92	0.316	784	0.288	656	0.91	0.84	0%
30/08/92	1.078	3461	1.069	3158	0.99	0.91	11.0%
31/08/92	0.349	647	0.368	563	1.05	0.87	0%
18/12/92	1.242	5837	1.249	5801	1.01	0.99	21.6%
24/05/93	0.322	762	0.344	912	1.07	1.20	0%
30/08/93	0.534	1163	0.654	1350	1.23	1.16	0%
19/09/93	0.652	970	0.656	976	1.00	1.01	0%
30/09/93	0.312	644	0.255	617	0.82	0.96	0%
17/10/93	0.548	989	0.495	955	0.90	0.97	0%
				mean	0.99	1.00	
				Standard Deviation	0.11	0.11	

Appendix 2 contains plots of the recorded and predicted hydrographs. The result for storm of 18/12/92 is shown as an example in Figure 6-5.

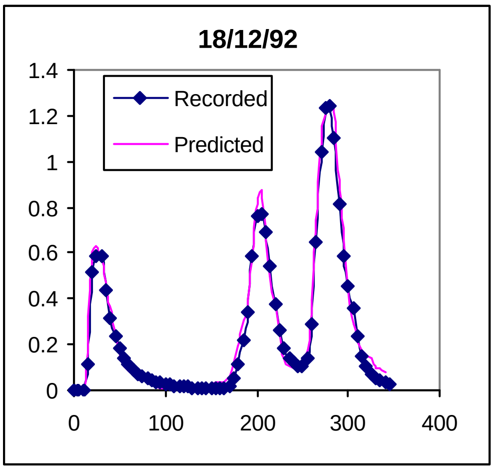
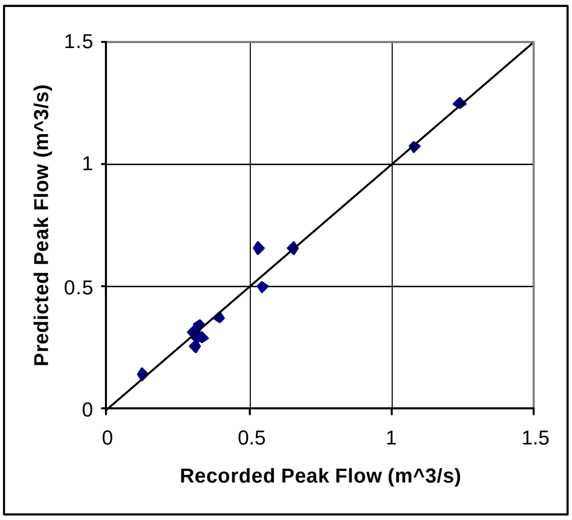


Figure 6-5 Frederick Street, Storm of 18/12/92



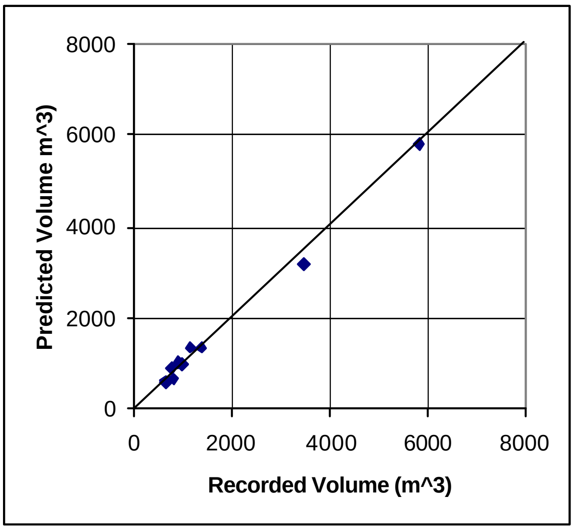


Figure 6-6 Frederick Street Catchment ILSAX Results

These results represent a good and consistent fit. The differences between storms could easily be explained by the fact that rainfall data is collected at only two stations within the catchment, so it could be argued that no model could be expected to model the storm events any better than the ILSAX model is.

The AMCs for the two storms with pervious area runoff were as expected, given the actual rain in the 5 days preceding the storm.

6.2.4 Frederick Street Catchment Summary

ILSAX performed well for the 1992 and 1993 storms, provided that the directly connected impervious area was reduced by 10%, with this area being transferred to the supplementary paved area. The mean predicted peak flow rate and volume was then within 5%.

6.3 Paddocks Catchment

The Paddocks catchment is also situated within the Adelaide metropolitan area, to the north east of the city. The catchment is predominantly residential, and can be also considered to be in a mature state, with little further development. Development occurred later than the Glenelg catchment, as most of the development occurred in 1950s to 1960s. The catchment area is 76ha. Soils are described as sandy to clay soils with abundant lime.

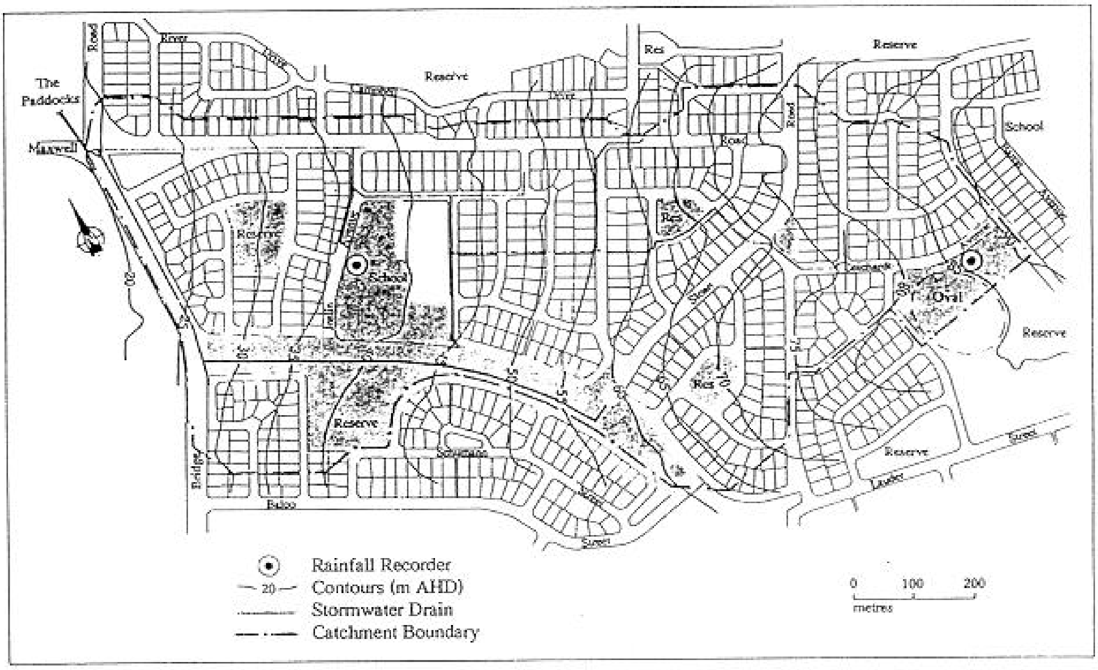


Figure 6-7 Paddocks Catchment (after Engineering & Water Supply Dept, 1993)

The catchment is described in detail in the Engineering & Water Supply Department report "The Paddocks" (1993). The average slope in the catchment is 5%, which is greater than the Glenelg catchment. The Department for Water Resources (Previously part of the Engineering & Water

Supply Department) carries out monitoring of the catchment, although the City of Salisbury and the South Australian Government Catchment Management Subsidy Scheme contributed to the cost of the installation of the pluviometers. There is a single gauging station at the outlet of the piped system, with a gauging weir forming the control. Two pluviometers are situated in the catchment mounted on towers, similar to those in the Glenelg catchment. The gauging station is a flat 'V' weir 10 metres downsteam of the catchment discharge pipe.

Figure 6-8 View of the Paddocks Catchment

The rainfall and flow monitoring stations are listed in Table 6-6:

Table 6-6 Paddocks Catchment, Monitoring Stations

Station	Number	Variable
Paddocks inlet	AW504546	Water Level
Leichardt Avenue	AW504566	Rainfall
Joslin Avenue	AW504567	Rainfall

The City of Salisbury carried a survey out of contributing areas and constructed the ILSAX model. The following contributing areas were determined for the total area:

Directly connected	26%
Supplementary paved	16%
Pervious	58%

6.3.1 The ILSAX Model

The ILSAX model for the Paddocks catchment was developed by the City of Salisbury, to the same standard and general specifications as the Glenelg model. The derivation of the model is covered in a separate report (Salisbury City Council, 1994). An electronic copy of the catchment plan is included on the CD with other thesis files.

6.3.2 The Storms Modelled

Data was obtained from the then E&WS Department in December 1993 for all recorded storms producing an outflow at the gauging station of more than 0.75 m3/s. This enabled a reasonable number of storms to be modelled.

A Plot of the rainfall volumes versus runoff volumes (Figure 6-9) indicated that no storms had obvious runoff from pervious areas. The line indicating the mean volumetric runoff ratio is also shown on Figure 6-9. This finding was unusual, given that there were two storms (19/12/92 and 14/12/93) that had rainfall intensities approaching the 10 year Average Recurrence Interval.

6.3.3 Initial Calibration

The fitting was carried out in a similar manner to that of the Frederick Street catchment.

The first runs were carried out with no sensitivity adjustment to the directly connected impervious area. When these runs were examined it was decided to reject several storms from the fitting procedure to determine the contributing impervious area. If these storms were included it would have reduced the confidence in the results.

Table 6-7 Storms Modelled in the Paddocks Catchment.

DATE	DURATION (mins)	PEAK FLOW (m^3/sec)	RAINFALL AW504566 (mm)	RAINFALL AW504567 (mm)	RAINFALL VOLUME (m^3)	RUNOFF VOLUME (m^3)	VOLUME RUNOFF COEFF.
08/10/92	40	0.960	8.6	7.8	6 050	1 574	0.260
08/10/92	80	1.286	8.4	11.0	7 708	2 275	0.295
17/11/92	50	2.230	12.0	13.7	9 940	2 316	0.233
21/11/92	50	0.771	6.0	5.4	4 148	984	0.234
03/12/92	28	1.407	6.4	5.9	4 553	955	0.219
18/12/92	20	1.453	7.6	6.6	5 190	1 124	0.217
19/12/92	30	2.464	18.6	18.9	14 164	3 164	0.223
24/01/93	20	0.843	3.2	3.2	2 409	763	0.317
27/02/93	60	0.860	7.6	7.4	5 616	1 395	0.248
21/05/93	60	1.378	11.0	8.4	6 899	1 448	0.210
31/05/93	20	0.831	2.6	2.6	1 958	465	0.238
03/06/93	58	1.144	11.2	10.2	7 901	1 632	0.207
11/06/93	60	0.943	2.6	4.6	3 022	648	0.214
30/08/93	40	1.391	9.6	10.7	7 814	1 793	0.229
17/10/93	16	1.048	6.1	4.1	3 529	629	0.178
18/10/93	40	1.054	6.2	5.0	4 030	802	0.199
13/12/93	30	1.670	12.8	7.8	6 977	1 379	0.198
14/12/93	80	1.797	30.4	29.0	22 144	5 572	0.252

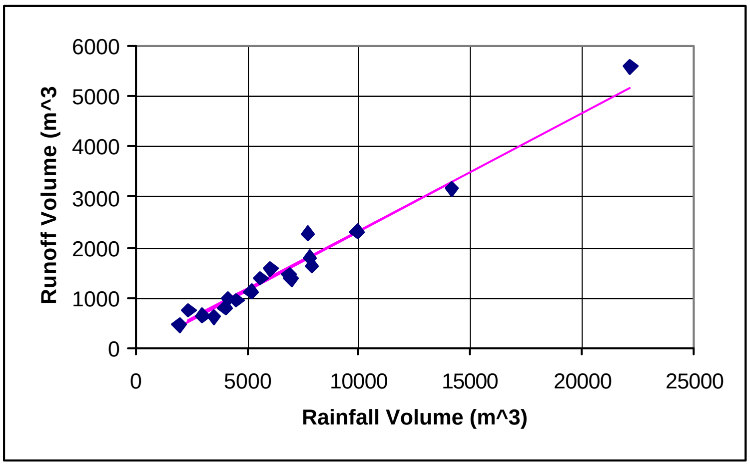


Figure 6-9 Paddocks Catchment Volumetric Runoff

The storms of 24/01/93 and 31/05/93 showed very poor fits, with the predicted flows being only approximately 50% of the actual. These two storms also have the smallest rainfall of all events selected, so the pluviometer record was less likely to be representative of total catchment rainfall. It may also be that in the case of the January storm evaporation had some influence. The storm of 14/12/93 was also rejected as the shape of the hydrograph suggested that there had been some blockage occurring in the pits or pipe system, leading to a reduction in peak flow, with water being released after the peak had occurred.

The initial runs also indicated that the shape of the predicted hydrograph was not good, with a trend for the predicted hydrographs to show timing error, with the predicted hydrographs following the measured hydrographs. This indicated that the flow times were over predicted, and that flow times in either the gutter or pipe needed to be reduced. The results are shown in Table 6-8:

Table 6-8 Paddocks Catchment ILSAX Fit, No Sensitivity Adjustment

Storm Date	Predicted Runoff (m^3)	Recorded Runoff (m^3)	P/ R	Predicted Peak Flow (m^3/s)	Recorded Peak Flow (m^3/s)	P/R
3/10/92	815	955	0.853	0.925	1.407	0.657
8/10/92	1612	1574	1.024	0.735	0.96	0.766
8/10/92	2065	2275	0.908	1.078	1.286	0.838
17/11/92	2611	2316	1.127	1.876	2.23	0.841
21/11/92	1118	984	1.136	0.677	0.771	0.878
18/12/92	1384	1124	1.231	1.529	1.453	1.052
19/12/92	3728	3164	1.178	2.051	2.464	0.832
27/02/92	1498	1395	1.074	0.725	0.86	0.843
21/05/93	1641	1448	1.133	1.329	1.378	0.964
3/06/93	2113	1632	1.295	1.272	1.144	1.112
11/06/93	640	648	0.988	0.720	0.943	0.764
30/08/93	1901	1793	1.060	1.304	1.391	0.937
17/10/93	739	629	1.175	0.877	1.048	0.837
18/10/93	873	802	1.089	0.976	1.054	0.926
13/12/93	1658	1379	1.202	1.588	1.67	0.951
	mean		1.098			0.880
	Standard deviation		0.119			0.115

It can be seen that the runoff volume is being overestimated, and the peak flow underestimated. Figure 6-10 shows the results of this initial run.

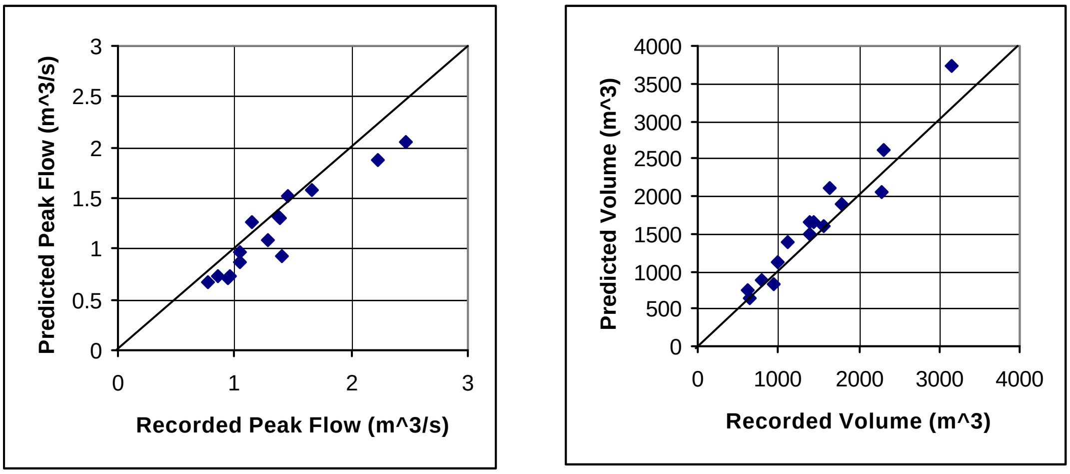


Figure 6-10 Paddocks Catchment Initial ILSAX Results

6.3.4 Calibration with PEST

The above initial fitting of the ILSAX model indicated that both the peak and the shape of the hydrograph were not being well modelled, and adjustments had to be made to both the pipe and gutter Manning's n values (which would change the shape of the hydrograph) and the directly connected impervious area (which will change the magnitude of both the predicted peak flows and the volume). With more than one parameter needing calibration the calibration of the model becomes more difficult, because of parameter interaction. For this reason an automatic calibration method was sought.

It was decided to use the parameter optimisation program, PEST (Watermark Computing, 1996) to calibrate the ILSAX model. This program provides an automatic and objective calibration method, by minimising the least squares error between the observed (recorded) hydrograph ordinates and the predicted ordinates.

It does this by taking control of the model and running it as many times as is necessary in order to determine this optimal set of parameters. The model user must inform PEST of where the adjustable parameters are to be found on the model input files. Once PEST is provided with this information, it can rewrite these model input files using whatever parameters are appropriate at any stage of the optimisation process. PEST must be taught how to identify those numbers on the model output files that correspond to the recorded hydrograph ordinates. Thus, each time it runs the model, PEST is able to read those model outcomes that must be matched observations. After calculating the mismatch between the two sets of numbers, and evaluating how best to correct that mismatch, it adjusts model input data and runs the model again.

It was decided for the Paddocks catchment to optimise the following parameters:

- The sensitivity adjustment for the impervious area;
- The Manning n of the pipe system;
- The GUT factor for the gutters, which effectively adjusts the Manning n of the gutters; and
- The initial loss applied to the directly connected impervious area.

Any pervious area contribution was ignored as the previous manual calibration showed that there was no contribution during the storms studied.

PEST allows for the application of rules associating parameters. For the Paddocks catchment the Manning's n of the pipe system and the gutter factor GUT were linked such that they were preferentially adjusted to replicate a constant change in the n value for both the pipe system and the gutter.

Table 6-9 summarises the results from the PEST optimisation, listed in order of increasing recorded peak flow.

Table 6-9 Paddocks Catchment Results of PEST Calibration of ILSAX

Date	Recorded Flow (m^3/sec)	n	GUT	IL (mm)	% paved adjustment
21/11/92	0.771	0.014	6.65	0.0	-13.4
27/02/93	0.866	0.014	7.29	0.4	3.1
11/06/93	0.943	0.013	7.63	1.0	9.3
8/10/92	0.964	0.016	6.18	0.0	10.3
17/10/93	1.048	0.007	13.75	1.9	20.4
18/10/93	1.054	0.008	10.00	1.6	3.2
3/06/93	1.144	0.012	7.76	1.0	-11.5
8/10/92	1.286	0.016	5.64	1.2	12.8
21/05/93	1.377	0.013	7.79	0.8	-9.9
30/08/93	1.391	0.006	10.00	1.6	1.0
3/10/92	1.407	0.011	9.21	2.3	40.2
18/12/92	1.452	0.010	9.83	3.0	49.8
13/12/93	1.669	0.007	13.30	2.4	3.5
14/12/93	1.796	0.014	17.74	1.5	0.1
17/11/92	2.238	0.010	9.81	2.3	10.2
19/12/92	2.464	0.010	10.00	4.0	17.1
mean		0.011	9.51	1.4	8.6

Comments can be made as follows on the PEST optimisation.

- There is no apparent pattern for any of the parameters with increasing recorded peak flow;
- The mean Manning n of the pipe system is 0.011, close to the normally used 0.012;
- The mean GUT factor of 9.51 is close to the default value of 7.6; and
- The % paved adjustment varies widely from storm to storm, with a mean of +8.6%

The ILSAX model can then be rerun with the mean values to determine the overall level of fit that could be achieved by the calibrated model. The results are given in Table 6-10. Appendix 3 contains plots of the measured and predicted hydrographs using ILSAX and the PEST optimisation. One typical storm (30/08/93) is shown on Figure 6-11.

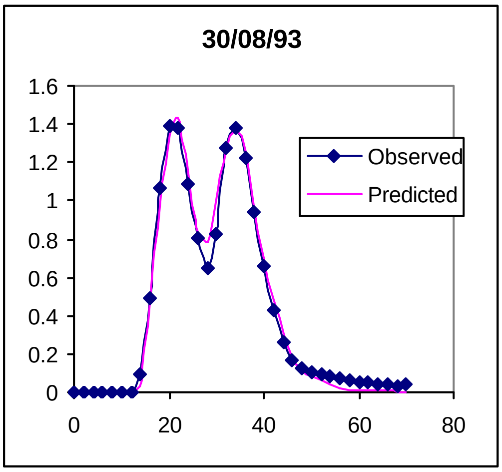


Figure 6-11 Paddocks Catchment ILSAX Fitted by PEST on Storm 30/08/93

Table 6-10 Paddocks Catchment ILSAX Fits With Mean Parameter Values From PEST

Storm Date	Predicted Runoff (m^3)	Recorded Runoff (m^3)	P/ R	Predicted Peak Flow (m^3/s)	Recorded Peak Flow (m^3/s)	P/R
3/10/92	816	955	0.854	0.949	1.407	0.674
8/10/92	1745	1574	1.109	0.81	0.96	0.844
8/10/92	1973	2275	0.867	1.184	1.286	0.921
17/11/92	2333	2316	1.007	1.884	2.23	0.845
21/11/92	1210	984	1.230	0.738	0.771	0.957
18/12/92	848	1124	0.754	1.052	1.453	0.724
19/12/92	3189	3164	1.008	2.148	2.464	0.872
27/02/92	1534	1395	1.100	0.792	0.86	0.921
21/05/93	1818	1448	1.256	1.456	1.378	1.057
3/06/93	2000	1632	1.225	1.149	1.144	1.004
11/06/93	758	648	1.170	0.912	0.943	0.967
30/08/93	1923	1793	1.073	1.332	1.391	0.958
17/10/93	602	629	0.957	0.738	1.048	0.704
18/10/93	814	802	1.015	0.974	1.054	0.924
13/12/93	1488	1379	1.079	1.513	1.67	0.906
	mean		1.047			0.885
	Standard deviation		0.146			0.111

Although this result is apparently no better than the manual calibration it in fact is more reliable. The calibration is objective, the fit being measured objectively by comparing each recorded and predicted hydrograph ordinate. It follows from this that the overall fit, and not just peak flow and volume should be better than the manual calibration. Figure 6-12 shows the level of fit achieved by this approach.

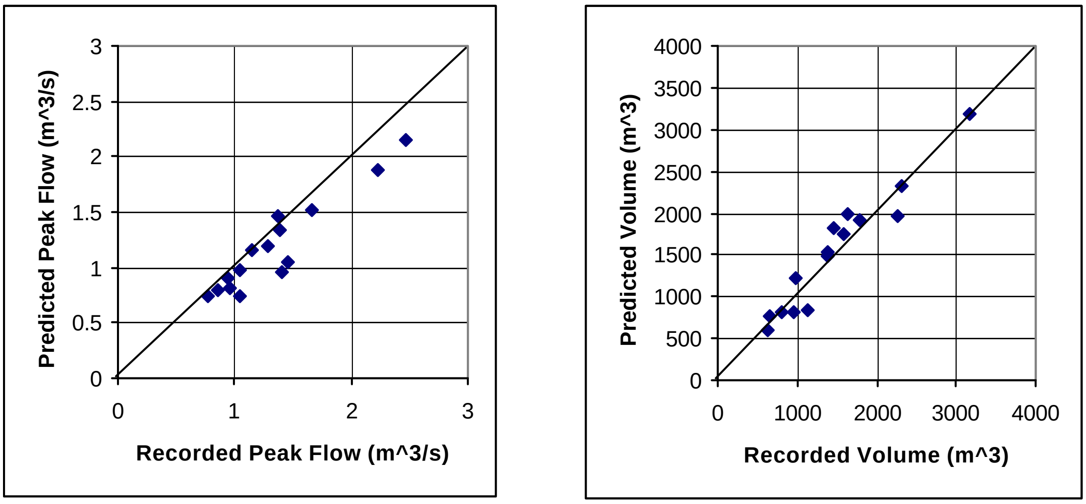


Figure 6-12 Paddocks Catchment ILSAX Results When Fitted by PEST

6.3.5 Paddocks Catchment Summary

When the ILSAX model was fitted to storm events in the Paddocks catchment the model initially overestimated the runoff volume, and underestimated the peak flow. The ILSAX model was then calibrated using the parameter optimisation program PEST, and the resulting peak flow and volume prediction is within 10%. The PEST calibration resulted in the use of a pipe Manning's n of 0.011, and a gutter flow factor GUT of 9.5. The directly connected impervious area was increased by 8.6%.

6.4 Conclusions

The conclusions of the ILSAX modelling on the two urban catchments in South Australia can be summarised as follows:

- The ILSAX model can be successfully applied to urban catchments, and can predict peak flows and runoff volumes given a rainfall input to the model. Once the model is calibrated it can on average predict peak flows and runoff volumes within 10% of recorded;
- In the case of the Paddocks catchment a better result is obtained if a pipe Manning's n of 0.011 and a GUT factor of 9.5 is used. This shows the value of obtaining data for calibration of the model. Unfortunately the results from the two catchments are not sufficient to recommend a value of GUT and Manning's n to be used in South Australia, so the default values should be used;

- The percentage of directly connected impervious area within the catchment can be estimated by a survey of development within the catchment. For the Frederick Street catchment these percentages are

Directly connected impervious	30%
Supplementary paved	17%
Pervious	53%

  And for the Paddocks catchment

Directly connected	26%
Supplementary paved	16%
Pervious	58%

- Most of the storms examined did not show any runoff from the pervious areas, even though the recurrence interval of the rainfall intensities was up to 10 years Average Recurrence Interval.

> *It is recommended that, in the near future, further analysis of data from urban and rural catchments be undertaken to establish with greater confidence the regression equations developed for use with the LRRM in this analysis program.*
>
> *A.P. Aitken (1975)*
>
> *(These regression equations are still in use without review in the RAFTS model)*

7. RAFTS Modelling of South Australian Catchments

7.1 Introduction

It was originally intended that the RAFTS model would be calibrated for South Australian catchments, to determine whether the default values of B derived by Aitken (1975) are appropriate, or whether a new relationship should be derived. However as shown in chapters 4 and 5, RAFTS should not be used with a regional relationship in any other form but with the same number of nodes as the RAFTS model for which the relationship has been derived.

The RAFTS model was applied to test the derivation of RAFTS storage parameter B based on the relationships derived in Chapter 4 between RAFTS and RORB for rural catchments and ILSAX for urban catchments. This was done for a single node model in both cases.

A very simple RAFTS model of a mixed urban and rural catchment was created to show that even with this level of detail an appropriate model could still predict catchment outflow. It is not appropriate to have more nodes than are necessary to define rainfall input and catchment type (urban or rural).

7.2 Rural Catchments - Single Node Model

The relationship between the RORB k_c and the RAFTS B parameter derived in Chapter 4 was tested by undertaking fit runs using the RAFTS model on catchments that had already been fitted using the RORB model, and setting the B parameter as $k_c/5$ and n = -0.200, to create the same non-linearity. A single node RAFTS model was used, to avoid the problems associated with the effect of the number of sub-catchments in the RAFTS model. It should be noted however that a single node RAFTS model has 10 sub-areas in series, all of the same area.

The selected catchments and storm events are as follows:

Table 7-1 Catchments and Events for Comparison of RORB and RAFTS

Station	Catchment Area	RORB sub-areas	Station Number	Event Date
Inverbrackie Creek	8.4km^2	7	AW503508	23/6/87 15/7/87
Aldgate Creek	7.9km^2	16	AW503509	15/7/73 30/7/75 17/6/77
Kanyaka Creek	180km^2	10	AW509503	14/3/89

The runs, using the simple relationship between the storage parameters indicated that there were only very minor differences in the fit achieved by the two models with the exception of the 23/6/87 storm on the Inverbrackie Creek catchment.

The level of fit achieved by the two models was tested by the use of the mean hydrograph ordinate error, defined as:

$$Mean\ error = \sqrt{\frac{\sum_{t=1}^{t=n} \left(Q_o(t) - Q_c(t)\right)^2}{n}}$$

Equation 7.1

Where $Q_o(t)$ is the observed peak flow at time t

$Q_c(t)$ is the calculated peak flow at time t

n is the number of hydrograph ordinates

Table 7-2 indicates that the RAFTS fit is not in general as good as the RORB fit. This would be expected given the total lack of data on the physical layout of the catchment in the RAFTS model. However there are not substantial differences between the two models, and in fact in some cases the RAFTS model is better at predicting the peak flow.

Table 7-2 Comparison of RAFTS and RORB on Rural Catchments

Catchment	Date	Observed peak flow (m^3/s)	RORB peak (m^3/s)	RAFTS peak (m^3/s)	Mean Error (m^3/s) (RORB)	Mean Error (m^3/s) (RAFTS)
Inverbrackie	23/06/87	5.67	5.70	5.08	0.31	0.40
	15/07/87	8.64	9.02	8.37	0.57	0.59
Aldgate	15/07/73	5.34	5.50	5.38	0.67	0.74
	30/07/75	4.55	4.57	4.65	0.65	0.57
	17/06/77	7.20	4.82	4.77	0.73	0.83
Kanyaka	13/03/89	129	113	115	11.5	14.8

The use of the more complicated RORB model, with the need to manually sub-divide the catchment should be questioned given this finding. This is particularly the case in ungauged catchments, where the uncertainty in the storage parameter selection is much larger than the potential errors due to the model selected.

As the storage parameters of the RORB model have been the subject of investigations over a large range of Australian catchments, it was considered not warranted to further pursue the calibration of the RAFTS model for rural areas, but instead use storage parameters based on the generalised RORB parameters for ungauged catchments where necessary.

7.3 Glenelg Catchment

A single node RAFTS model was applied to the Glenelg catchment, initially with the same 1992 and 1993 storms as were tested with the ILSAX model. The catchment response was assumed to be linear.

The method used in the calibration of the model is as follows:

- The storms with flow only from the directly connected impervious area were modelled first. The initial loss was set to model the start of the rise of the recorded hydrograph;
- The directly connected impervious area was adjusted to match the volume of the recorded hydrograph. This is equivalent to using the sensitivity parameter on the impervious area in ILSAX;
- The parameter B was adjusted to match the shape of the hydrograph;
- The unconnected area (supplementary paved plus pervious area) was set to give the correct total catchment area; and
- The storms with unconnected area runoff were then modelled, using the best fit value of B and contributing area from the above storms. The initial loss on the unconnected area was adjusted to start the unconnected area contribution when the impervious area contribution was insufficient to match the recorded hydrograph. The continuing loss was set such that the best fit was obtained for the hydrograph.

7.3.1 Frederick Street

The Frederick Street catchment was modelled first, as there was a continuous set of flow data from the System Q instrument, for 1992 and 1993. Table 7-3 shows the fitted values. The rainfall used in all cases was the Thiessen weighted mean rainfall of the two appropriate stations. It is of note that the final calibrated directly connected impervious area in the ILSAX model was 13.2ha.

As a measure of the level of fit achieved by the model, an objective function was used. The objective function chosen was as per Dyer (1994), as follows:

$$OF=\frac{\sum_{t=1}^{t_{end}}\left|\frac{\{Q_o(t)-Q_c(t)\}}{Q_{op}}\right|^2}{\sum_{t=1}^{t_{end}}\frac{Q_o(t)}{Q_{op}}}$$ Equation 7.4

where Q_o is the observed flow (m^3/s)

Q_c is the calculated flow (m^3/s)

Q_{op} is the observed peak flow (m^3/s)

Note that a lower objective function implies a better fit. Table 7-3 gives the summary of the RAFTS fit runs. Appendix 4 contains plots of the measured and predicted hydrographs, with one typical event reproduced as Figure 7-1.

Table 7-3 Summary of RAFTS Fits for the Frederick St Catchment.

DATE	Bi (hrs)	Bp (hrs)	Directly Connected Impervious Area IL (mm)	Directly Connected Impervious Area (ha)	OBJECTIVE FUNCTION
3-4/07/92	0.032		1.0	13.4	0.062
11/07/92	0.047		1.5	12.5	0.022
19/07/92	0.038		0.8	12.9	0.017
7/08/92	0.047		0.4	13.2	0.040
30/08/92	0.040	0.050	2.0	13.2	0.013
31/08/92	0.028		1.0	14.7	0.119
18/12/92	0.040	0.050	2.0	13.2	0.010
21/05/93	0.047		0.6	14.2	0.029
29/08/93	0.047		3.0	14.7	0.048
18/09/93	0.040		3.0	17.8	0.020
28/09/93	0.047		1.5	15.4	0.058
16/10/93	0.047		1.5	14.5	0.011

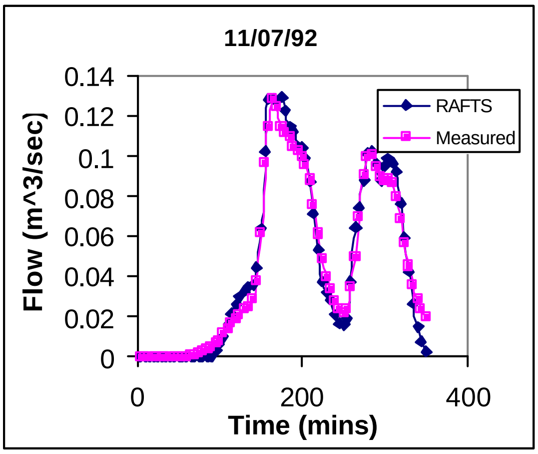


Figure 7-1 Frederick Street, Glenelg RAFTS fit for 3/07/92

It was found necessary in some of the storms to introduce into the model a time translation to match the recorded and predicted hydrograph. This translation time was not found to be consistent, and in one case needed to be negative (3-4/07/92). The reason for this translation is not known, but it is suspected that it is because of differences between actual rainfall distribution on the catchment, and the assumed distribution in the model (evenly across the catchment). The rainfall distribution on the catchment may also affect the calibrated lag, with the lag being less if rainfall was occurring close to the gauging station.

To determine a design value of Bi the fitted Bi values can be weighted by 1/OF, where OF is the objective function. The mean of the weighted Bi values is then 0.042. The value of Bp for the two storms having pervious area runoff was 0.050.

7.3.2 Maxwell Terrace and Torrens Square

The period of record for these two catchments was only available for some storms in 1992, and from the Detectronics instrument, which shows the inconsistencies. The apparent volumetric runoff coefficients were less than those derived at the Frederick Street gauging station, (at about 15%, compared with 27% at Frederick Street) and after discussion with the hydrographers providing the data it was identified that the instrument was not recording properly. The instrument was situated in the base of the pipe, and sediment slugs going past caused the instrument to read zero velocity, and thus flow for some periods.

However, it is considered that the data is still useful in that the B value is not affected by the absolute value of the flow recorded, provided that the error is consistent. This follows from the assumption that the catchment behaves linearly.

The fitting of these storms thus involved adjustment of the impervious area, to account for the gauging error as well as adjusting the impervious area initial loss and Bi.

For the initial fits rainfall data from Frederick Street was used, as this station was reasonably central in the catchments.

Table 7-4 summarises the fit runs carried out on the two catchments.
It can be seen from the above that the general standard of fit is not as good as that attained at Frederick Street, because of the Detectronics instrument error and the use of a single rainfall input to model the rainfall on the entire catchment.

In view of the above values, and weighting the Bi values with the objective function, a value of Bi of 0.048 for Maxwell Terrace and 0.060 for Torrens Square can be adopted. These values are consistent with the results of Frederick Street.

Table 7-4 RAFTS fits for Maxwell Terrace and Torrens Square

Catchment	Date	Directly Connected Impervious Area IL (mm)	Total Area (ha)	Directly Connected Impervious Area (ha)	Bi (hrs)	Objective Function
MAXWELL TCE	30/08/93	2.0	106	20	0.051	0.118
	19/09/93	1.0	106	24	0.045	0.007
	30/09/93	0.6	106	20	0.060	0.025
	16/10/93	2.0	106	18	0.045	0.020
TORRENS SQ	28/09/93	0.0	183	30	0.060	0.122
	30/09/93	0.0	183	30	0.060	0.017

7.4 Paddocks Catchment

The RAFTS model was applied to the Paddocks catchment for the storms modelled by ILSAX. The ILSAX modelling showed that none of the storms had a contribution from the pervious area, so it was assumed that the RAFTS model will show no contribution from the pervious area. Calibration was then simply carried out by firstly selecting a directly connected paved area that matched well with the observed runoff volumes. The initial loss was adjusted to match the start of rise of the hydrograph. The directly connected impervious area was adjusted to match the runoff volume. It was found in most cases that an area of 18ha gave a reasonable match. This compares with the expected directly connected impervious area of 19.8ha, obtained by survey of the area (Salisbury City Council, 1994).

It was found however that there was an apparent time shift of 3 to 5 minutes between the recorded and predicted hydrographs. The value of the impervious storage parameter Bi was adjusted to match the shape of the hydrograph. The value of Bi was assessed to the nearest 0.005 hours, as this was the minimum increment at which a noticeable change in shape of the predicted hydrograph occurred. It soon became apparent that a single value of Bi was applicable to most storms examined.

Table 7-5 is a summary of the fit runs carried out. Appendix 5 contains plots of the measured and predicted hydrographs, one of which is reproduced as Figure 7-2.

Table 7-5 Paddocks Catchment RAFTS Fits

Storm Date	Fitted Bi (hrs)	Directly Connected Impervious Area IL (mm)	Directly Connected Impervious Area (Ha)	Actual Peak Flow (m^3/s)	Predicted Peak Flow (m^3/s)	Objective Function
21/05/92	0.010	1.0	18	1.378	1.726	0.052
03/10/92	0.015	2.0	20	1.407	1.501	0.005
08/10/92	0.015	0	18	1.286	1.230	0.026
08/10/92 (2)	0.015	0	18	0.964	0.782	0.016
17/11/92	0.020	0	18	2.239	2.448	0.009
20/11/92	0.015	0	18	0.772	0.760	0.038
18/12/92	0.015	0	18	0.786	0.488	0.081
18/12/92 (2)	0.015	0	18	1.453	1.829	0.036
24/01/93	0.015	1.0	18	0.843	0.512	0.108
27/02/93	0.015	0	18	0.866	0.802	0.015
30/05/93	0.015	0	18	0.831	0.796	0.034
03/06/93	0.015	0	18	1.144	1.287	0.017
11/06/93	0.015	1.0	18	0.943	0.928	0.041
30/08/93	0.015	1.0	18	1.391	1.596	0.063
17/10/93	0.015	1.0	18	1.048	1.012	0.079
18/10/93	0.015	1.0	19	1.054	1.060	0.012

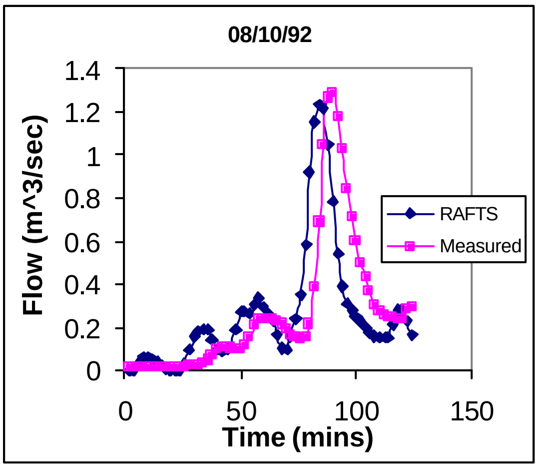


Figure 7-2 Paddocks Catchment RAFTS fit 08/10/92

Some of the recorded hydrographs had a very long tailing limb that could not be matched by the model. Reasons for this part of the hydrograph may be due to the presence of sub-soil drainage, infiltration into the pipe system, back of block drainage, or debris on the gauging weir.

The most common value of B was 0.015, and the weighted mean value was 0.016.

7.5 Happy Valley Catchments

As an example of the calibration of a simple model on a mixed urban and rural catchment the RAFTS model has been applied on the catchment of Sauerbier Creek at Happy Valley, south of Adelaide.

There are two catchments associated with the Happy Valley project, initiated by the University of Adelaide and supported by Happy Valley Council and the Stormwater Drainage Subsidy Scheme (Daniell & McCarty 1994).

The two catchments are adjacent to each other. The Sauerbier Creek catchment has a substantial proportion of area (141 ha out of a total of 254 ha.) rural, and with the natural creek system still in place. Modelling was carried out on Sauerbier Creek. The catchment is shown in Figure 7-3. Data is obtained from three rainfall stations and two gauging stations located just upstream of road culverts on the Hub Drive (Sauerbier Creek) and Happy Valley Drive (Minkara Creek).

The gauging stations incorporate weirs associated with small permanent ponds.

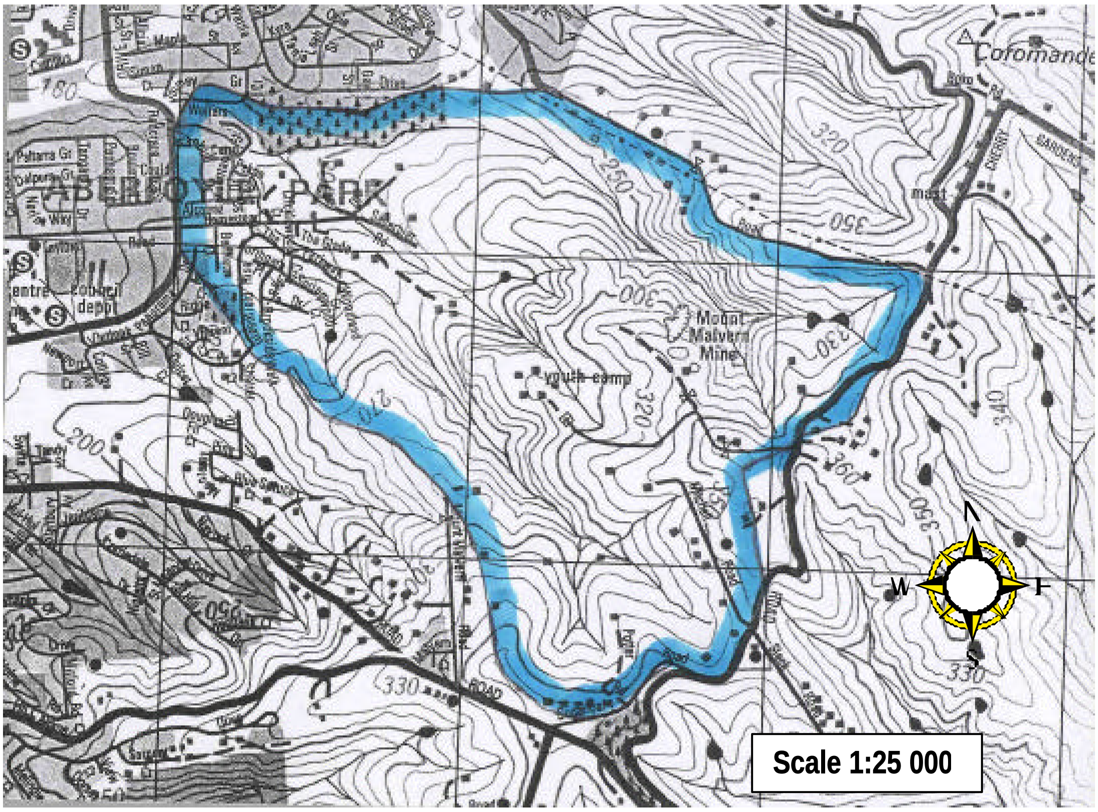


Figure 7-3 Sauerbier Creek Catchment

Figure 7-4 View of the Sauerbier Creek Catchment

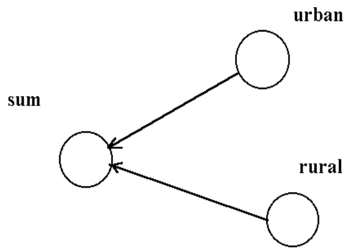


Figure 7-5 Sauerbier Creek Model Layout

The RAFTS model in its simplest form was set up as a three node model where two nodes represented the rural and urban portions of the catchment and the third node summed the two contributions (Figure 7-5)

As the catchment retained natural creek channels it was necessary to fit the storage exponent n to model non-linear behaviour.

It was decided after initial inspection and trial fit runs that the fitting of the RAFTS model to the Sauerbier Creek catchment would proceed from the start to the end of the runoff hydrograph, initially using the storm producing the largest runoff. The approach to the calibration was as follows:

- It was assumed that the initial runoff would occur from the impervious part of the urban catchment directly connected to the pipe or main channel system. The continuing loss on the impervious area was considered to be zero. The initial loss was determined from the start of the rise of the hydrograph when runoff was occurring from only directly connected impervious areas. The impervious catchment area, Bi value and exponent n were then fitted to match the initial period of the storm. The contributing area was adjusted such that the predicted flow matched the recorded hydrograph, with the adopted losses;
- It was found that a lag of 12 minutes for the impervious urban area contribution was required to produce good fits;
- As time progressed the modelled runoff was insufficient to match the recorded hydrograph. This was evidence that runoff was occurring from another part of the catchment, or by another process. Runoff was assumed to come next from pervious areas within the urban portion of the catchment. This area is the unconnected area, being the total of the supplementary paved area

and the pervious area of the ILSAX model. The area was determined as the balance of the total urban catchment area;

- The values of initial loss were determined from the time at which runoff contribution from the directly connected impervious area was insufficient to match the recorded hydrograph. The values of Bp, exponent n and the continuing loss were adjusted to match the runoff from the next portion of the storm.
- When the tail of the storm hydrograph was not correctly modelled, it was determined that a contribution occurred from the rural part of the catchment. The area of this rural part was determined and the initial loss set to commence contribution at the appropriate time. Figure 7-6, for the storm of 13/12/93 shows the point at which rural runoff is assumed to commence, where the predicted and gauged hydrographs no longer match. The values of B, n and the continuing loss were adjusted to fit the remaining part of the hydrograph.

Table 7-6 summarises the storms fitted on the Sauerbier Creek.

Table 7-6 Saubier Creek Storms Fitted

STORM	DATE	START	DURATION (mins)	RAIN (mm)
1	21/05/93	14:00	120	10.5
2	07/07/93	03:00	1200	48.0
3	30/08/93	16:30	210	11.0
4	19/09/93	11:00	540	7.8
5	17/10/93	08:00	780	8.6
6	13/12/93	22:00	660	61.4

Fitting commenced with the storm of 13/12/93 that exhibited flow from all three areas. The fit obtained is shown on Figure 7-6. The directly connected impervious area was determined from the above approach to be 22 ha., which was consistent for all storms fitted. Based on this method and because the total developed urban area is 113 ha. the three contributing areas were determined as follows

Directly Connected Urban;	22ha.	(impervious)
Unconnected Urban	91ha.	
Rural	141ha.	

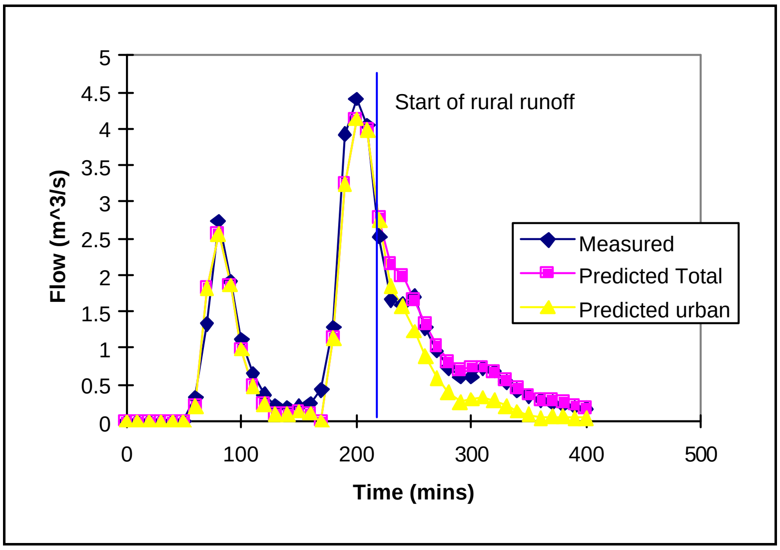


Figure 7-6 RAFTS Model fit for Sauerbier Creek 13/12/93

Given the type and extent of development within the urban area the directly connected percentage of 19.4% is considered reasonable. Table 7-7 summarises the fitted parameters for the storms examined. Appendix 6 contains plots of the measured and predicted hydrographs for all storms examined.

Table 7-7 Saubier Creek Fitted Parameters

Storm	Contributing Area	B	IL (mm)	CL (mm)	Runoff (mm)
21/05/93	Unconnected Urban Impervious Rural	- 0.06 -	- 4 -	- 0 -	0 6.5 0
07/07/93	Unconnected Urban Impervious Rural	0.08 0.06 0.30	20 3 46	5 0 0	5.4 45.0 2.0
30/08/93	Unconnected Urban Impervious Rural	- 0.055 -	- 0.8 -	- 0 -	0 10.2 0
19/09/93	Unconnected Urban Impervious Rural	0.035 0.035 0.3	1 1.5 4	15 0 5	1.0 6.3 1.4
17/10/93	Unconnected Urban Impervious Rural	0.05 0.05 0.3	5 2.5 6	13 0 8	0.3 6.3 0.3
13/12/93	Unconnected Urban Impervious Rural	0.08 0.05 0.2	25 0 50	25 0 10	6.9 59.5 3.0

In all cases a value for the exponent n of -0.2 was found to give the closest match to the shape of the recorded hydrograph. This non-linearity could be expected given that most of the trunk drainage follows the original creek channels. It is interesting to note the relative contributions of the three areas to the outflow hydrograph, with the directly connected impervious area producing by far the most runoff.

The fits obtained could be considered to be good especially given the simplicity of the model.

The fits were relatively insensitive to the parameters used for the unconnected or rural areas because of the lower relative contribution from these two areas. This is evident in the effective rainfall for the contributing areas.

In summary the fitting of the model on the catchment has shown that it is possible to model complex catchments with a very simple model, provided that contributing processes are identified and allowed in the model.

7.6 Comparison of Urban Bi Values With Theoretical Values

The calibrated values of the impervious area lag parameter Bi from the Glenelg and the Paddocks catchments can be compared with the theoretical values derived in Chapter 4, based on the ILSAX model.

Table 7-8 Comparison of Calibrated and Theoretical B Values

Catchment	Calibrated Bi (hrs)	Pipe Flow Time (mins)	Mean Gutter Flow Time (mins)	T_e (mins)	Total (mins)	Theoretical Bi (hrs)
Glenelg - Frederick St.	0.042	14.32	13.37	5.00	32.69	0.054
Glenelg - Maxwell Tce	0.048	21.11	13.37	5.00	39.48	0.066
Glenelg - Torrens Sq.	0.060	31.25	13.11	5.00	49.36	0.082
Paddocks	0.016	9.23	3.66	5.00	17.89	0.030

It can be seen that in all cases the values from the fitted RAFTS model are less than the theoretical value. In the case of the Paddocks catchment a lower Manning's n and a GUT factor of 9.5 had to be used with the ILSAX model, both of which reduce the modelled catchment lag. This may go some way to explaining the discrepancy, but it is most likely that the problem lies in the assumption that the lag of the catchments is 50% of the total storage delay time within the catchment. This will only be true if contributing area is evenly distributed in time through the catchment. This is not always the case. Also the catchment lag is made up of the delay time for the entry to the gutter system, and the time within the gutter and pipe system. Even near the catchment outlet there will be a storage delay time because of the time of entry to the gutter. Indeed, this is one explanation of the 2 to 5 minute lag that had to be inserted at the catchment outlets with the RAFTS model to match the recorded hydrographs. This extra lag should be added to the calibrated Bi values to give the true catchment lag, which then would better match the theoretical values.

7.7 Conclusions

A comparison has been made between a single node RAFTS model and a RORB model on the same catchment, and both models give a very similar result. The added complexity of the RORB model with catchment sub-division may not be warranted, particularly given the uncertainty in the selection of the storage and loss parameters to be applied to the model. The RORB model, with catchment sub-division is warranted only to model variability of rainfall or storage across the catchment.

Similarly the application of a simple RAFTS model to a complex partly urbanised catchment shows that a simple model can give good results.

The calibrated Bi values for the RAFTS model were less than the theoretical values derived in Chapter 4. This is most probably due to the distribution of storage, and thus storage lag within the catchment, and in particular the lag due to the time of entry to the gutter. This lag will be apparent even for contributing area near the catchment outlet. The time shift of 2 to 5 minutes that was required to match actual hydrographs can also be explained by this storage delay time to the gutter.

A runoff routing model designed for use on urban catchments should be able to separate the storage lag due to entry to the gutter from the storage lag due to flow within the gutter. This is one reason for the development of a new model, to be undertaken in the next chapter.

The comparison of the theoretical Bi value with the calibrated value on four urban catchments did not give good results for the Glenelg and the Paddocks catchments. This is most probably due to the distribution in time of area within the catchments, and in particular the time of entry to the gutter and pipe system, even near the catchment outlet. This indicates the need for a runoff routing model that allows separately for this entry time to the gutter system, rather than lumping both entry to the gutter and transport along the gutter into one series of storages. A new model is developed in the next chapter that overcomes this limitation.

For a good mathematical model it is not good enough to work well. It must work for the right reasons. It must reflect, even if only in simplified form, the essential features of the physical prototype.

Vit Klemeš (1986)

8. The RRR Model

8.1 Introduction

The findings so far, particularly with regard to the problems associated with the number of sub-catchments in runoff routing models leads to the conclusion that there must be a better model structure available that does not suffer from the limitations of the existing models.

This chapter discusses these limitations, and develops the new model structure, taking into account the statement of Klemeš (1986), that the model should reflect, even if only in a simplified form, the essential features of the physical prototype.

8.2 The Limitations of RORB, WBNM and RAFTS

8.2.1 RORB

There are two major limitations with the RORB model. Firstly, the model result is dependent on the number of sub-catchments. Both Boyd and Dyer have investigated the effect of catchment sub-division on runoff routing models (Boyd 1985, Dyer 1994). The main conclusion is that below a minimum number of sub-catchments, depending on catchment size, hydrograph properties are not stable. As the number of sub-catchments becomes very large, the model response approaches the case of pure translation, where the model instantaneous unit hydrograph approaches the catchment time-area diagram.

Secondly, the model is not internally consistent, that is the storage discharge relationship for each storage in the model is not independent of the model structure (Yu and Ford 1989). The characteristic arises as a result of the use of the total catchment lag parameter k_c, which varies with catchment area.

The model is also inflexible in that the storage parameter remains constant across the catchment. The model cannot be fitted to more than one gauging station for any storm, as a single catchment wide value of k_c must be used.

8.2.2 WBNM

The WBNM model is internally consistent, that is the model structure is such that the storage discharge relationship for each storage is independent of the model structure or number of sub-catchments. However, as for the RORB model there is a required minimum number of sub-catchments, dependent on the catchment area. There is also an empirical factor (usually set at 0.6) used to model the transfer of flows via the main stream from upstream sub-catchments. In chapter 5 it was shown that to retain constant lag in a catchment model, the factor must change with the number of sub-catchments. Since it remains constant, it is possibly the reason that the predicted flows vary with the number of sub-catchments.

8.2.3 RAFTS

It has been shown that the RAFTS model is not internally consistent, and that a storage multiplication factor BX must be applied depending on the total number of nodes within the model. Following from this any regional derivations of the storage parameter B are of use only with a model having the same number of nodes.

The use of a default storage lag exponent of -0.285 for urban areas is not in accordance with the finding that response of these catchments is generally linear (Bufill and Boyd 1992).

The use of split catchment representation with a non-linear storage lag exponent is also not theoretically correct, as the storage lag must depend on the total catchment flow, and not the contribution to flow from either component. Only with a linear storage, and constant storage lag is the split catchment model appropriate. If it is assumed that catchment lag is $K = Bq^n$, where q is the catchment outflow, the lag of a split sub-catchment model will be:

$$K_{split} = K\left(\frac{q_{split}}{q}\right)^n \qquad \text{Equation 8.1}$$

where K is the true catchment lag;
K_{split} is the apparent lag of the split sub-catchment;
q is the catchment outflow; and
q_{split} is the flow from one of the split sub-catchments.
n is the exponent in the storage equation $K = Bq^n$

If n is negative (as it is in the default equation) K will be overestimated, and thus outflow will be reduced. This may be the reason for the difference in behaviour of the lumped and split sub-catchment models when varying the number of sub-catchments (Hood 1991). Hood found that for the same catchment area with a lumped model and low numbers of sub-catchments the discharge was overestimated, whereas split catchments with low number of sub-catchments underestimated discharge.

The default equation for the RAFTS B value was derived by Aitken (1975), on a small number of catchments and contains a single expression for both urban and rural response, which has been extrapolated to be used on completely impervious areas. The use of such a single expression cannot be supported, given the vastly different responses from impervious areas within an urban area and rural catchments.

8.3 Storage Lag in Runoff Routing Models

The value of the storage parameters for runoff routing models has been the subject of much research.

For the WBNM model, a regional storage parameter c has been determined for a non-linearity exponent m = 0.77. Boyd (1983) and Sobinoff et al (1983) carried out this research. Boyd recommends a value of c = 1.68, based on five catchments in Eastern New South Wales. Sobinoff et al considered that this value may be an overestimate based on the calibration of 21 catchments in eastern New South Wales, where it was found that most values were below 1.68.

For the RORB model Yu (1990) proposed that a storage parameter k^* be used instead of k_c where k^* is k_c/d_{av}. This proposal was based on the dependence of k_c with catchment area and particularly with the average flow distance in the catchment d_{av}. Yu examined the value of k^* for basins in Western Australia, Victoria, New South Wales and Queensland and concluded that there was evidence of dependence on mean annual rainfall for southern Australia. For Western Australia a relationship was given as:

$$k^* = 2.28\left(\frac{RF}{1000}\right)^{1.51}$$
$$\left(r^2 = 0.80\right)$$ Equation 8.2

And for Victoria:

$$k^* = 1.89\left(\frac{RF}{1000}\right)^{1.64}$$
$$\left(r^2 = 0.52\right)$$ Equation 8.3

An imaginary line drawn through Derby, Western Australia and Sydney defined Southern Australia. North of this line no relationship was found. The fact that the area having dominant winter rainfall is coincidental with reasonably good relationships between k^* and mean annual rainfall may be far from fortuitous. It was speculated that in northern Australia where, in the summer, heavy rainfall associated with monsoon southern excursions, tropical cyclones, and local heavy thunderstorm events form the channel network within catchments, that mean annual rainfall may not represent or be related to the condition in which the channel network was formed.

Dyer et al (1994) and Dyer (1994) contain the most recent assessment of the k_c/d_{av} parameter, designated by Dyer as c (being equivalent to k^*). The prediction equations in this work were based on 72 catchments across Australia, however there were no catchments represented from northern coastal or central Australia.

In order to standardise the determination of k_c and m values for the catchments Dyer recalibrated all the catchments, with a standard automated calibration procedure based on minimising an objective function related to hydrograph ordinate error and the observed peak flow. A total of 49 parameters were determined for each catchment and model, including the morphology of the drainage network, meteorological characteristics and characteristics related to the RORB model. It is of interest that catchment area is not included, although d_{av} is. Catchment area was included in 18 of the 22 regionalisation studies for the RORB k_c parameter quoted by Dyer and as such it would be expected that c may be related to catchment area also.

It was decided to standardise the value of m at 0.8 on the basis of earlier work by Dyer (Dyer et al, 1993).

Dyer’s analysis proceeded as follows:

- cluster analysis was carried out to give the initial groups;
- these groups were then adjusted using Andrews Fourier Plots;
- regression analysis was performed on each of the adjusted groups to determine regional prediction equations; and
- the accuracy of the sets of regional prediction equations was compared to determine which parameter set is the most suitable for general release.

Eight variables were selected for use with the Andrews curves, these being longitude, d_{av}, the number of conceptual storages type 2 (storages where a rainfall excess is added to the running hydrograph) in the RORB model, maximum catchment elevation, the ratio of annual rainfall to evaporation, the percent forest cover, the annual number of raindays and the ratio of the modelled RORB length to catchment area.

A total of seven catchment groups were identified using this approach.

The prediction equations for c for the seven groups contain up to five variables, mostly not related to the variables selected for the grouping. It would be expected that the significant variables in the regression equations would be similar to those used to select the groups. In one group three variables were related to the RORB model structure, and only one (raindays per year) to the physical catchment.

The equations are as follows:

group 1:

$$c = 0.405\, pem^{-1.82}\, lrat^{0.18}$$ Equation 8.4

group 2:

$$c = 139\, min\, el^{0.27}\, rr^{-0.55}\, nn^{-0.38}\, sa^{-1.46}$$ Equation 8.5

group 3:

$$c = 0.445\, d_{av}^{-0.73}\, rla^{-0.90}\, strm^{-2.22}\, nn^{-0.70}\, medrn^{0.88}$$ Equation 8.6

group 4:

$$c = 1.04\, rlm^{0.77}\, ln\, n^{-0.36}$$ Equation 8.7

group 5:

$$c = 0.232\, rlen^{-0.34}\, rrd^{1.13}\, sa^{1.13}\, cd2^{-1.45}$$ Equation 8.8

group 6:

$$c = 20.6 circ^{0.79}\, for^{1.38}\, rlt^{-0.28}\, lmns^{0.32}$$ Equation 8.9

group 7:

$$c = 11.1 circ^{1.08}\, pe^{0.74}\, sa^{0.79}$$ Equation 8.10

Where: pem is the ratio of median annual rainfall to evaporation
lrat is the ratio of the largest RORB sub-catchment to the total area

minel	is the elevation of the catchment outlet
rr	is the relief ratio (maximum elevation - minimum elevation over main stream length)
nn	number of streams of order one less than the outlet
sa	is the number of sub-catchments in the RORB model
d_{av}	is the average flow distance on the catchment
rla	is the RORB length over area
strm	is the stream order at the outlet
medrn	is the median annual rainfall
rlm	is the RORB length over the mainstream length
lnn	is the length of streams having an order of one less than the outlet
rlen	is the length of the reaches in the RORB model
rrd	is the number of raindays per year
cd2	is the number of type 2 conceptual storages in the model
circ	is the catchment area / perimeter^2
for	is the fraction of forest
rlt	is the RORB stream length / total stream length
lmns	is lnn / the mainstream length
pe	is the ratio of mean annual rainfall to evaporation

There are no common variables occurring throughout, neither is there any relationship with the variables used in the Andrews Fourier Plots. The sorting of catchments into groups has not directly included annual rainfall (although the ratio of median annual rainfall to class A pan evaporation and the number of raindays is used). Thus the finding of Yu (1990) that annual rainfall is a variable has therefore neither been supported nor discounted by Dyer.

It is also difficult to see how some of the variables in the regressions can have any direct effect on the value of storage lag.

It is of interest that apart from a strong relationship between storage lag and catchment length (as represented by d_{av}) no consistent relationship between physical catchment characteristics and storage lag was found.

One reason for this may be in the basic assumption of runoff routing models, that only one process is being modelled, that of direct surface runoff. This being the case the total storage in the catchment can therefore be represented by the storage available within the channel system of the catchment.

If it can be shown that channel storage alone is not a good representation of the total catchment storage then this can go some way to explaining the problems associated with the derivation of regional storage parameters. If catchment lag is related to the processes occurring on the catchment there will not only be a large variation in catchment lag for one catchment, it will also be difficult to derive good regional relationships if physical parameters do not indicate the processes that are occurring.

8.4 The Evidence for Runoff Process Related Storage Lag

8.4.1 Investigations into Channel Storage as a Representation of Catchment Storage

Research into the physical basis of the storage parameters in runoff routing models has previously been performed. Examples are Laurenson and Mein (1990) and Zhang and Cordery (1999).

If channel storage is the dominant storage component then the power function as used in RORB, RAFTS and WBNM can be related to storage in open channels. The RORB model uses a storage equation of the form $S = 3600\ k_c\, k_r\, Q^m$ for each modelled channel section, k_c being a catchment wide parameter and k_r being a parameter that could be related to the individual reach $L/s^{0.5}$, where s is the channel slope. This was based on the assumption that uniform flow is occurring and that channel storage is related to channel slope, as it would be theoretically if the mean flow velocity in the reach varied with slope in accordance with Manning's formula. Laurenson and Mein (1990) give theoretical values of the exponent m for uniform flow in open channels (reproduced in Table 8-1).

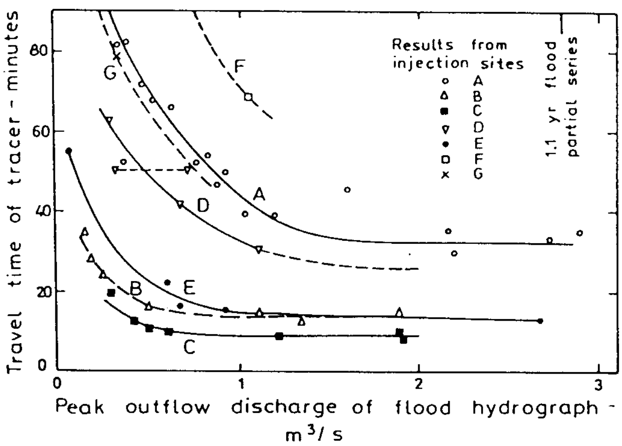
Table 8-1 Theoretical m Values For Regular Cross Sections (After Laurenson and Mein, 1990).

Section Shape	m
Triangular	0.75
Trapezoidal	0.74
Parabolic	0.69
Wide Rectangular	0.60

If the channel properties and thus channel storage were the determining factor then values of m would be expected to be in the above range. Instead the value of m is generally of the order of 0.8, and is reasonably constant (Laurenson and Mein, 1990).

More importantly catchment lag for rural catchments would be related to a slope term for the catchment. However there are few regional assessments for catchment lag that include slope as a significant parameter.

Travel-time discharge studies carried out by Pilgrim (1976, 1977, 1980 and 1982) found that lag times in a catchment were approximately constant above a certain value of flow indicating linear storage (lag time not dependent on flow). Pilgrim explained this as increasing channel roughness at high flows (due to the hydraulic resistance of vegetation and irregularities) that overcame the more efficient hydraulic section. The flows where linearity was found varied with catchment area, but were less than the 1.1 year ARI. This flow at which constant lag was observed could be considered to be less than "overbank" and thus the change in channel properties with the introduction of floodplains could not be expected to explain the constant lag. Figure 8-1 shows the Research Creek catchment stream velocity and the travel times at different flows.



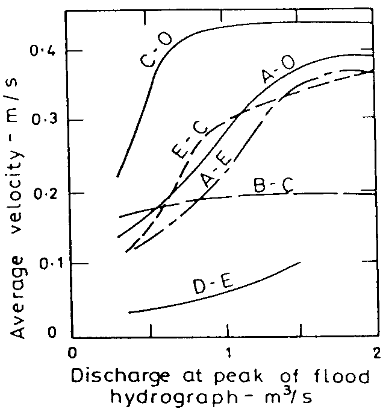


Figure 8-1 Travel Time Results and Catchment for Research Creek (After Pilgrim, 1982)

In order to introduce this constant lag time into the lag equation for runoff routing models Bates and Pilgrim proposed the piecewise linear model (Bates and Pilgrim, 1986). It was proposed that a storage function of the form $S = S_0 + KQ$ be used where S_0 is a threshold storage. Bates and Pilgrim implemented the storage relationship in the WBNM model. However the model does not preserve continuity as each conceptual storage retains a volume of S_0. To overcome this problem when implementing the storage-discharge relationship in WBNM, Bates and Pilgrim set the threshold storage to zero and added a translation element immediately prior to the catchment outlet to allow for the delay of the hydrograph caused by the threshold storage.

The resultant model is close in performance therefore to other linear models.

More recently Dyer gave a very comprehensive review of the physical basis of the storage parameter of runoff routing models whilst considering the parameter values to be used for extreme events (Dyer, 1994). It is assumed that all storage occurs within the channel system. Some of the conclusions of this work were that:

- The storage-discharge relationship applicable to surface runoff flows for an event is quite different to the absolute storage-discharge relationship. The storage-discharge relationship for surface flows (ie. as used in runoff routing models) can be considered to have a constant gradient at flows higher than a given value. There is no evidence that the constant gradient relationship passes through the origin;
- There is no support for the value of the exponent m in the storage equation to increase to unity for extreme events; and
- The storage parameter does vary systematically between events and that this variation can be related to the shape of the hydrograph. No relationship could be found to the magnitude of the event.

Therefore in summary the theoretical values of m are generally lower than found in practice when calibrating runoff routing models. In addition it appears that although channels tend towards linear behaviour (constant lag) at higher flows, the evidence that catchments tend towards linear behaviour with increasing flows is not supported from the evidence forwarded by Dyer.

For these reasons it can be seen that the evidence for channel storage being dominant is not strong. Dyer's conclusions support the notion that **catchment lag is related to the dominant catchment runoff process.**

8.4.2 The Lidsdale Catchments

There is published evidence to support the notion that a catchment does not have a single storage lag. In 1963 the University of New South Wales established a group of experimental catchments within the Lidsdale State Forest 12 km west of Lithgow in New South Wales (Casinader et al, 1989). The group comprised eleven small forested catchments and the original purpose was to study the differences in water yield between catchments planted with a commercial pine forest and catchments remaining under natural eucalypt forest.

To this end some of the catchments were grouped in four pairs, each consisting of one catchment covered in pine forest and another covered in eucalypt forest, selected to be as similar as possible in all other respects.

Casinader et al (1989) reported on the analysis of flood response data from the catchments. The method chosen was to derive response functions (unit hydrographs) for a number of flood events on the catchments. It was found that four categories of flood response could be found for the catchments. The categories named were related to the shape of the unitgraphs derived from the flood records as follows:

- Large non-linear: unitgraphs with unusually large peaks;
- Partial area: unitgraphs having a similar shape to the above, but having lower peaks. These were interpreted as events where only a small sub-section of the catchment was contributing to runoff;
- Small non-linear: These were unitgraphs with very small peaks, and came from very small floods; and
- Normal: The rest of the unitgraphs could be identified as a consistent set, which was termed normal.

The differences in flood response were deemed to be related to the three distinct physical processes by which runoff is generated (Hortonian runoff, saturated overland flow and throughflow, a type of sub-surface flow).

An attempt was then made to relate the type of hydrograph displayed by the catchment to parameters for the event, including runoff depth, total rainfall, percentage runoff, average rainfall intensity, length of runoff event and continuing loss.

It was found that distinct differences occurred in rainfall characteristics between categories. Partial area events occur in storms producing a large amount of rainfall at high intensities, characteristic of convective activity such as thunderstorms. Large non-linear events also originate from large rainfall bursts, but falling at a lower intensity, representing sustained non-convective rainfall. Small non-linear events are associated with small, low intensity rainfall bursts.

Inspection of individual events indicated that antecedent wetness of the catchment could significantly modify the type of event resulting from a particular rainfall pattern, but nevertheless the above associations did support the event classification system.

It was concluded that each event category represents the action of a different combination of mechanisms of runoff generation, and of a different pattern of runoff source areas. The type of runoff event that results from a given rainfall burst depends on the short-term characteristics of the rainfall and on the antecedent wetness of the catchment. The relative frequency of each category of runoff event is determined to a large extent by the topography, soils and geology.

8.4.3 The Common Unitgraph

Chapman (1993) developed a technique for estimating a common unitgraph and event input hyetographs for a set of surface runoff events, without using rainfall data. However the common unitgraphs obtained in this way from streamflow data typically have an earlier and higher peak and shorter duration than average unitgraphs derived by conventional methods using rainfall and loss data. The calculated input hyetographs from the technique continue after rainfall has ceased, and they have peaks that occur later than the corresponding rainfall peaks. It was found that these problems could be resolved by the insertion of a non-linear storage between the usual rainfall loss algorithm and the common unitgraph. The final prediction of runoff from rainfall from this approach was found to be at least as good as those obtained by conventional methods and extend over a wider range of flows.

It was postulated that a rainfall loss model and non-linear storage model can be used together to model the processes of infiltration and overland flow, while the common unitgraph routes the stream flow to the outlet.

This splitting of "out of channel" processes and channel routing assuming linear storage supports the runoff process dependence of lag. The linear channel routing is supported by the findings of constant flow times for most events found by Pilgrim. The common unitgraph represents a minimum catchment response time, which occurs if the total catchment storage lies within the stream channels. If the response time for the overland flow non-linear storage were also minimised, as occurs in major events with a non-linear storage outside the channels, then the catchment response would appear to become linear at high flows.

8.5 The RRR Model (Single Sub-catchment)

The view that a catchment does not have a single lag therefore leads to the proposal for a better model structure for a runoff routing model that mimics the actual catchment behaviour. This model is based on the concept proposed by Chapman (Chapman, 1993).

As the model represents a single sub-catchment it cannot account for spatial variability within the catchment. It may however be used for calibration and prediction of flows for a catchment where there is limited data available, for instance flow at the outlet, and only one rainfall input. In this case there is no need to account for spatial variability, as there is no information available on which to base changes in rainfall or response across the catchment.

The model needs to have separate channel storage and hillside or process storage. This represents a major change to existing runoff routing models that assume that only surface runoff is being modelled, and that the total storage within the catchment can be represented by a series of storages along the watercourses. The runoff routing model will then be similar to modern hydrodynamic models, such as KINDOG (Kuczera, 2000).

As discussed by Ball (1992) existing runoff routing models such as RORB and RAFTS consist of only two components, the generation of runoff and collection to the catchment outlet. The collection system is modelled by a series of storages along the channels within the catchment. The proposed model provides for both the collection of runoff (via the hillside storage) and the transportation (via the channel storage).

The model type is therefore worthy of recognition as a separate class of model, to be named the RRR (or Rainfall Runoff Routing) model. The model is so named because like rainfall runoff models it models hydrological processes, and like runoff routing models these processes are represented by a series of concentrated storages.

The hillside storage must be able to be split to allow for the different contributions from the different processes occurring. Since each process on the hillside is assumed to enter the channel by a separate path it is allowable to have non-linear storage in the hillside part of the model. The RAFTS model contribution to each node, lumps channel and hillside (or process) storage. Part of the lag is due to the entry to the channel system, and part is due to flow in the channel. Both are non-linear in response. It is this non-linear channel response that causes the problem, as the catchment lag will be due to the sum of the components. There cannot be different lags for each component. In the RRR model flow is moving towards the channel system by different flow paths, not the same flow path as in the RAFTS model. It is therefore acceptable to model each process in a non-linear fashion.

The channel storage is likely to be linear for most flows as evidenced by Pilgrim's travel time work, supported by Chapman's common unitgraph.

A model structure is therefore proposed as follows,

- The model has ten equal channel reaches of length d/10, where d is the longest flow path length in the catchment (km). It is assumed that the area contributing to the downstream end of each reach is also equal, ie. total catchment area/10;

- Channel storage for each channel reach is modelled as a linear storage of the form $S = 3\,600\ k\ Q$;
- Contributions from any number of separate hydrological processes can be added at the downstream end of each channel reach before routing through the channel storage.
- Each of these processes is modelled as per Laurenson's Runoff Routing Model (Laurenson, 1964), as used in the RAFTS model (WP Software, 1994) ie. ten equal storage elements in series each with a storage $S = 3\,600\ k_p\ Q^m$, k_p being a lag related to runoff process. The total area of each process model is the total catchment area/10, so that the area contributing to each process storage is the (total catchment area/100).
- Each of the hydrological processes has an initial and continuing or proportional loss associated with it.

The use of ten elements for both the process and channel storages follows the Laurenson Runoff Routing Model, and provides for differing elements of rainfall excess to pass through different amounts of storage. The catchment is not however delineated with equal travel times, but with equal areas, as per the RAFTS model.

Laurenson (1964) reported that when using five sub-areas instead of ten less satisfactory results were obtained.

Figure 8-2 shows diagrammatically the structure of the RRR model. In a single sub-catchment model there is no actual catchment sub-division to be carried out, as must be carried out in the RORB or WBNM models.

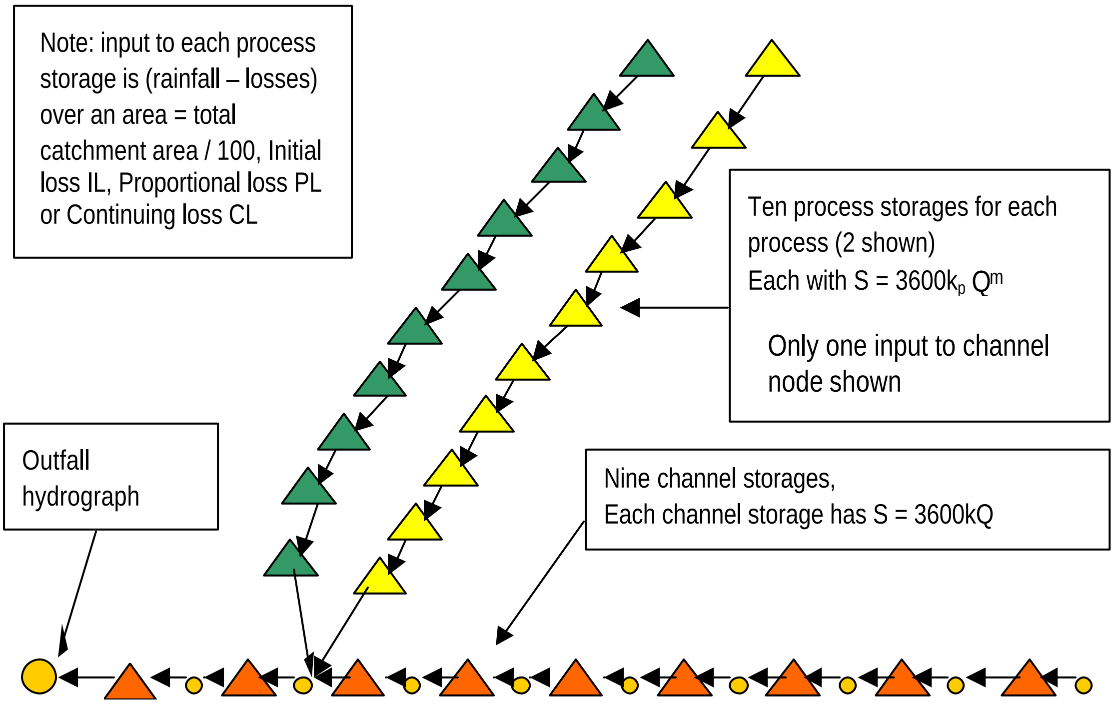


Figure 8-2 Structure of the RRR Model

Although the model may initially look complicated it is in effect simple as all elements are the same area, and storage parameters and losses need be input only once for the sub-catchment or node.

8.5.1 Identified Runoff Processes

It is considered that the model must conform to some basic restrictions if it is to be considered to be reasonable. These are:

- The predicted total runoff depth must be less than the total rainfall depth, to satisfy continuity;
- The number of runoff processes must be reasonable, and not in excess of the number that have been physically verified to occur in rural catchments; and
- There must be some uniformity across a range of catchments in the storage parameters to indicate that the model is a true representation of catchment behaviour, and that can be transferred to ungauged catchments.

The number of runoff processes can be further discussed, by reference to published findings on catchment processes and comparison with other models, including the rainfall runoff model, AWBM.

The identification and description of runoff processes is a continuing field of hydrology. However a description of current knowledge can confirm the number of separate runoff processes that may reasonably be incorporated into the RRR model.

Jayatalika and Connell (1996) summarise runoff generation mechanisms.

The dominant mechanisms and sources of runoff can vary depending on the effective climatic, geologic and topographic factors, vegetation characteristics and the antecedent moisture condition of the catchment. Most traditional concepts associate runoff generation with water from the rainfall event (event-water) and pre-event water (groundwater), which is a minor component of the streamflow. By contrast field studies in humid regions have indicated that groundwater could constitute a considerable proportion of streamflow.

Horton (1933) proposed that streamflow is generated from the infiltration-excess runoff, which occurs when the rainfall intensity exceeds the infiltration capacity of the surface soil. This Hortonian overland flow can be a major runoff generating mechanism in arid and semi-arid environments, where the presence of a less permeable soil surface layer and sparse vegetation cover allows the formation of a crust and the compaction of the soil which would enhance runoff generation by this mechanism.

A special case of the Hortonian overland flow is the partial-area contribution concept (Beston (1964)) where runoff is generated from certain fixed portions of the catchment with low permeability soils.

In more humid areas it has now been recognised that near stream wet areas cause runoff to be generated by several simultaneous processes. Overland flow caused by rain falling on wet regions close to streams is described by the partial-area effect of Ragan (1968). Runoff generation from rainfall excess on areas saturated by the emerging water table (saturation-excess runoff) is described by the variable source area-overland flow concept (Dunne and Black

(1970)). The contributing areas expand during rain events and contract during inter-storm periods. The variable source area-subsurface flow concept (Hewlett and Hibbert (1967)) implies discharge of subsurface water from the near stream wet regions. Streamflow generated by this mechanism would be comprised of water that existed in the porous medium prior to rainfall, that was subsequently displaced by the water from the rainfall event. Generation of runoff by this mechanism would be favoured by permeable soils in more humid regions where subsurface flow to the streams occurs.

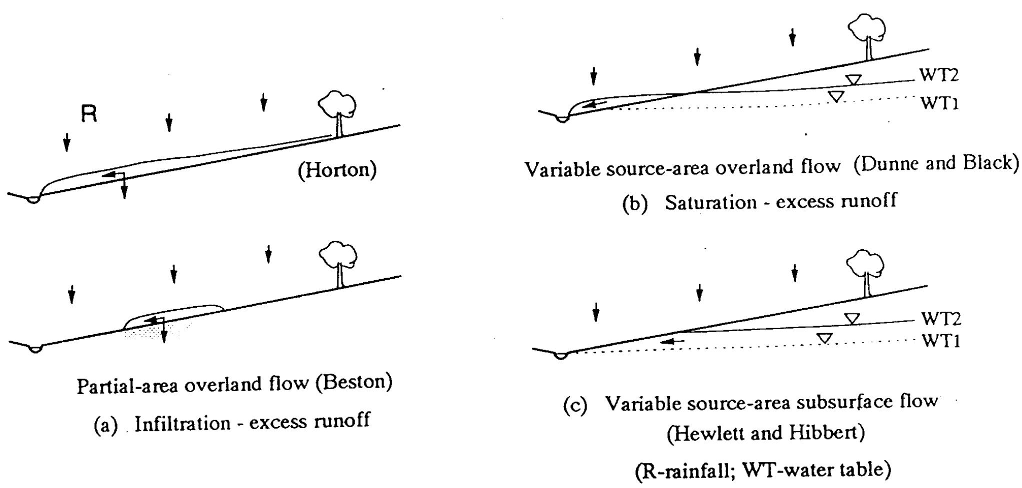


Figure 8-3 Runoff Generation Mechanisms (after Jayatilaka & Connell, 1996)

The use of chemical and isotopic tracers to separate streamflow into event and pre-event water components has shown results different to those that would be expected given the assumption that most flow apart from baseflow is derived from storm rainfall. Rodhe (1989) reported 65%-95% pre-event water in peak streamflow during rain events, based on studies of Swedish catchments. Chapman and Maxwell (1996) support this finding.

The results of these tracer studies were viewed with some degree of scepticism in that a mechanism that could cause the large and rapid increase in groundwater discharge to a stream in response to rain was not evident. A further mechanism, the capillary fringe mechanism has been proposed that would explain the rapid response of the groundwater flow.

Gillham (1984) described the physical basis of the mechanism. In the near stream areas of humid catchments the capillary fringe or zone of tension saturation above the water table often extends close to or is at the ground surface. The lateral extent of the area depends on the depth to the water table and the height of the capillary fringe, which depends on the texture and structural characteristics of the geologic material. In such situations, the medium above the water table has little or no storage capacity, and the application of a small amount of water can result in a large and rapid rise in the water table as a result in the conversion of tension-saturated capillary fringe to a positive pressure zone. Because of this, the water table in the vicinity of the stream would rise to the ground surface, creating a watertable mound near the stream.

Associated with this watertable mound a seepage face would develop adjacent to the stream, and the hydraulic gradient towards the stream would increase causing high groundwater discharge. The flow system established near the stream is highly transient because the magnitude of the flow components can change according to the growth and decay of the water table mound during and after the rainfall event respectively.

Figure 8-4 from Jayatilaka and Connell (1996) shows diagrammatically the capillary fringe mechanism.

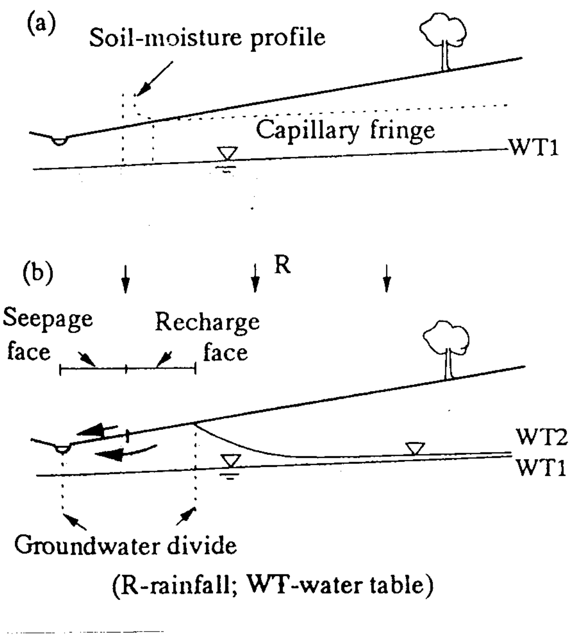


Figure 8-4 Schematic Showing Capillary Fringe Mechanism, (a) prior to rainfall, (b) shortly after onset (after Jayatilaka & Connell, 1996)

The groundwater contribution from the capillary fringe mechanism could be much larger than would be determined on the basis of the prevailing steady state regional groundwater flow system. The development of the water table mound adjacent to streams has been established by field studies, for example Ragan (1968), O'Brien (1980) and others.

Uhlenbrook and Leibundgut (1999) reported that observations and modelling of a small (39.9km^2) catchment in the Southern Black Forest, in southwestern Germany identified three runoff processes. These were labelled by the authors slow runoff (flow through fissured aquifers), delayed runoff (soil water displacement, or capillary fringe flow) and fast runoff (from saturated areas).

The above evidence suggests that the processes modelled by RRR can be separated into three processes. However the boundaries between one process and another may be blurred due to the non-homogeneity of catchment soils and structure.

The three processes and the associated characteristics are as follows:

- **Baseflow. This is the traditional concept of baseflow and is what is referred to above as the steady state regional groundwater system. It is known that the lag between rainfall and runoff by discharge to streams can be substantial, due to the long flow path length in the groundwater system;**
- **Slow flow, most probably capillary fringe flow. This mechanism acts with a lag from rainfall to stream flow that is less than that of the baseflow above, due to the quicker response time from rainfall to runoff into the stream; and**
- **Fast flow, most probably similar to Hortonian overland flow, either from a part of the catchment area, or the full catchment area. The response time of this mechanism is short compared with the two above, as no groundwater flow is involved**.

The RRR model structure is able to model the dominance of the Hortonian overland flow in arid and semi-arid catchments, and baseflow and capillary fringe flow in humid catchments because of the separation of processes on the catchment.

8.5.2 Other Models

Other models to compare with RRR are models that reflect more than one runoff process. These include hydrodynamic models such as those developed by Mesa and Mifflin (1986), Naden (1992), Kuczera (2000) and Littlewood and Jakeman (1992, 1994). All these models have a fast and slow response component. None of the models have three responses, with the third having no lag time for the hillside response. This is possibly not surprising, given that the third component (Hortonian overland flow) rarely occurs in humid areas where the models were developed.

The AWBM is a catchment water balance model that can relate runoff to rainfall with daily or hourly data, and calculate losses from rainfall for flood hydrograph modelling. It has been combined with a runoff routing model (RORB) by Muncaster et al (1997) to produce a continuous design flood estimation model.

The model uses three surface stores to simulate partial areas of runoff. The water balance of each surface store is calculated independently of the others. The model calculates the moisture balance of each partial area at either daily or hourly time steps. At each time step, rainfall is added to each of the 3 surface moisture stores and evapotranspiration is subtracted from each store. The water balance equation is:

$$store_n = store_n + rain - evap \quad (n = 1 \text{ to } 3) \qquad \text{Equation 8.11}$$

If the value of moisture in the store becomes negative, it is reset to zero. If the value of moisture in the store exceeds the capacity of the store, the moisture in excess of capacity becomes runoff and the store is reset to the capacity.

When runoff occurs from any store, part of the runoff becomes recharge of the baseflow store if there is baseflow in the streamflow. The fraction of the runoff used to recharge the baseflow store is BFI*runoff, where BFI is the baseflow index, ie. the ratio of baseflow to total flow in the streamflow. The remainder of the runoff, ie. (1.0 - BFI)*runoff, is surface runoff. The baseflow store is depleted at the rate of (1.0 - K)*BS where BS is the current moisture in the baseflow store and K is the baseflow recession constant of the time step being used (daily or hourly).

The surface runoff can be routed through a store if required to simulate the delay of surface runoff reaching the outlet of a medium to large catchment. The surface store acts in the same way as the baseflow store, and is depleted at the rate of (1.0 - KS)*SS, where SS is the current moisture in the surface runoff store and KS is the surface runoff recession constant of the time step being used.

The AWBM model is primarily a water balance runoff model, and only in a rudimentary fashion deals with the routing of flows to the catchment outlet.

The three surface storages of the AWBM may be represented in two different ways in the RRR model. Firstly it may be found that several process storages are necessary to model catchment runoff, each with a different lag (because of different flow paths to the channels). Each process has a loss related to the contributing area within the catchment. Secondly it may be found in the RRR model that one process has a variable continuing loss and storage lag, dependent on the magnitude of the contribution of this process. It would then be expected that there would be a relationship between continuing loss and the process lag c_p.

Baseflow in the AWBM can only occur when runoff occurs from the surface storages, so the initial loss of the baseflow and flow from the surface storages would be the same.

8.6 Running the RRR Model

The model can be easily set up using the XP-RAFTS interface. Figure 8-5 shows the RRR model in the XP-RAFTS format, with 3 processes being modelled. Each node of the RAFTS model on the main channel contains a linear reservoir, representing the channel storages. To each of these nodes up to two contributions can be made, each being modelled by a series of ten non-linear reservoir storages as per the Laurenson model. In the usual RAFTS model these two contributions are from the impervious and pervious parts of the catchment. However in the RRR model these represent two processes. In Figure 8-5 extra nodes contribute to each main channel node with zero delay time. It is therefore possible to model three processes.

Although the XP-RAFTS interface has been used in this study there is no reason why the structure could not be included into a single node, so the user would only have to input the process and channel storage parameters, as is the case in a single node RAFTS model.

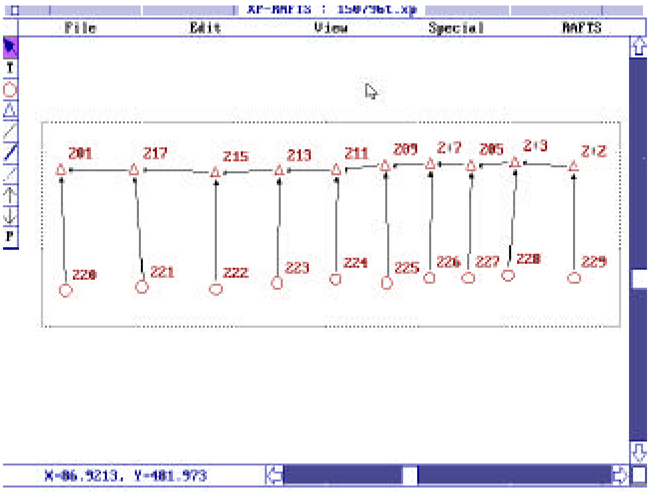


Figure 8-5 The RRR Model in XP-RAFTS Format

8.7 Parameters

The number of parameters applied to the RRR model needs to be addressed. Runoff routing models such as RORB have four parameters normally applied, two relating to storage (k_c and m) and two losses (initial loss and continuing or proportional loss). However these models can only model one runoff process.

The definition of a parameter must be considered. A parameter is a value that may vary from model run to run, usually due to changes in catchment conditions. It is not a constant but a variable. Thus physical catchment characteristics that do not vary cannot be considered to be parameters.

If the RRR model is applied in urban catchments the storage lags for both the directly connected and unconnected areas will not change, as the physical characteristics of overland flow (from a grassed area to a gutter) and gutter and pipe flow times do not change with catchment antecedent conditions.

By the above definition of a parameter the RRR model applied to urban areas does not have storage parameters. However in many cases the storage lags will have to be calibrated, but it can be expected that this will be a relatively simple exercise. If it is assumed that the proportional loss on impervious areas is zero then this is no longer a parameter. There are then only three parameters for the application of RRR to urban areas, being the impervious area initial loss, and the unconnected area initial and proportional loss.

For rural areas the model must be considered differently. Experience with the RORB model has shown in a wide range of application the use of the non-linearity exponent m = 0.8 is applicable. By extrapolation to the RRR model it can be expected that a value of m = 0.8 can be applied to most catchments, and will not be a parameter. This value will be used in the verification of the model unless it can be shown that the model does not perform well.

The number of parameters in the RRR model cannot be compared with that of the RORB model as they perform different functions. The RORB model handles only surface flow, and not baseflow. The only runoff routing model that does model baseflow is the RAFTS model, using Phillip's infiltration model to determine baseflow. This model uses a total of 17 parameters to determine runoff and baseflow. In addition one storage parameter (B) is required. The RRR model uses two loss parameters and one storage parameter per process, plus one parameter common to all processes, being the channel storage parameter k. Thus to model two processes a total of three storage parameters and four loss parameters are required, a total of seven parameters.

There are currently no runoff routing models that can predict the occurrence of three separate distinct runoff processes. To do this RRR needs a further two parameters, being the initial and proportional loss for the third process. In many cases this will not have to be modelled.

Another comparison can be made with a different type of model, being the AWBM (Boughton, 1996). For the use of AWBM with baseflow and 2 surface routing parameters to enable continuous hydrograph generation using hourly rainfall a total of 9 parameters is necessary.

Thus the RRR model compares favourably with existing models in terms of the number of parameters.

8.8 Fitting The Model

Since the concept of the RRR model is original it was necessary to investigate the applicability of the model to various catchments. Two rural catchments and one urban catchment that had been previously modelled by RORB and RAFTS were selected for testing. The location of the catchments is shown on Figure 8-6.

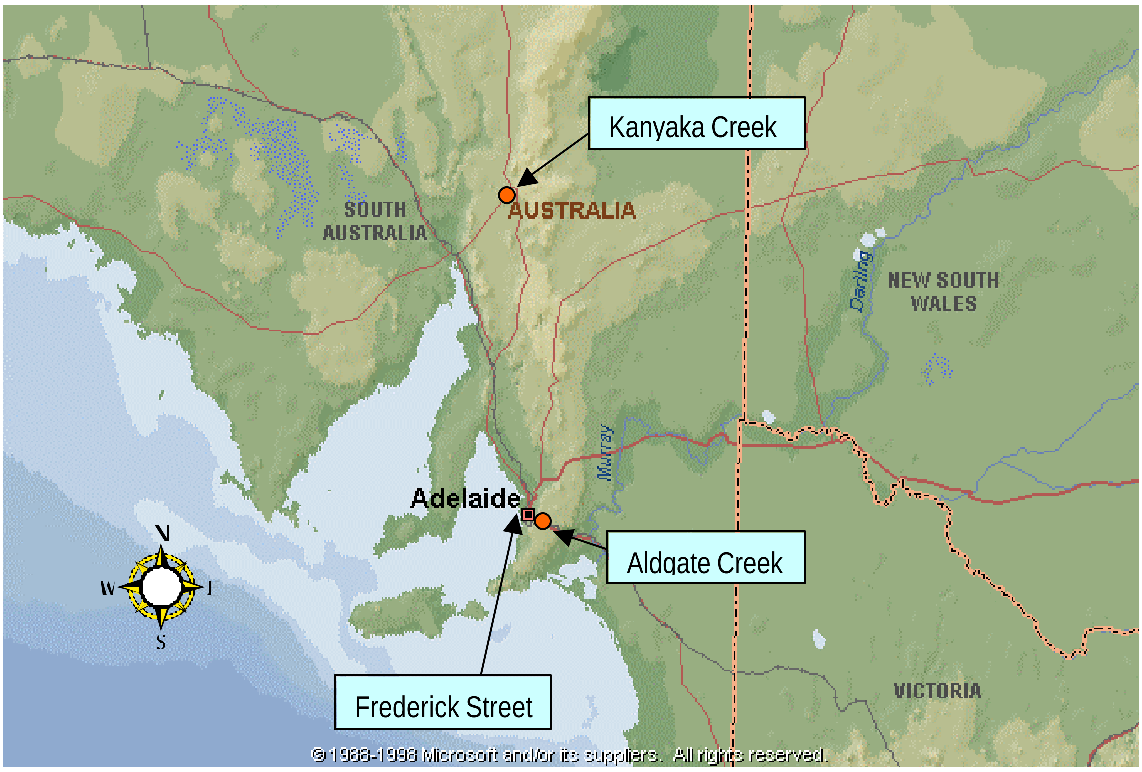


Figure 8-6 Catchments Chosen for Initial RRR Modelling

8.8.1 Aldgate Creek

The model was first tested on the Aldgate Creek catchment (AW503509), which is located in the Mount Lofty Ranges. Aldgate Creek has a catchment area of 7.8km^2 and an average annual rainfall of approximately 1000mm. The model was set up as described previously and calibrated using the following procedure for an event in September 1973. The event was selected due to the inability of the RORB model to match the second peak of the gauged hydrograph.

Figure 8-7 shows the recorded rainfall and hydrograph.

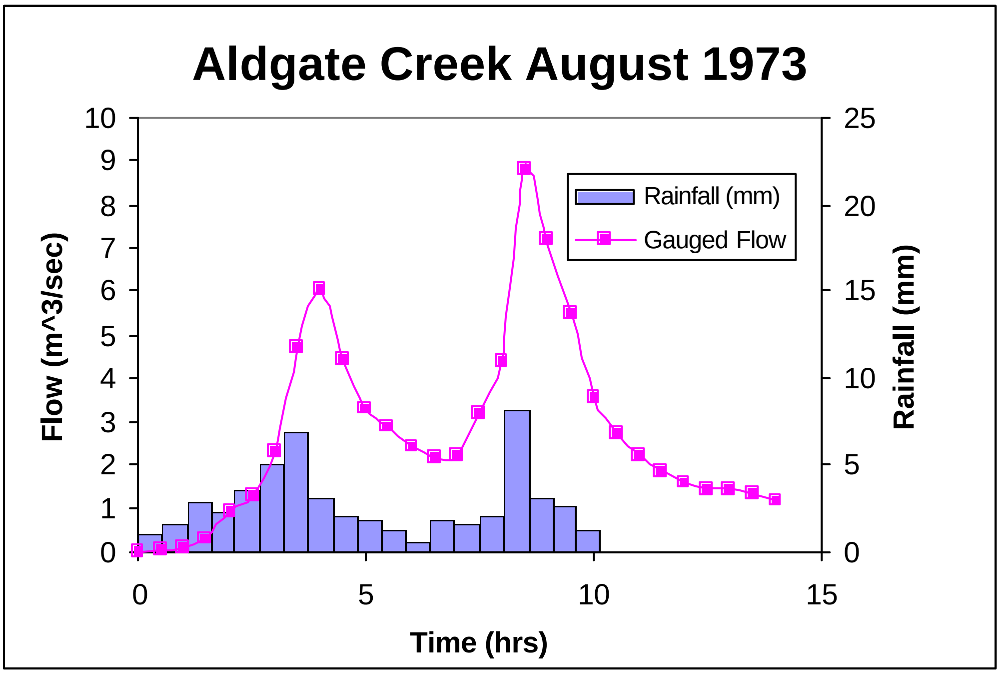


Figure 8-7 Aldgate Creek, 1973 Event

The RRR calibration was carried out as follows:

- Modelling was initially carried out without the baseflow contribution (baseflow separation was carried out in the previous investigation);
- It was assumed that flow was occurring from one process alone. The initial loss was then set to model the start of rise of the hydrograph. A proportional loss was set and the value of k for the channel and k_p for the first process varied to match the outflow hydrograph for the commencement of the runoff event. A value of m = 0.8 was used, in line with normal practice for ungauged catchments. It was found that the main effect of k_p was to vary the shape of the hydrograph, and k was to provide a translation. It was thus possible to match the start of the event, but as the event progressed in time there was obviously another contribution to runoff;
- The next contribution was assumed to be from another process. Thus a second process storage was introduced, leaving the contribution from the first process as calibrated. The initial and proportional losses were set to model the start of the second contribution and the total volume of the contribution. It was found that k_p for the second process needed to be set to a very small value (RAFTS allows 0.001) to model the contribution;

- The full hydrograph was then modelled, including baseflow. All other parameters were set as above, and k_b, initial loss and proportional loss for the baseflow contribution were varied to match the total hydrograph;

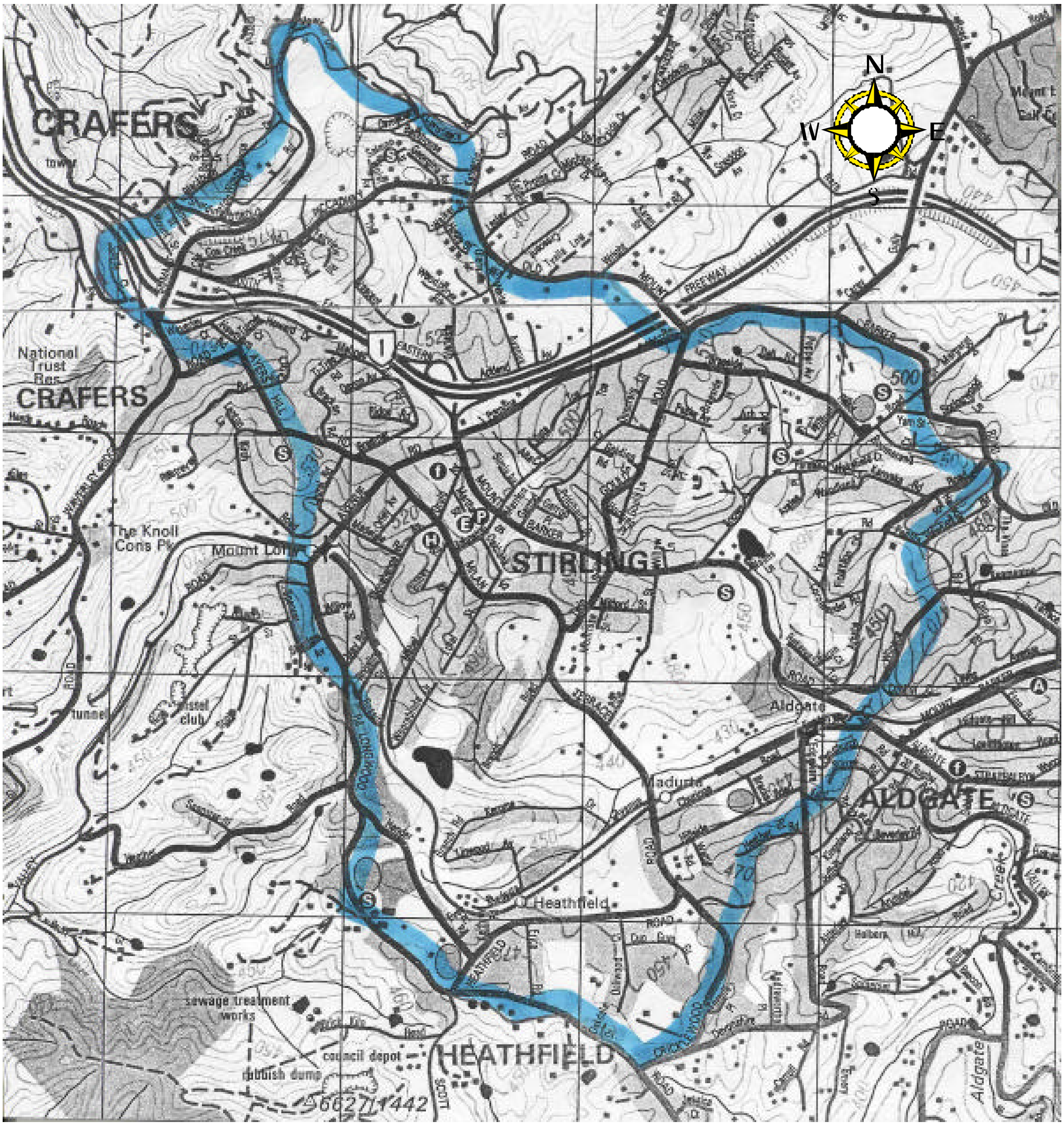


Figure 8-8 Aldgate Creek Catchment

Figure 8-9 shows the fit obtained using the RRR model for the September 1973 event. Table 8-2 lists the fitted parameters. The channel lag parameter k was 0.05 hours.

Table 8-2 Aldgate Creek RRR Model Fitted Parameters, September 1973.

Contribution	IL (mm)	Proportional Loss	k_p
Base	0.0	0.80	0.9
Slow	3.0	0.78	0.1
Fast	42	0.82	0.0

Several important findings came out of the calibration:

- There were three distinct process related lags;
- The baseflow contribution shape was as expected if a baseflow separation method was used; and
- The contribution of the fast runoff did not require any greater lag than would be expected from the channel storage (k_p = 0.0). This is a surprising result, but indicates that when surface runoff is occurring the catchment lag is equivalent to the lag within the channel system.

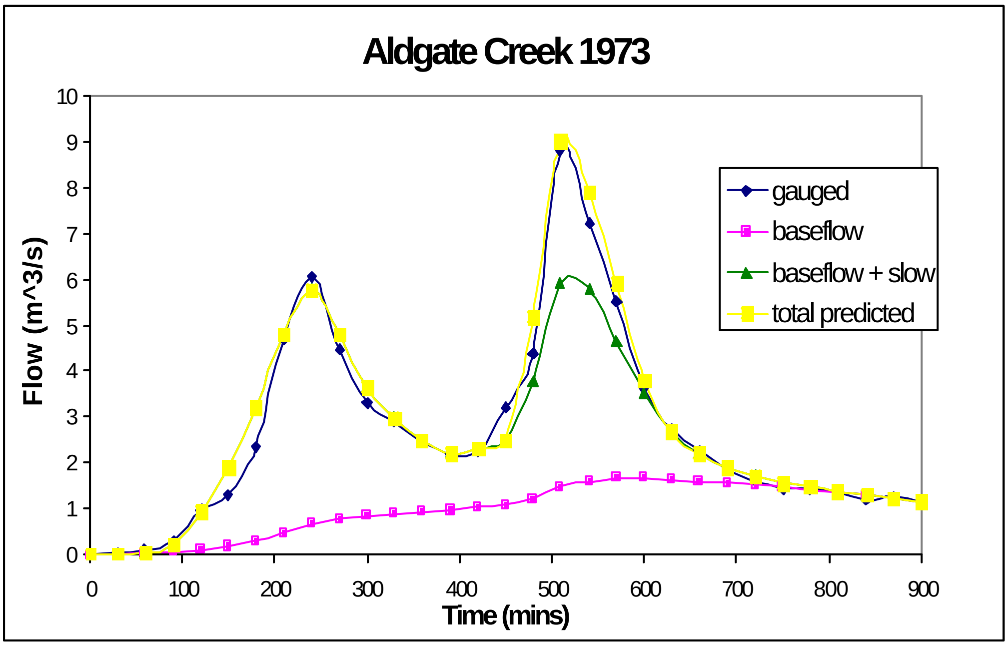


Figure 8-9 RRR Model Applied to Aldgate Creek

The best fit that could be obtained using the RORB model with a single rainfall burst approximated that of the baseflow plus slow flow contribution, without matching the second peak of the hydrograph (Figure 8-10). If the rainfall was modelled as two bursts, with a lower loss for the second burst then a better fit could be achieved. However the shape of the second hydrograph rise is not modelled as well in RORB, as the same storage parameter is being used.

Table 8-3 Aldgate Creek 1973 RORB Model Parameters

	K_c	m	Initial Loss (mm)	Runoff Coefficient
Single Burst	1.0	0.8	2.0	0.18
Two Burst – First Burst	1.0	0.8	5.0	0.15
Two Burst – Second Burst	1.0	0.8	3.0	0.31

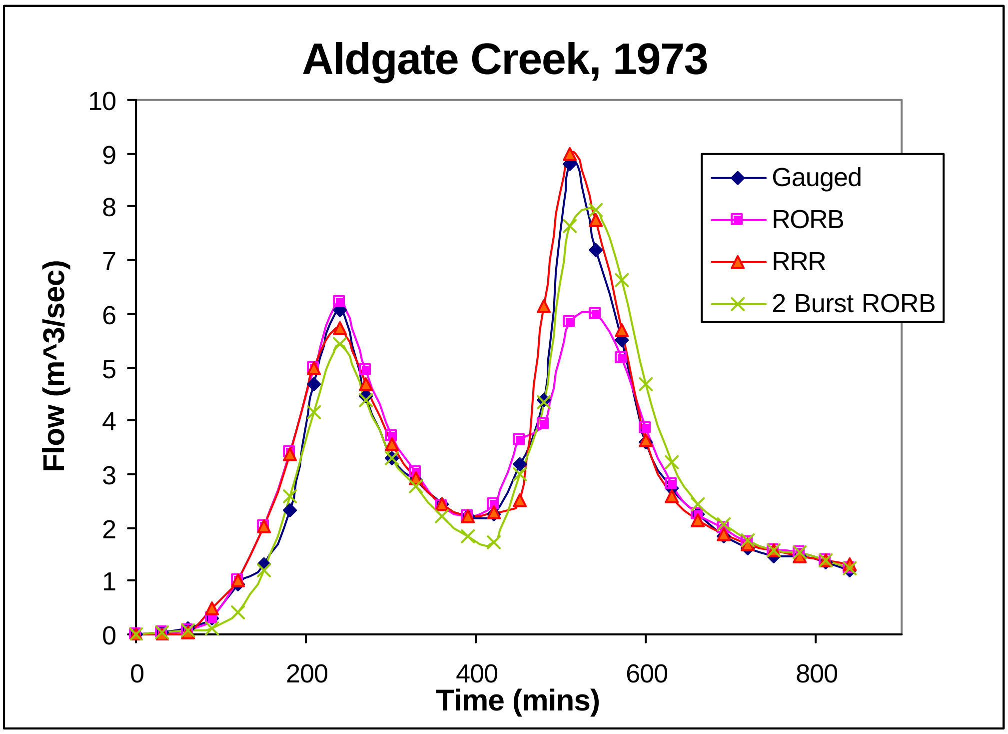


Figure 8-10 Comparison of RORB and RRR on Aldgate Creek

8.8.2 Kanyaka Creek

Kanyaka Creek (AW509503) is located in the Flinders Ranges in an area having an annual rainfall of approximately 300mm. It has a catchment area of 180km^2. A RORB fit run was carried out for the storm event of March 1989. Figure 8-11 shows the rainfall and recorded hydrograph.

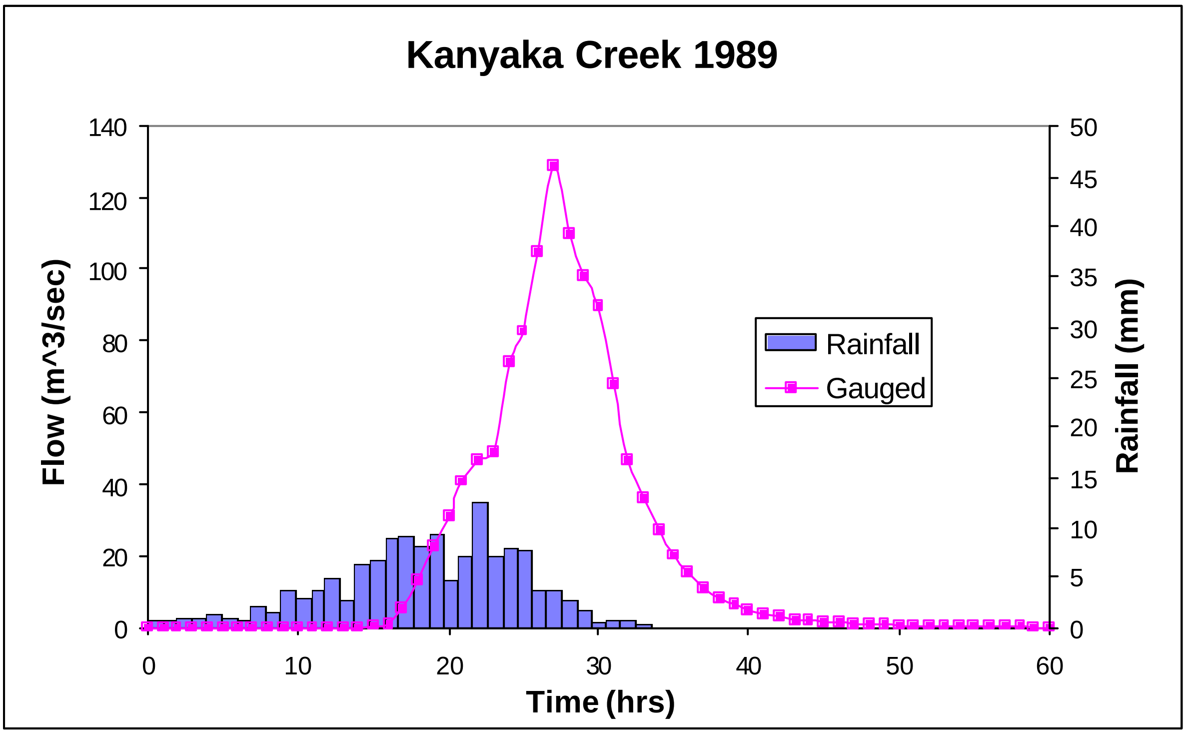


Figure 8-11 Kanyaka Creek March 1989

The RORB model had difficulty matching the start of the rise of the hydrograph and the peak. If the start of rise was matched in the RORB model, the peak could not be matched.

A RRR model fitted using the procedure adopted for Aldgate Creek resulted in a much better fit, and indicated contribution from both slow and fast flow. No baseflow was apparent in the gauged hydrograph.

Table 8-4 and Table 8-5 summarises the fitted parameters. The channel lag parameter k was 0.25 hours. Figure 8-13 shows the fit obtained, and Figure 8-14 shows the comparison of RRR and RORB.

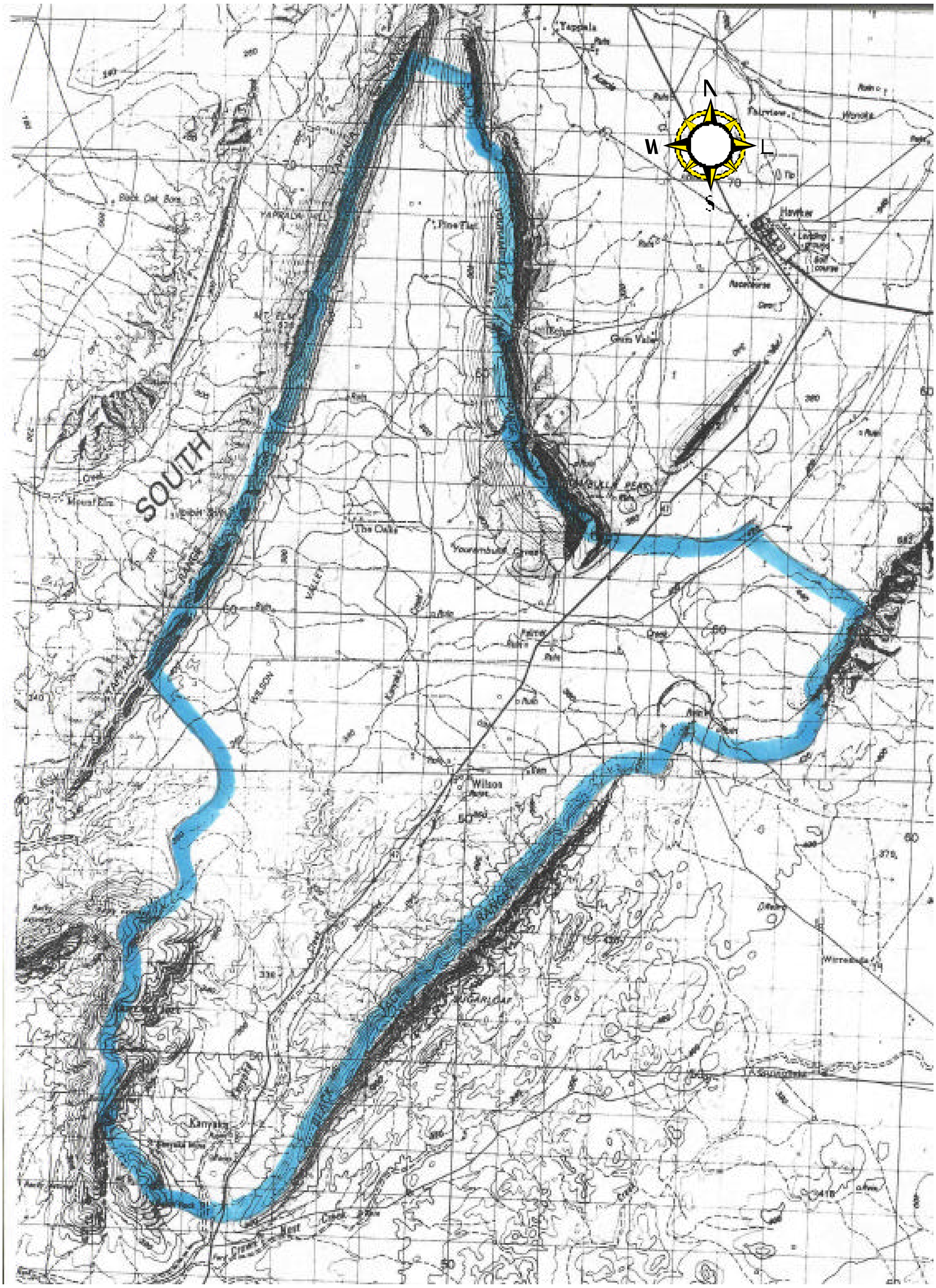


Figure 8-12 Kanyaka Creek Catchment

Table 8-4 Kanyaka Creek RRR Model Fitted Parameters, March 1989.

Contribution	IL (mm)	Proportional Loss	k_p
Base	-	-	-
Slow	32	0.85	0.9
Fast	105	0.80	0.0

Table 8-5 Kanyaka Creek RORB Model Fitted Parameters, March 1989

	K_c	m	Initial Loss (mm)	Runoff Coefficient
Match Time of Rise	8.0	0.8	35	0.23
Match Peak	10.0	0.8	75	0.37

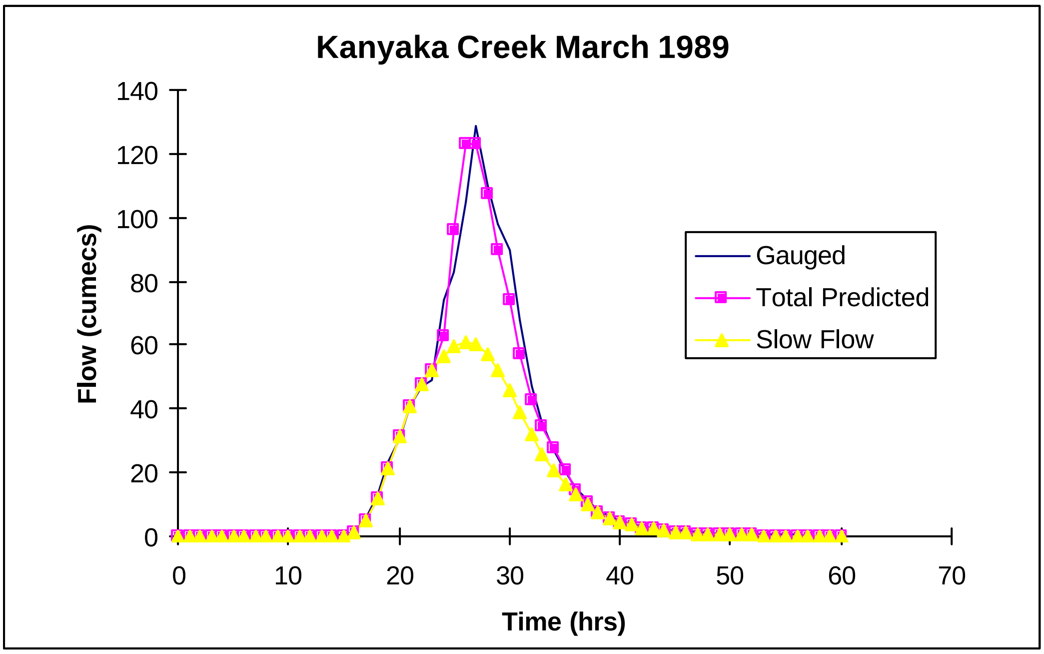


Figure 8-13 RRR Model Applied to Kanyaka Creek

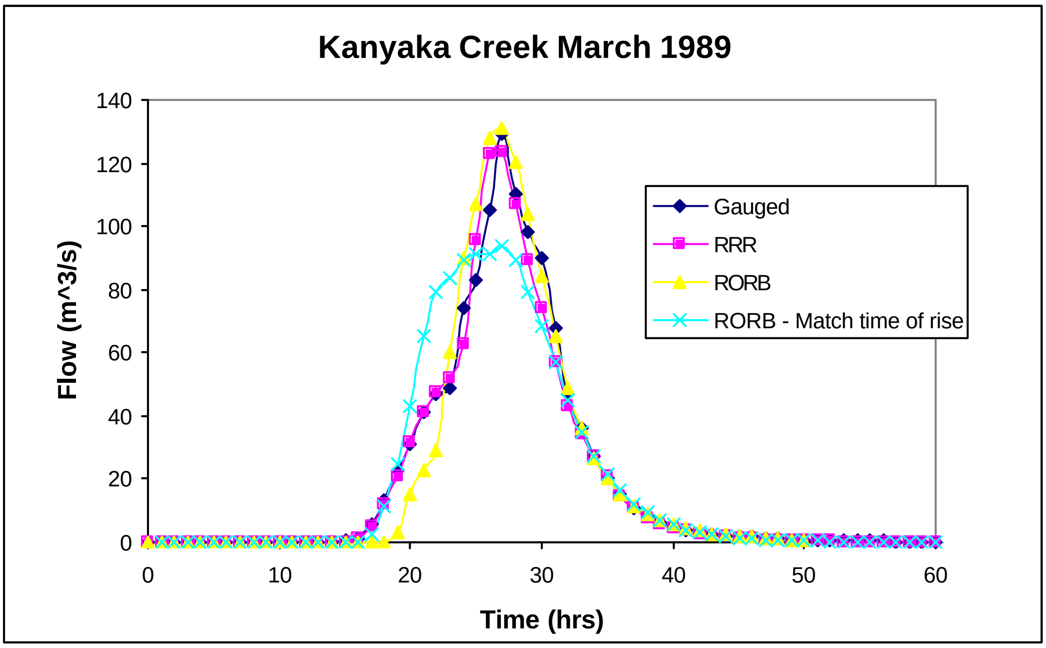


Figure 8-14 Kanyaka Creek March 1989, Comparison of RORB and RRR

8.8.3 Frederick Street, Glenelg

The RRR model was fitted to one storm event on an urban catchment at Glenelg, South Australia. This was done to see how the model worked on an urban catchment. A good fit was achieved without using a time shift that was required when fitting the RAFTS model to the catchment.

Figure 8-15 shows both the fit obtained by the RAFTS model and the RRR model.

Both the process and channel storage in this case were found to be linear (ie. m=1). Only one process was required, which represented the runoff from the directly connected impervious area within the catchment. The process lag k_p was 0.011 hours indicating a total storage delay time of 0.11 hours or 6.6 minutes. This is of the same order as the normal time of entry of the ILSAX model (5 mins), which has been shown to be equivalent to a storage delay time.

The fitted channel lag k for each storage was 0.036 hours (2.16 mins). As the process lag represents the time of entry it can be concluded that the channel storage lag represents the total pipe plus gutter lag.

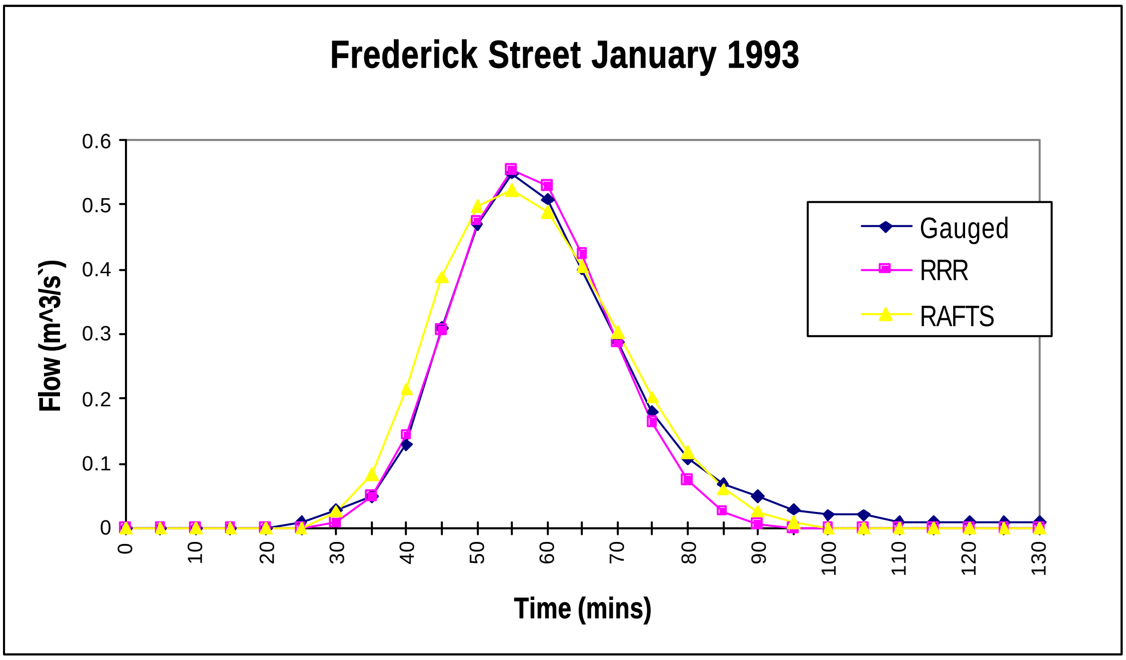


Figure 8-15 RRR Model Applied to Glenelg Catchment

8.9 Summary of Trial Application of the RRR Model

In the above trial applications the simple model structure of the RRR model improves the level of fit observed in the events examined so far compared with the normal runoff routing model. This is because the model provides a better representation of the processes that area actually happening on the catchment. The downside is that there are more parameters to be determined.

One great advantage of the RRR model is that it directly models baseflow. Thus no baseflow separation, with the attendant uncertainty has to be carried out. When design flows are calculated the baseflow contribution can be incorporated like any other contribution, instead of as an arbitrary allowance.

However if baseflow is occurring at the time modelling commences the model would have to be "run-in" with prior rainfall data to model the correct baseflow.

The model is expected to give a better representation of the catchment response during extreme events when the fast flow contribution may become dominant. In extreme events catchments will appear to be giving a more linear response with a lag that is dependent on the characteristic flood wave velocity v_c of the channel. This velocity can be obtained by fitting more frequent events on the catchment. For extreme event prediction the use of the RRR model will therefore change the present approach, where catchment behaviour for extreme events is based on catchment behaviour with the more common events. Examples of the application of the RRR model to extreme events are given in Chapter 11.

A RORB, RAFTS or WBNM model calibrated on events having only base and slow flow runoff should not be extrapolated to apply with extreme rainfall, as they cannot account for the change in runoff process that may occur with extreme rainfall. The RRR model can be calibrated on smaller events, and used on extreme events, as the storage parameter for fast runoff can be determined from the events generally used for calibration.

With further investigation of how the losses for each process vary it is possible that the RRR model can be extended to cover the time steps usually associated with rainfall runoff models, and be used for daily flow prediction. The process sub-catchments form the storages associated with the normal rainfall runoff model, with the losses representing the exchange of water between the storages

8.10 Expected Generalised Parameters

8.10.1 Lag Parameters

It could be expected that for rural catchments channel lag is highly correlated with the mainstream length of the catchment. Indeed for the purposes of the derivation of a generalised parameter, a further variable representing the characteristic flood wave velocity v_c could be introduced. This can be related to channel lag k on the basis that there are ten channel reaches, and that the channel response is linear. Equation 8.12 can be derived, allowing for the number of channel reaches and the conversion of lag time, which is in hours.

$$v_c = \frac{d}{36k}$$ Equation 8.12

Where	v_c	is the channel characteristic flood wave velocity (m/sec)
	d	is the longest flow path length (km)
	k	is the channel storage lag parameter (hrs)

For the Aldgate Creek and Kanyaka Creek v_c is 2.5 m/sec and 3.0 m/sec respectively, based on the fitted k value and the longest stream length within the catchment.

8.10.2 Losses

It would be expected that losses would be related to catchment moisture condition, for example by the catchment Antecedent Precipitation Index. If these relationships can be found then the model can be extended to run as a full rainfall - runoff model.

8.11 The RRR Model - Multiple Sub - Catchments

The RRR model so far has been derived for use as a single uniform catchment, with one rainfall input. It is thus limited in that spatial variability cannot be accounted for. In addition, the model cannot predict flows at points within the catchment, as the total storage within the catchment is evenly distributed within the model, unlike real catchments.

To overcome these limitations an extension of the simple RRR model as proposed must be undertaken, so that the simple RRR model can be used as a sub-catchment within a larger catchment. In doing so it is important that internal consistency is retained within the model.

8.11.1 Rural Catchments

To model catchments having spatial variability a model is required that has sub-catchments. **The simple RRR model described above can be used as a sub-catchment in a multiple sub-catchment model. However consideration must be given to deriving generalised parameters so that there is not an effect on predicted flows due to the number of sub-catchments. From these generalised parameter values individual sub-catchment storage parameters must be determined.**

Only in this way will the model retain internal consistency.

The RRR model is fundamentally different to other runoff routing models in that the storage lag within the catchment is made up of a component due to the process storage, and a component due to the channel storage.

Consider first the channel storage component. If a catchment is sub-divided it can be expected that the mean translation time to the outlet of all process storage elements contributing to channel storage elements will remain constant. This follows from the assumption of linear channel response, where the storage lag time in the channel is not dependent on the flow within the channel. Thus the number of sub-catchments has no effect on the channel storage lag. If this is not the case, and the storage lag time is dependent on the flow in the channel, then as the catchment is sub-divided then the storage lag will change, as the flow within the channel storage elements change.

However the non-linearity of most process storages creates a problem in that the process storage lag depends on the storage outflow, which is in turn dependent on the modelled catchment or sub-catchment area. The process storage represents the flow of water from the hillsides to the channel. It would be expected that over a catchment the length of the flowpath would remain relatively constant, as would the inflow per unit length of the channel.

It is proposed therefore that a new variable be used, being the catchment characteristic lag parameter, c_p, where:

$$k_p = c_p\, A^{1-m} \qquad \text{Equation 8.13}$$

where A is the area of the catchment or sub-catchment (km^2)

m is the exponent in the process storage relationship

k_p is the process storage parameter for an individual sub-catchment

The reason for the use of this parameter is as follows. The lag of a single set of process storage elements contributing to the channel is given by the equation:

$$lag = k_p\, Q^{m-1} \qquad \text{Equation 8.14}$$

Where Q is the flow contributed to each channel storage by the set of process storages. But it can be seen that the lag of the catchment process storages changes as the area of the modelled catchment changes, as Q is dependent on the area represented by the process storages. If c_p is used the lag is then:

$$\begin{aligned} lag &= c_p\, A^{1-m}\, Q^{m-1} \\ &= c_p \left[\frac{Q}{A}\right]^{m-1} \end{aligned} \qquad \text{Equation 8.15}$$

But Q/A is the inflow per unit length of the channel, which will remain constant as the width of the hillside will remain constant. The process lag is then also constant. It will not now depend on catchment area.

The use of this form of storage lag equation was used in the formulation of a catchment model having a variable number of sub-catchments. The Aldgate Creek catchment was modelled, as was done for the investigation of the effect of the number of sub-catchments in the RAFTS model. Models with 1, 2, 5 and 10 sub-catchments were set up in two different formulations. Firstly the sub-catchments were joined in series, that is the channel reach storages of one sub-catchment contributed to the upstream storage reach of the next downstream catchment. In the other formulation the model was set up with each sub-catchment outflow contributing at the downstream end of the sub-catchment, with the contribution of the upstream sub-catchment being translated through the main stream with the velocity v_c. The mainstream length d of each sub-catchment then became the longest stream length within the sub-catchment. This formulation is similar to RAFTS and WBNM in the splitting of upstream and sub-catchment storage.

Table 8-6 gives the results of both models with the same storm and losses, keeping v_c and c_p constant for all models, and from these deriving the channel storage parameter k and the process parameters k_p for each catchment or sub-catchment.

Table 8-6 Aldgate Creek Multiple Sub-catchment RRR model

Number of Sub-catchments	Series Model		Split Model	
	Peak Outflow (m^3/s)	Time to Peak (mins)	Peak Outflow (m^3/s)	Time to Peak (mins)
1	59.5	74	59.5	74
2	60.1	71	60.0	75
5	61.6	75	60.2	75
10	62.6	73	62.2	76

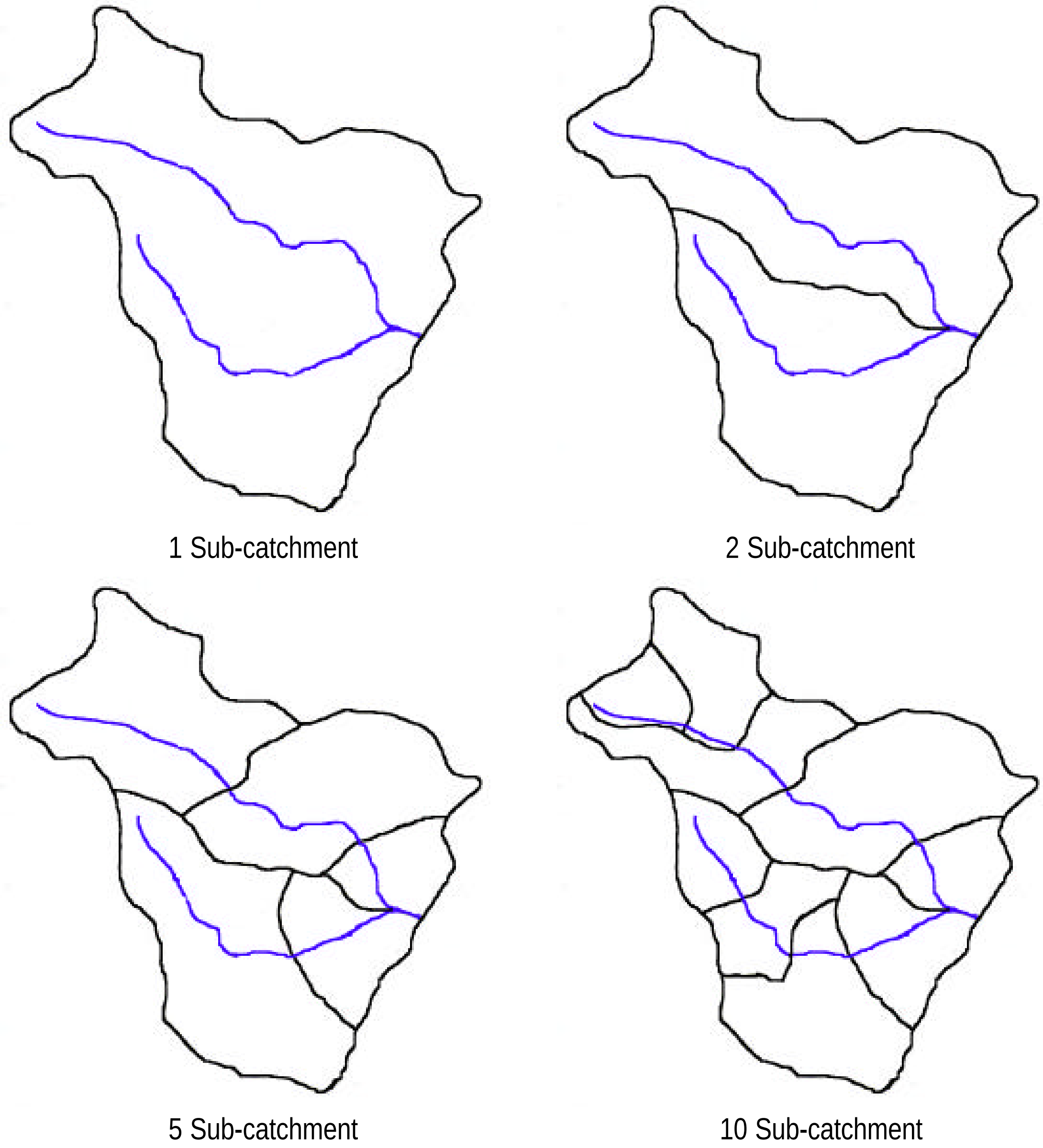


Figure 8-16 Aldgate Creek RRR Model Sub-division

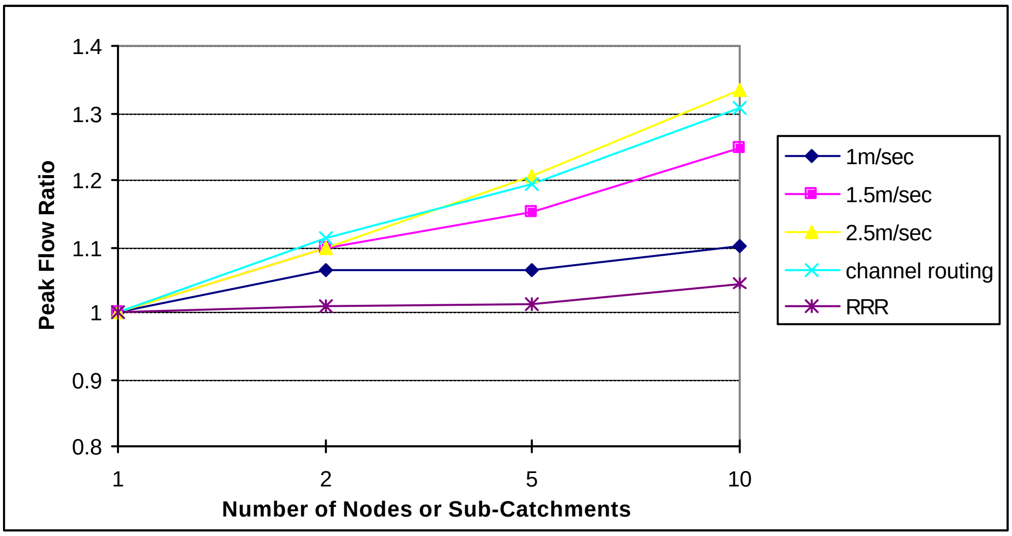


Figure 8-17 Comparison of RRR and RAFTS Models - Aldgate Creek

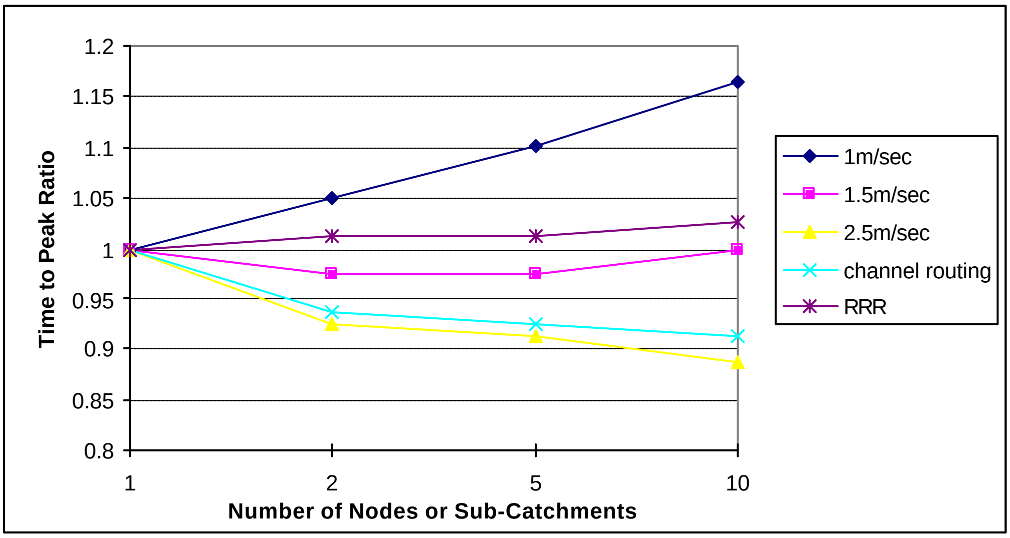


Figure 8-18 Comparison of RRR and RAFTS Models - Aldgate Creek

The results show that the models are stable, and certainly better than a RAFTS model. Although there is an effect of the number of sub-catchments for the model, the number of sub-catchments is not as significant as for any other model in current use. The difference in peak flow and time to peak is less than 5%.

In summary, provided the global storage parameters, being the characteristic velocity v_c and the lag parameter c_p are used, the number of sub-catchments in the RRR model does not have a significant effect on the predicted flows. It can be seen also that the model will be internally self-consistent, as the layout of the network of sub-catchments has no effect on the results.

8.11.2 Urban Catchments

The form of the RRR model with more than one sub-catchment on urban catchments can be expected to follow that of the split format of the rural catchment. Each sub-catchment has its own k and k_p, with flow from upstream sub-catchments being translated at the velocity of flow in the pipe or channel connecting the sub-catchment outfalls.

The value of k for each sub-catchment is dependent on the longest pipe and gutter flow time within the sub-catchment. A difficulty arises when calibrating the RRR model on a multiple sub-catchment urban catchment in that an assumption must be made because of the relationship between gutter and pipe flow lengths and slopes, and the value of k.

However, some clue may be gained from the previously derived equation (Equation 4.19) for lag in urban catchments, based on the ILSAX model. In this case m=1, ie. the catchment is linear. The derived equation is:

Equation 8.16

$$B_i = \left(\left(0.333x10^{-3}\right) \sum_{i=1}^{n} \frac{L_{pi}}{r_i^{0.667}\sqrt{s_i}} + \left(3.63x10^{-3}\right) \frac{L_g}{\sqrt{s_g}} + 8.3 \right) x\ 10^{-3} \quad hours$$

It can be seen that the lag is made up of two parts, the first being the lag due to the gutter and pipe flow and the second a constant being the time of entry to the gutter and pipe system. It could be expected then that the first part represents k, and the second k_{pi}, the impervious area process lag parameter. The values of the two parameters are then:

Equation 8.17

$$k = \left(\left(0.333x10^{-3}\right) \sum_{i=1}^{n} \frac{L_{pi}}{r_i^{0.667}\sqrt{s_i}} + \left(3.63x10^{-3}\right) \frac{L_g}{\sqrt{s_g}} \right) x\ 10^{-3} \quad hours$$

$$k_{pi} = 0.0083\, hours$$ Equation 8.18

This can be compared with the value of 0.011 hours derived by calibration on the Frederick Street, Glenelg catchment. The k_{pi} represents the time of entry to the gutter system, which in an urban area could be expected to remain constant.

As the catchment is divided up into sub-catchments this sub-division will not affect the time of entry to the gutter for either the impervious or pervious portion of the catchment. The sub-division will not affect the mean translation time within the catchment. **Thus in urban areas catchment sub-division will not affect the predicted flow from the RRR mode.**

8.11.3 Mixed Urban and Rural Catchments

It has been shown that the outflow from the RRR model does not depend on the number of sub-catchments. Therefore a mixed urban and rural catchment can be modelled by the use of separate sub-catchments for the urban and rural portions, with the flows from upstream sub-catchments being translated at the characteristic velocity v_c for natural channels, or the pipe or channel flow velocity as appropriate.

8.12 Conclusions

A model structure has been formulated that overcomes the limitations identified in other runoff routing models, these being the limitation on the number of processes modelled and the lack of internal consistency in the models. The model has been applied to three catchments to demonstrate that the model functions according to the theory. It was found that three processes were present on one of the rural catchments, and two processes on the other.

Provided global storage parameters, being the characteristic velocity v_c and the lag parameter c_p are used, the basic RRR model can be used as a sub-catchment model in a multiple sub-catchment model. The number of sub-catchments in the model does not have an effect on the predicted flows. The model will be internally self-consistent, as the

layout of the network of sub-catchments has minimal effect on the results throughout the catchment.

It is difficult to specify the point at which the user should say "the model is acceptable" or "the model is not acceptable". This depends on the use to which the model is being put and the quality of the test data available.

Eric Laurenson, 1975

9. Confirmation of the RRR Model

9.1 Introduction

The performance of the RRR model during a process of calibration and verification will confirm the benefit of the model in the prediction of outflow hydrographs from catchments, given a rainfall input.

The difference between calibration and verification needs explanation. A model is calibrated by the applying the model on a catchment with given rainfall events and adjusting model parameters to match the predicted outflow hydrograph with the measured hydrograph.

Once the model has been calibrated on a number of storm events and the parameter values predicted the model is verified by the application of the predicted parameter values to an independent set of rainfall events. A measure of the fit is determined between the predicted and measured outflow hydrograph.

If the model verification is considered reasonable it can be used for the prediction of flows from the catchment.

The RRR model has two groups of parameters, these being the storage parameters and the loss parameters. The loss parameters determine the amount of runoff, given the rainfall input. The storage parameters vary the response time of the catchment to the runoff. In the case of the RRR model this occurs for several different processes.

The estimation of both loss and storage parameters for all runoff routing models has been ongoing since the initial development of the models. The development of the RRR model is partly in response to the lack of success in parameter estimation for existing runoff routing models. The splitting of the model such that several runoff processes can be modelled allows better representation of catchment response, and thus more stable values for both the loss and storage parameters.

However this current study reviews only the basic requirements to predict loss and storage parameters, especially for ungauged catchments. There is a particular problem in the definition of initial loss, in that the definition of the start of the event determines the initial loss that must be used to calibrate the model. Unless a better estimation of initial loss is found it may be difficult to achieve reasonable results in the verification of the model. This is a problem with the verification of most models, apart from those that provide continuous simulation, and thus do not have an initial loss as such. Some modellers calibrate initial loss for each verification event (Kuczera, 2000) on the basis that this allows a fairer comparison of model performance.

Hill et al (1998) have presented a review of loss modelling and developed a procedure for determining design losses for use with the RORB, RAFTS or WBNM models. The initial loss was examined for both a total storm and for a rainfall burst within the storm, and the burst loss used for design purposes, as Australian Rainfall and Runoff design storms are derived from rainfall bursts. Some attempt was made to incorporate the concept of variable source area in the loss modelling by the inclusion of a variable proportional loss model.

Because of the differences in the way in which urban and rural catchments behave the process of model calibration and verification will be dealt with separately for each type of catchment. In both cases the process will be explained, followed by detail of the application of the model on a range of catchments.

9.2 Urban Catchments

The ability of the RRR model to model urban catchments can be gauged by comparing the level of fit achieved by the RRR model and the ILSAX model on the same catchments, with the same storm events. However it would be fair to ignore those storm events that produced flows in excess of the capacity of the pipe system. The simple one sub-area RRR model cannot model these flows, as the lag of the surcharge flows would be different. However a more complex RRR model can be formulated to account for this, as is used in the Keswick and Brownhill Creek catchments, detailed in Chapter 11. The storage parameters to be used are those derived in Chapter 8.

This process is a direct verification of the RRR model, as the storage parameters have been fixed by means other than by calibration. The only calibration required relates to loss parameters. In the case of urban catchments the continuing loss on the impervious part of the catchment directly connected to the pipe system can be set at zero, as minimal loss is expected from these areas. This follows the ILSAX model procedure. The initial loss for the directly connected impervious area of the RRR model is then set to match the start of the time of rise of the recorded hydrograph. This loss is generally less than 2mm and again follows ILSAX, in which depression storage of 2 mm is generally used in design runs.

The model has been evaluated against the ILSAX model on three catchments, the Glenelg catchment, the Paddocks catchment and the Jamison Park catchment in New South Wales. The ILSAX models and fit runs for Jamison Park are described in Haig (1989), and were obtained from Associate Professor Geoffrey O'Loughlin, then of the University of Technology, Sydney.

9.2.1 Glenelg Catchment (Frederick Street)

The RRR model was set up for Frederick Street using the physical data from the ILSAX model, and the given formula for the storage parameters. The pipe $l/s^{0.5}$, and the mean gutter $l/s^{0.5}$ were calculated from the ILSAX pipe file.

There are two pluviometers within the Frederick Street catchment, and to include information from both the RRR model consisted of two sub-catchments. Unlike other models, the RRR model does not have a minimum number of sub-catchments, and so two only are required. The catchment rainfall input is thus similar to the ILSAX model. The directly connected impervious area was set to 13.2ha, which was found to give the best fit for the 1992 and 1993 storms fitted with ILSAX. This represented an adjustment of -10% on the measured directly connected impervious area, which was transferred to unconnected area. If ILSAX modelling had not been carried out to determine the directly connected area, it would have to have been assumed on the basis of like catchments and calibrated by comparing predicted and actual runoff volumes for those storm events having runoff only from the directly connected impervious area.

The channel lag parameters for the two sub-catchments were determined as given in Table 9-1.

Table 9-1 Frederick Street Catchment RRR Model Channel Lag Parameters

Sub-catchment	Pipe Flow Time (mins)	Mean Gutter Time (mins)	Total (mins)	K (hrs)
To ILSAX reach A16	10.98	12.08	23.06	0.0384
A16 to gauging Station	3.34	15.82	19.16	0.0319

The outflow from the upstream sub-catchment (to A16) was translated by 3.34 minutes to the Frederick Street gauging station.

The largest storms of 1992 and 1993 were then fitted with adjustments made only to the initial and continuing loss for the impervious and pervious contributing areas with the results given in Table 9-3. Figure 9-1 shows the result of the RRR modelling. Appendix 7 gives hydrograph plots for both measured and predicted flows, with one hydrograph example shown as Figure 9-2.

Table 9-2 Frederick Street RRR Model Calibrated Losses

Date	Initial Loss (Impervious) (mm)	Initial Loss (Pervious) (mm)	Proportional Loss (Pervious)
03/07/92	1.0	*	*
07/08/92	1.5	*	*
11/07/92	1.0	*	*
19/07/92	1.0	*	*
30/08/92	1.0	8.8	0.95
31/08/92	1.0	*	*
18/12/92	2.0	26.5	0.75
24/05/93	2.0	*	*
30/08/93	2.0	*	*
19/09/93	3.0	*	*
30/09/93	2.0	*	*
17/10/93	2.0	*	*

* indicates no pervious area contribution

Table 9-3 Frederick Street, Glenelg Catchment RRR Fits

Date	Runoff Volume			Peak Flow (m^3/s)		
	Predicted (m^3)	Recorded (m^3)	Predicted/ Recorded	Predicted (m^3/s)	Recorded (m^3/s)	Predicted/ Recorded
03/07/92	1262	1370	0.918	0.321	0.343	0.936
07/08/92	934	920	1.015	0.304	0.306	0.993
11/07/92	1022	980	1.046	0.152	0.128	1.188
19/07/92	720	780	0.922	0.276	0.316	0.873
30/08/92	3172	3460	0.917	1.000	1.078	0.928
31/08/92	580	620	0.940	0.330	0.394	0.838
18/12/92	6071	5360	1.133	0.971	1.241	0.782
24/05/93	982	762	1.289	0.265	0.322	0.823
30/08/93	1232	1163	1.059	0.618	0.534	1.157
19/09/93	992	970	1.023	0.574	0.652	0.880
30/09/93	534	644	0.829	0.200	0.312	0.641
17/10/93	977	989	0.988	0.529	0.548	0.965
		Mean	1.007			0.917
		ILSAX Mean	0.988			1.000
	Standard Deviation		0.120			0.151
	ILSAX Standard Deviation		0.114			0.110

The results are good, given the level of detail in the model compared with the ILSAX model. Both the mean runoff volumes and peak flows are within 10% of the measured means.

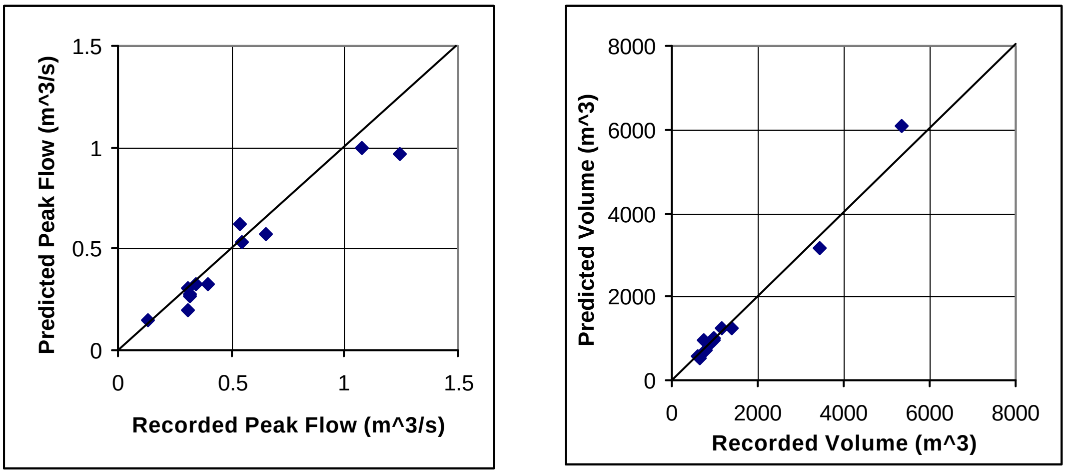


Figure 9-1 Glenelg Catchment RRR Results

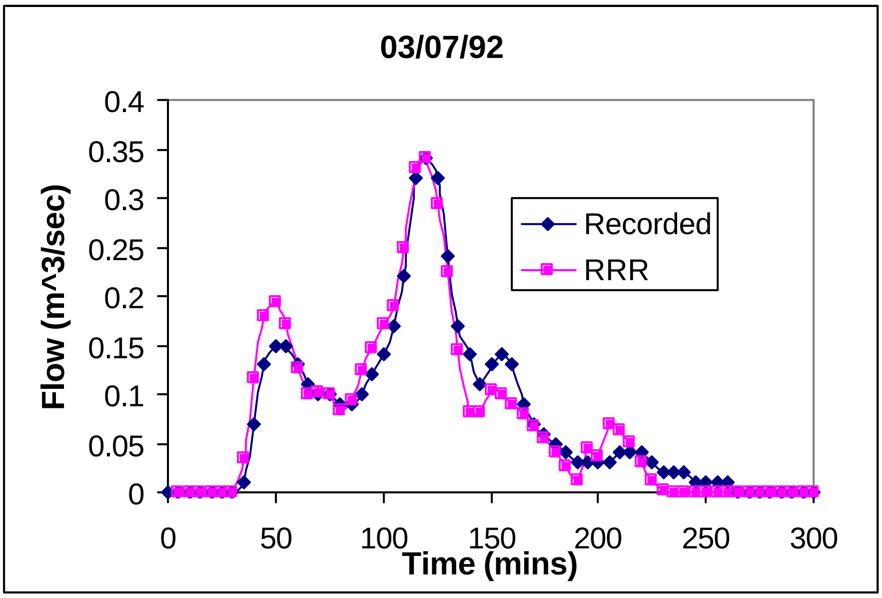


Figure 9-2 Glenelg Catchment RRR Fit 03/07/92

9.2.2 Paddocks Catchment

The RRR model was set up for the Paddocks catchment using the established formulae for the lag parameter k and with the directly connected impervious area being reduced by 10%, as was indicated in the ILSAX fitting to give the best fit to the recorded volumes. No pervious area runoff was included in the modelling.

It was found during the ILSAX modelling that no pervious area runoff was present. The application of the RRR model with the calculated lag parameters is then a direct verification of the model.

The RRR model consisted of two sub-catchments, to represent the rainfall from the two pluviometers within the catchment.
The channel lag parameters for the two sub-catchments were determined as given in Table 9-4.

Table 9-4 Paddocks Catchment RRR Channel Lag Parameters

Sub-catchment	Pipe Flow Time (mins)	Mean Gutter Time (mins)	Total (mins)	K (hrs)
To ILSAX reach A12	4.43	3.15	7.58	0.0126
A12 to gauging Station	4.80	3.93	8.73	0.0146

The outflow from the upstream sub-catchment (to A12) was translated by 4.80 minutes to the gauging station.

The summary of fitting results is given in Table 9-5. All storms showed reasonable fits, apart from the storm of 19/12/92. This storm was omitted from consideration as the peak flow predicted was in excess of the capacity of the pipe system. Figure 9-3 shows an example of the fit of the RRR model for most storms, and Figure 9-4 shows the fit achieved for the storm of 19/12/92. The capacity of the final pipe in the system is only 2.7m^3/s, and thus the measured peak flow is limited to this value. Hydrographs for the other storms fitted are plotted in Appendix 7.

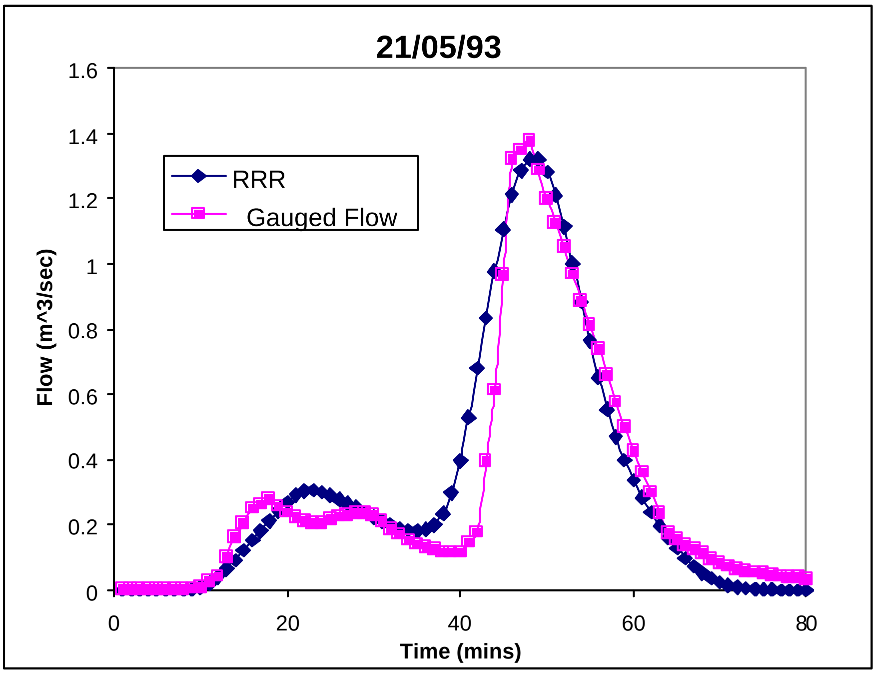


Figure 9-3 Paddocks Catchment - RRR Fit for Storm of 21/05/93

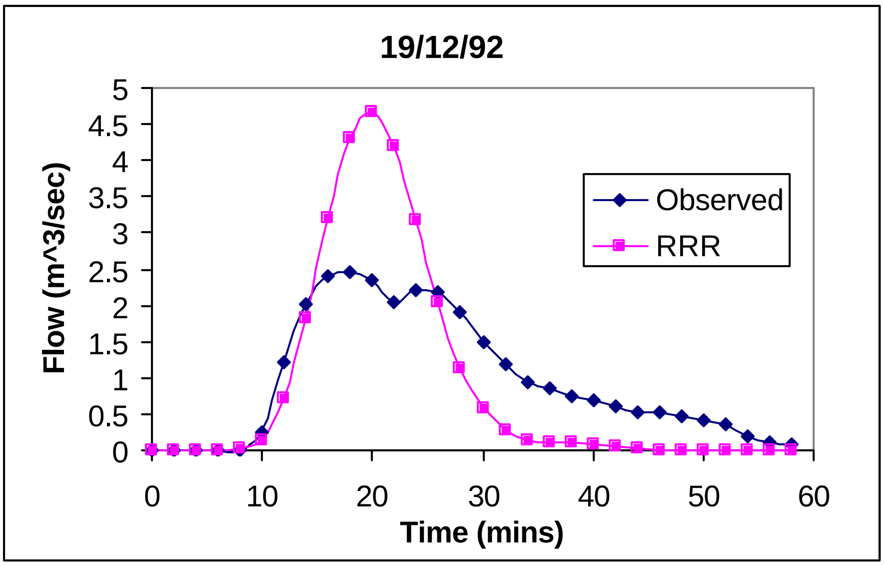


Figure 9-4 Paddocks Catchment - RRR Fit for Storm of 19/12/92 (Omitted)

Table 9-5 Paddocks Catchment RRR Fit Summary

Date	IL (impervious) (mm)	Runoff Volume (m^3)			Peak Flow (m^3/s)		
		Predicted (m^3)	Recorded (m^3)	P/R	Predicted (m^3/s)	Recorded (m^3/s)	P/R
03/10/92	1.5	875	955	0.916	1.089	1.407	0.774
08/10/92	0.0	1545	1574	0.982	0.723	0.960	0.753
08/10/92	0.0	2178	2275	0.957	1.041	1.286	0.809
17/11/92	2.5	2026	2316	0.875	1.874	2.230	0.840
21/11/92	0.0	978	984	0.994	0.660	0.771	0.856
18/12/92	2.0	1001	1124	0.891	1.088	1.453	0.749
27/02/93	1.0	1290	1395	0.925	0.716	0.860	0.833
21/05/93	1.0	1543	1448	1.066	1.322	1.378	0.959
03/06/93	1.0	1654	1632	1.013	0.956	1.144	0.836
11/06/93	1.0	571	648	0.881	0.625	0.943	0.663
30/08/93	1.0	1831	1793	1.021	1.287	1.391	0.925
17/10/93	1.0	1158	629	1.841	0.884	1.048	0.844
18/10/93	1.0	1019	802	1.271	0.956	1.054	0.907
13/12/93	1.0	1633	1379	1.184	1.906	1.670	1.141
			Mean	1.058			0.849
			ILSAX Mean	1.047			0.885
		Standard Deviation		0.252			0.114
		ILSAX Standard Deviation		0.146			0.111

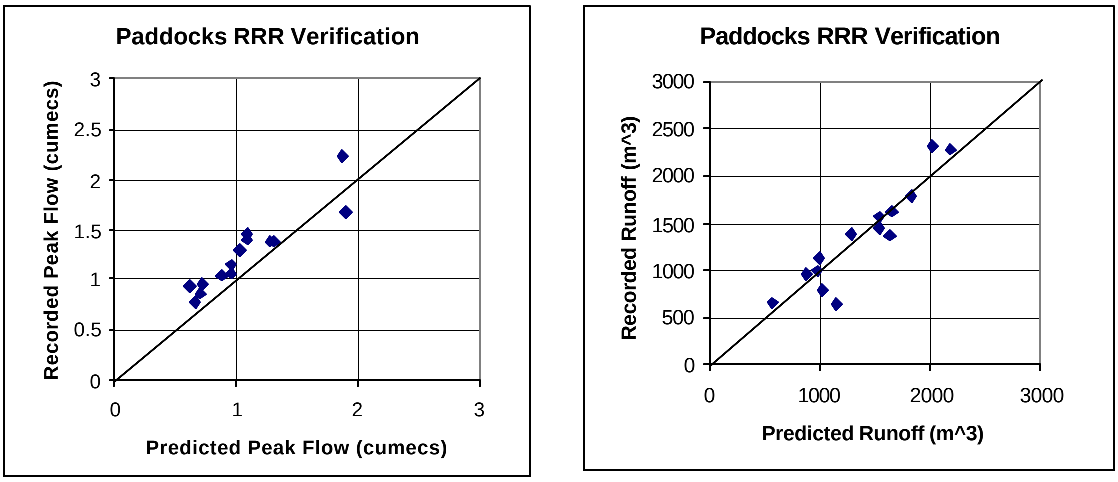


Figure 9-5 Paddocks Catchment RRR Results

Figure 9-5 shows the results of the modelling. The overall level of fit is similar to that of the ILSAX model, with the mean predicted ratios for peak flow and volume being similar to ILSAX.

Given that the RRR model is much simpler to set up, and contains therefore a much simpler representation on the catchment the performance is considered to be satisfactory.

9.2.3 Jamison Park

The Jamison Park catchment is one of three catchments at Penrith, a western suburb of Sydney, gauged by the University of Technology, Sydney (UTS). It has an area of 22.1ha. It is mainly residential land with some parkland, and was developed in 1970 -1975.

The station was established in January 1983.

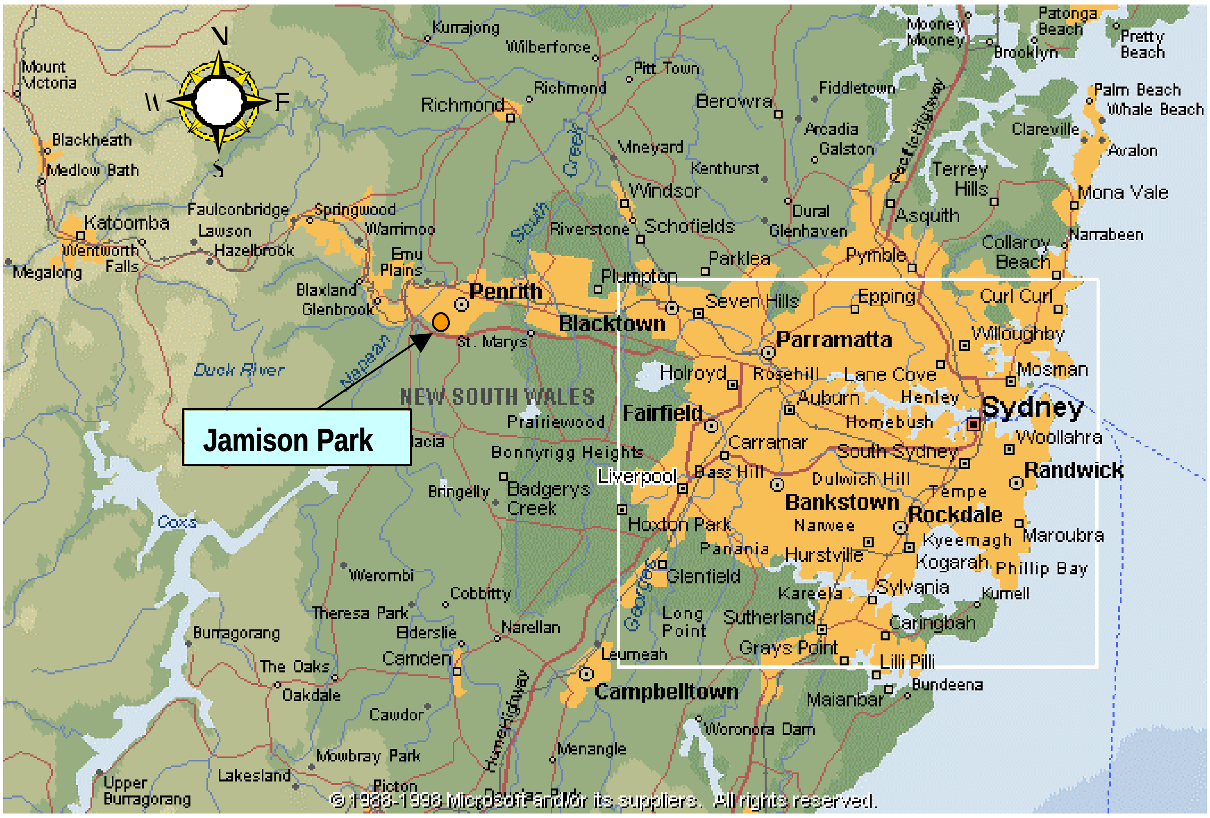


Figure 9-6 Location of the Jamison Park Catchment

Figure 9-7 View of the Jamison Park Catchment

It was decided to set up the RRR model for those storms considered by Haig (1989) for special consideration when calibrating the ILSAX model on the catchment. The storms were selected to be a representative sample of events recorded. The events cover a wide range of events of different magnitude, duration and antecedent moisture condition. Haig's results are summarised in Table 9-6:

The RRR model was designated to have the same directly connected impervious area as the ILSAX model and the storage parameters derived from the formulae. The pipe $l/s^{0.5}$ was determined from the ILSAX pipe file. As the gutter $l/s^{0.5}$ was not available in the ILSAX file, the lag time was determined from the mean gutter flow time in the ILSAX model.

The channel lag parameter k was determined from the pipe flow time (4.81 minutes) and the mean gutter flow time (2.9 minutes).

Calibration consisted of adjusting the initial loss for the impervious area and the initial and continuing losses for the pervious area contribution. Calibration was then on the losses only. Each event was first run with a pervious loss high enough to prevent any pervious area runoff. The initial loss for the pervious area was then set to provide pervious area contribution at the time when the modelled and recorded hydrographs diverged. It was found by trial and error that a proportional initial loss provided the better fit to the recorded hydrograph.

Table 9-6 Jamison Park ILSAX Fit Summary

Date	Runoff			Flow		
	Predicted (m^3)	Recorded (m^3)	Predicted/ Recorded	Predicted (m^3/s)	Recorded (m^3/s)	Predicted/ Recorded
04/03/89	400	403	0.993	0.325	0.454	0.716
24/08/88	1560	1450	1.062	0.250	0.251	0.996
02/04/85	3620	2090	1.732	2.472	1.270	1.946
23/01/88	557	641	0.869	0.274	0.319	0.859
27/07/84	7290	16700	0.435	0.861	1.544	0.558
05/07/88	15400	24800	0.622	0.499	0.868	0.575
01/01/88	1120	1240	0.902	0.792	1.139	0.695
14/03/89	1730	1440	1.197	0.157	0.170	0.924
09/10/86	2230	3270	0.682	0.151	0.188	0.803
07/11/84	4490	3370	1.335	2.527	1.399	1.0806
31/03/89	3630	4130	0.879	0.798	0.548	1.456
21/03/83	6430	7700	0.824	1.060	1.023	1.036
		Mean	0.961			1.031
	Standard Deviation		0.330			0.443

The calibrated losses are given in Table 9-7.

Table 9-7 Jamison Park RRR Loss Model Calibration

Date	AMC	Impervious		Pervious	
		Initial loss (mm)	Continuing loss (mm/hr)	Initial loss (mm)	Proportional loss
04/03/89	1	1	0	*	*
28/04/88	1	2	0	*	*
02/04/85	1	6	0	*	*
23/01/88	2	2	0	*	*
27/07/84	2	0	0	0	0.1
05/07/88	2	0	0	30	0.2
01/01/88	3	0	0	*	*
14/03/89	3	1	0	*	*
09/10/86	3	0	0	*	*
07/11/84	4	2	0	22	0.2
31/03/89	4	0	0	10	0.35
21/03/83	4	0	0	10	0.4

* indicates that there was no pervious area contribution.

The AMCs used are as per the ILSAX model. It is interesting that the proportional loss rises with an increase in antecedent moisture, perhaps because the pervious area behaves differently when very wet. The results are given in Table 9-8:

Table 9-8 Jamison Park RRR Fit Summary

Date	Runoff Volume			Peak Flow		
	Predicted (m^3)	Recorded (m^3)	Predicted/ Recorded	Predicted (m^3/s)	Recorded (m^3/s)	Predicted/ Recorded
04/03/89	400	403	0.990	0.420	0.454	0.925
24/08/88	1495	1450	1.020	0.255	0.251	1.016
02/04/85	2891	2090	1.383	2.405	1.270	1.894
23/01/88	465	641	0.725	0.289	0.319	0.906
27/07/84	16466	16700	0.983	1.180	1.544	0.764
05/07/88	26512	24800	1.067	0.743	0.868	0.856
01/01/88	1158	1240	0.935	0.958	1.139	0.841
14/03/89	1738	1440	1.203	0.186	0.170	1.094
09/10/86	2247	3270	0.688	0.191	0.188	1.016
07/11/84	2704	3370	0.804	1.411	1.399	1.009
31/03/89	4628	4130	1.120	0.584	0.548	1.066
21/03/83	7722	7700	1.003	0.959	1.023	0.937
		Mean	0.993			1.027
	Standard Deviation		0.196			0.290

The results show that the fit achieved by the RRR model is better than the ILSAX model. The difference is mainly due to the prediction of losses from pervious areas. The ILSAX model used a constant soil type and an AMC calculated for each event. The RRR model pervious area losses were fitted for each event individually. A better fit would therefore be expected.

As an alternative to the above approach, which is really a calibration it was decided to derive a loss model to apply to all the events to determine if a reasonable level of fit could be achieved.

The proposed loss model is shown in Table 9-9. This model is based on the calibrated losses from the fitted storms, and uses the same AMC classifications as the ILSAX model.

Table 9-9 Jamison Park Derived Loss Model

AMC	Rainfall in 5 days preceding storm (mm)	pervious IL (mm)	pervious proportional loss
1	0	100	0.3
2	0 - 12.5	60	0.3
3	12.5 - 25	35	0.3
4	over 25	15	0.3

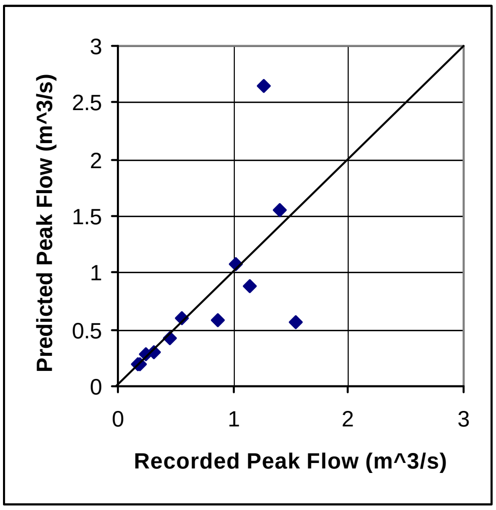
The initial loss in all cases on the impervious area is taken as 1mm.

Table 9-10 shows the result of the modelling.

Table 9-10 Jamison Park RRR Fit Summary With Derived Loss Model

Date	Runoff Volume			Peak Flow		
	Predicted (m^3)	Recorded (m^3)	Predicted/ Recorded	Predicted (m^3/s)	Recorded (m^3/s)	Predicted/ Recorded
4/03/89	399	403	0.990	0.406	0.454	0.894
28/04/88	1554	1470	1.060	0.266	0.251	1.060
2/04/85	3162	2090	1.513	2.401	1.27	1.891
23/01/88	549	641	0.856	0.289	0.319	0.906
27/07/84	7734	16700	0.462	0.499	1.544	0.323
5/07/88	24043	24800	0.968	0.704	0.868	0.811
1/01/88	1056	1240	0.852	0.854	1.139	0.750
14/03/89	1738	1450	1.2030	0.186	0.17	1.094
9/10/86	2171	3270	0.664	0.191	0.188	1.016
7/11/84	3416	3370	1.015	1.509	1.399	1.079
31/03/89	4230	4130	1.024	0.585	0.548	1.068
21/03/83	7795	7700	1.012	1.023	1.023	1.000
		mean	0.968			0.991
	Standard Deviation		0.260			0.356

As expected this is not as good a fit as the calibration runs, but it still represents a reasonable level of fit, equal to that of the ILSAX model as given in Table 9-6, but with variation being lower. The fitting results are shown on Figure 9-8, and one of the events on Figure 9-9.



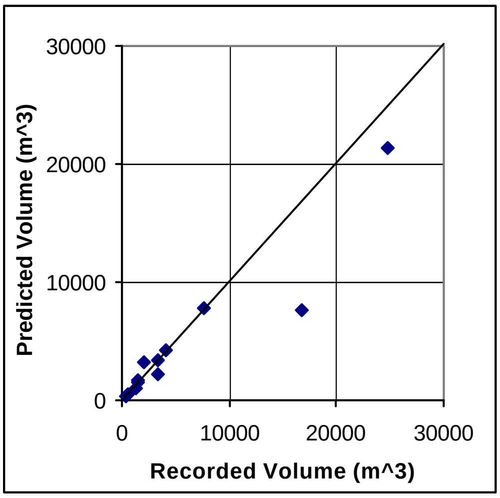


Figure 9-8 Jamison Park RRR Results

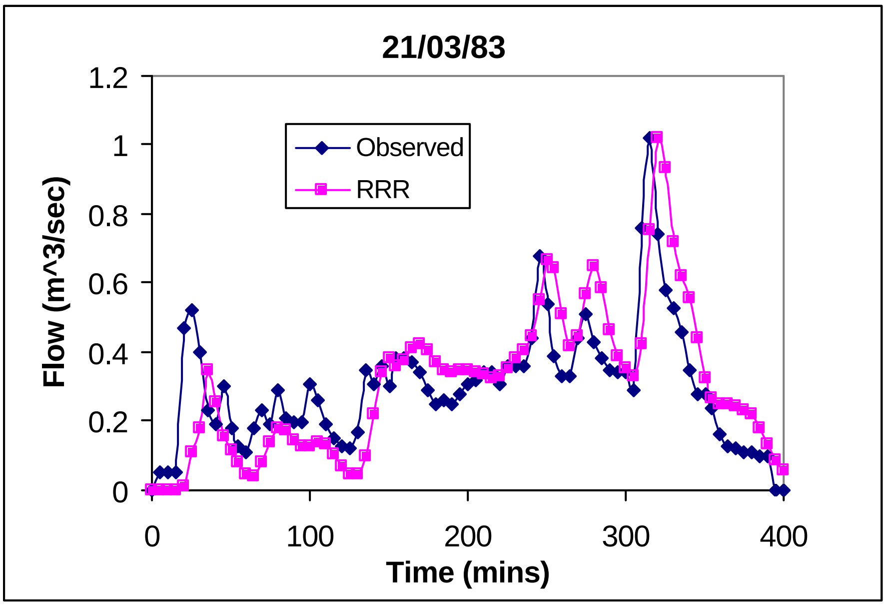


Figure 9-9 Jamison Park RRR Fit 21/03/83

A comparison can be made between ILSAX and the RRR model (with the derived loss model) by plotting the ratios of predicted and recorded peak flows for each storm event. This is shown on Figure 9-10, which shows a high degree of correlation between the performance of both models on the same event. Both models provide similar transformations of rainfall into flow at the outlet. Error in the prediction of flows may be due to the inadequacy of the data, rather than the inadequacy of the model itself in providing the transformation.

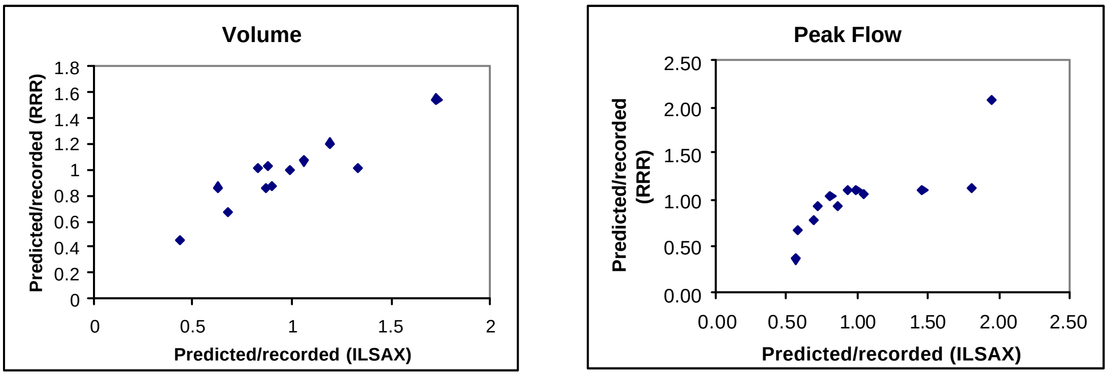


Figure 9-10 Comparison of ILSAX and RRR on Jamison Park Catchment

9.2.4 Summary - Urban Catchments

The performance of the RRR model with the derived storage parameters has been determined for three gauged catchments. The model performed in a comparable fashion to the well known and widely used ILSAX model, which gives confidence in the use of the RRR model on urban catchments for design purposes.

9.3 Rural Catchments

The RRR model has been used to model flows from two rural catchments in Chapter 8. However the model is now applied to a wide range of rural catchments to verify that the model appropriately models catchment behaviour, and to determine the likely ranges of the storage parameters (v_c and c_p). These parameters need to be estimated for ungauged catchments for the model to be used for design purposes.

9.3.1 Catchment Selection

The application of the RRR model to rural catchments was verified by selecting a number of rural catchments, calibrating the model to determine mean parameters and then applying the calibrated model to a number of independent events to verify the model.

Catchments were initially selected within South Australia as this was the focus of the research. The chosen catchments had concurrent flow and rainfall data available. For simplicity the RRR model was verified where possible as a single sub-catchment, with uniform rainfall input. This limited the size of the catchment, as the applicability of the single rainfall input can be expected to diminish with increasing catchment area. Following examination of available streamflow stations four stations were selected in the Mount Lofty Ranges, all with a catchment area less than 40km². It was not possible to find small catchments having several rainfall stations and a long enough period of record to have sufficient events for calibration and verification.

Two catchments outside South Australia were selected for verification. These catchments were chosen to be in different climatic regions, but ideally were small catchments (less than 40 km²) and having a good pluviometer record. In the end the criteria of catchment area could not be met, but two catchments were selected to be of different climatic conditions. The two chosen were the Burra Creek catchment, close to the ACT, but situated in New South Wales, and the Celia Creek catchment in the Northern Territory. The catchment locations are shown on the following figures.

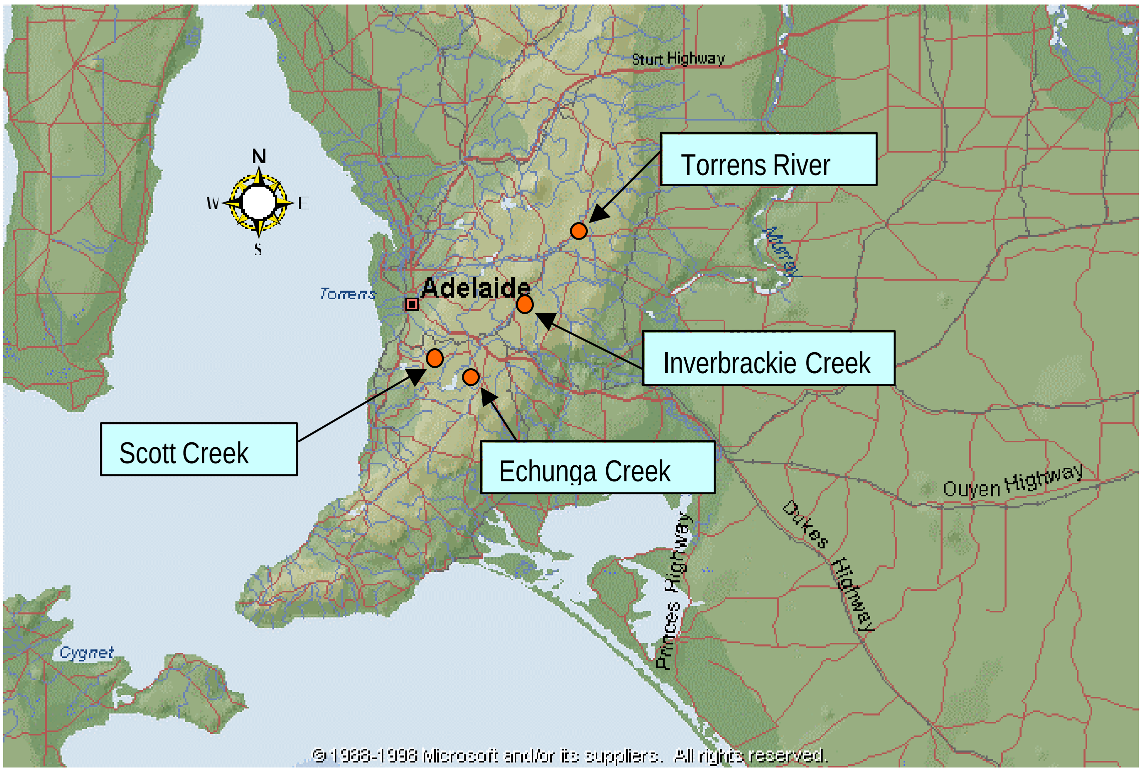


Figure 9-11 Mount Lofty Ranges Catchments Locations

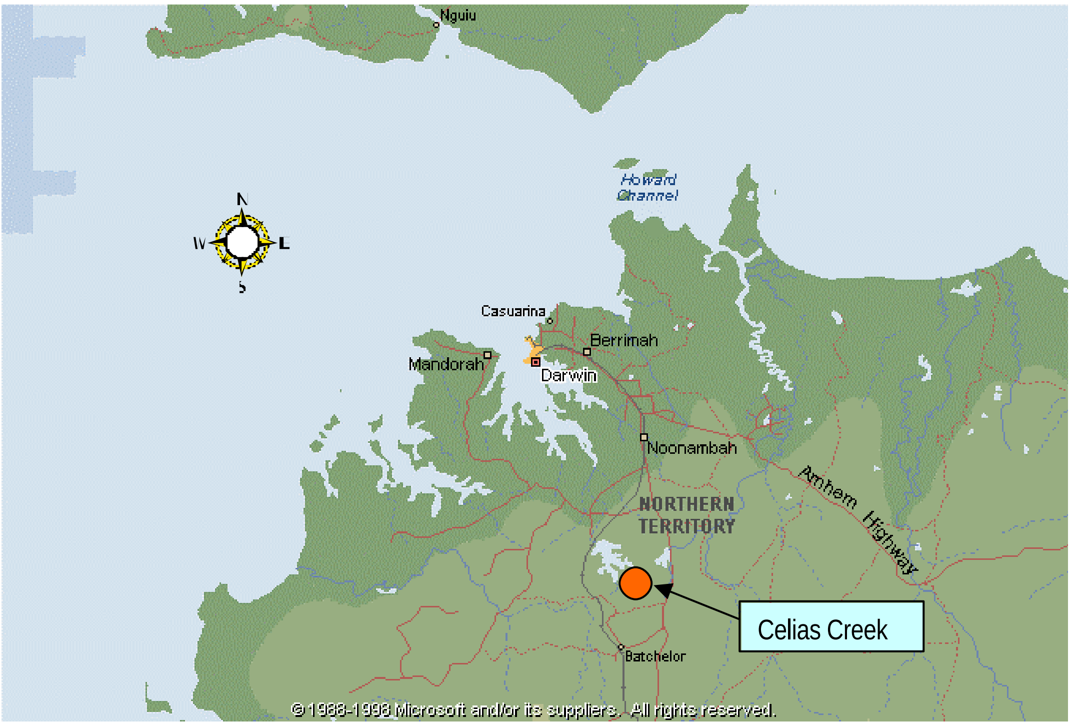


Figure 9-12 Celia Creek Catchment Location

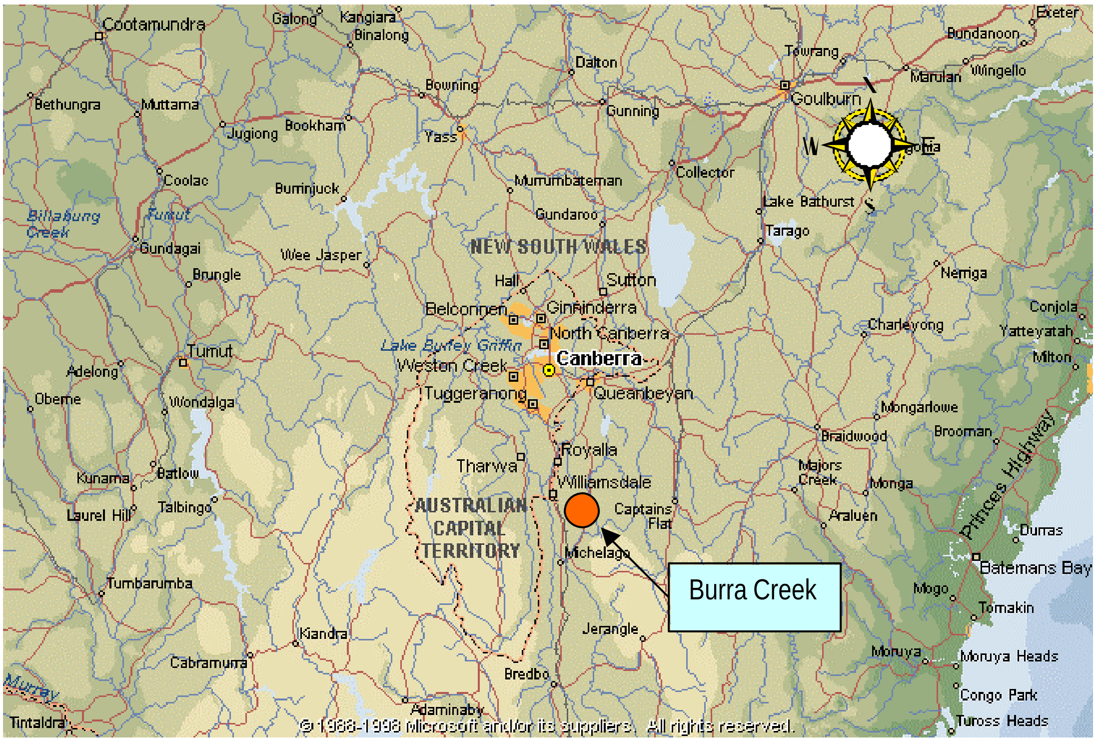


Figure 9-13 Burra Creek Catchment Location

9.3.2 Calibration and Verification Strategy

The parameter estimation program PEST (Watermark Computing, 1996) was selected for the calibration of the models. The advantages of the use of PEST are twofold. Firstly the estimation would be objective, minimising an objective function that measures the level of fit of the calculated hydrograph. Secondly, the overall time taken to undertake calibration was minimised by the use of the automated procedure used by PEST.

The recommended criteria for calibrating runoff routing models are given in Australian Rainfall and Runoff as follows:

"The overall hydrograph shape of the hydrograph is important where the effects of storage need to be considered, such as the design of a spillway. Conversely, only the peak is important where storage has little or no effect, such as the design of a bridge waterway. It should be noted that use of different criteria will usually lead to different derived parameter values."

Dyer (1994) derived an objective function for the automatic calibration of the RORB model. The objective function was:

$$OF = \frac{\sum_{t=1}^{t_{end}} \left| \frac{\{Q_o(t) - Q_c(t)\}}{Q_{op}} \right|^2}{\sum_{t=1}^{t_{end}} \frac{Q_o(t)}{Q_{op}}} \qquad \text{Equation 9.1}$$

Where Q_o is the observed flow at time t

Q_c is the calculated flow at time t

Q_{op} is the observed peak flow

This expression can be simplified however to:

$$OF = \sum_{t=1}^{t_{end}} \left| \frac{Q_o(t) - Q_c(t)}{Q_o(t)} \right|^2 \qquad \text{Equation 9.2}$$

However, this gives no emphasis to the peak flow. Ibbitt (1991) provides a review of the use of objective functions to calibrate hydrological models. He points out that it is known that the differences between the observed and calculated flows (the residuals) are "heteroscedastic", or of unequal variance. In other words there are likely to be larger differences between the calculated and measured flood peaks than between the corresponding flows low down on a hydrograph recession.

Thus if a simple least squares objective function is used there is automatically emphasis given to flood peaks, and for this reason an objective function such as the logarithm of the residuals has been suggested.

Chapman (1970), Pilgrim and Bloomfield (1980) and Sefe and Broughton (1982) examined a range of criteria based on different combinations of measured and calculated flows and/or their residuals, raised to a variety of different powers. It was concluded that none of the criteria offered significant improvements in fitting performance when compared with the use of the sum of the squares criterion.

It was considered given the above that a straight least squares criterion be used. If this gave more than reasonable emphasis to peak flows the objective function would be reviewed.

For the purpose of calibrating the RRR model the observed flows during each event were given equal weighting, for the same reason. No further weighting was given to peak flows. The RRR model models the whole hydrograph including baseflow. If the calibration was more biased to the peak flow it is probable that the calibration of the baseflow component would be substantially less reliable than if equal emphasis was given to all components.

For each catchment selected for verification a minimum of twelve events were selected from the period of record. This was done so that the model could be calibrated on six events and then verified on the remaining six events.

Data for 5-7 days duration was obtained for each event to be modelled such that a start time could be selected 6-12 hours before the start of the rise of the hydrograph. The duration of the event was selected such that in most cases flows were returning to what would normally considered to be baseflow. Figure 9-14 shows the hydrograph of a typical event used for the modelling, in this case Burra Creek.

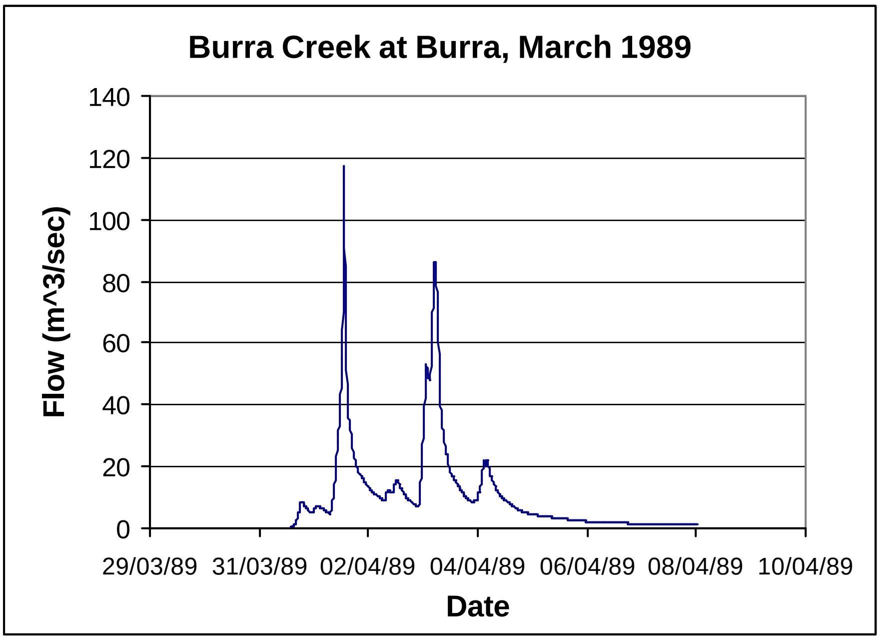


Figure 9-14 Typical Hydrograph Data Obtained for Each Storm Event

Two separate sets of parameters were calibrated with each of the catchments, the first being the model storage parameters. In the case of RRR these are the process storage parameters and the channel storage parameter. In each catchment it was assumed in the calibration that three processes were potentially present for each event, based on the analysis of probable processes and the preliminary fit runs of RRR on the Aldgate and Kanyaka Creek catchments. The processes are labelled 1, 2 and 3 for simplicity in the RRR verification.

- Process 1 is assumed to be what is traditionally known as baseflow.
- Process 2 is termed slow flow; and
- Process 3 is fast flow. For process 3 it is assumed in the calibration that the process storage parameter is zero.

If the RRR model is valid it was expected that the storage parameters would be stable. That is it would be found that for each event calibrated the same processes are present and the values of each storage parameter would be similar. Some variation is expected due to non-uniformity of catchment rainfall and the possible different responses of the catchment with initial catchment conditions. It was possible that the catchment response would change depending on whether the catchment is in a wet or dry antecedent condition.

The second set of parameters is the loss parameters. There is an initial and proportional or continuing loss for each of the modelled processes. Based on the initial model fits it was assumed that a proportional loss would occur for each of the processes. A continuing loss was applied to process 2 and the level of fit tested the use of this continuing loss. If this provided a better fit the use of a continuing loss with process 1 would then also be investigated.

The initial loss for process 1 (baseflow) was assumed to be zero if there was flow present at the commencement of the modelled event.

Unfortunately, both the initial and proportional or continuing loss are much more susceptible to the variability inherent in a catchment than are the storage parameters. The initial loss is defined as the loss that occurs between the start of the storm event and the start of the runoff that is modelled. However the definition of the start of the storm event is open to debate, particularly in the case of RRR that seeks to model all contributions to the event hydrograph.

The initial loss for process 2 is clearly dependent on the time selected for the start of modelling.

The calibrated proportional losses during the event are subject to noise due to the rainfall distribution across the catchment. A single pluviometer record defined each catchment rainfall modelled, apart from Celia Creek. Averaged across a large number of events it would be expected that the mean catchment rainfall would be represented by the single pluviometer, unless there is a strong rainfall gradient across the catchment. The mean proportional loss would then be representative of catchment behaviour with a known rainfall input.

However during each rainfall event that is modelled the mean catchment rainfall may not be well represented by a single pluviometer. The loss parameters will be found during calibration, but these may not be representative of catchment behaviour with a known rainfall input. If the pluviometer rainfall is less than the true mean catchment rainfall the calibrated proportional loss will be lower than the true value, and the reverse is also true.

This leads to problems during the verification runs. Clearly the pluviometer record for the verification run may also not be representative of mean catchment rainfall. It would be expected then that even if the true mean proportional losses are known, the verification event may not model the volume or peak of the hydrograph well.

It was determined therefore that emphasis must be given to the parameters in the model that are not subject to the effect of the limited knowledge of the true mean catchment rainfall. The model was considered to be verified if factors such as the time to peak and the shape of the hydrograph were well predicted. If the mean ratio of modelled/actual peak flow and volume is close to unity then the loss parameters can also be considered to be verified, but there may be large differences in individual verification events.

The above problems with the loss parameters also tends to hide relationships that may exist between such things as event peak flow and proportional loss.

One well known measure of catchment condition is the Antecedent Moisture Condition (AMC) which is represented by the Antecedent Precipitation Index (API). The API is based on daily rainfall data, where the API for each day is simply a proportion of the API of the previous day plus the daily rainfall.

The API is defined by Nordenson and Richards (1964) as;

$$API_0 = P_0 + P_1K + P_2K^2 + ... \; ..... + P_nK^n \qquad \text{Equation 9.3}$$

Where K = a recession factor less than unity

P_n = daily rainfall n days antecedent to the storm event

The factor K is usually taken as 0.9.

However the API may not be the most reasonable measure of catchment condition. Siriwardena et al (1997) investigated the use of a variable proportional loss model for use in flood estimation. Their conclusion was that pre-storm baseflow is a convenient and robust measure of antecedent wetness that can be incorporated in a loss model to model the catchment response to rainfall.

For this reason it was decided to investigate the relationship of calibrated parameters to both the initial baseflow (ie. at the time of start of the simulation) and the API for the event.

For South Australian catchments there is also a marked difference between summer and winter rainfall and temperature, leading to a difference in vegetation and soil moisture condition. It is perhaps difficult to find a single variable that will account for these differences, but likely variables are monthly mean rainfall or temperature.

The above factors were considered during validation, by determining the relationships between parameter values and both API and initial baseflow. A preliminary investigation showed that there were insufficient events calibrated representing different vegetation and soil moisture conditions to investigate any relationship with monthly rainfall or temperature.

Once each of the selected storms had been calibrated using PEST mean parameter values were chosen for the verification runs. It is necessary to weight the calibrated parameters for each calibration event by a measure of the level of fit achieved in the calibration run (a weighting factor). A parameter calibrated from an event having a very good fit should be given more weight than one from an event that does not provide a good fit, as there is more confidence in the parameter value.

A mean error of the estimate is defined as:

$$Mean\ Error = \sqrt{\frac{PHI}{n}} \qquad \text{Equation 9.4}$$

where n is the number of observations, or hydrograph ordinates

PHI Is the objective function used by PEST, being the sum of the squares of the differences between the observed and predicted ordinates at each time step

The mean error of estimate will not however provide a good measure of the overall fit that can be used for the weighting of calibrated values. The calibrated events are of varying magnitude, and account must be taken of this. Higher emphasis must be given to a good fit to an event having a higher peak flow. The weighting factor chosen was the observed peak flow divided by the mean error of estimate.

The weighting factor used is:

$$WF = \frac{Observed \quad peak \quad flow}{Mean \quad error} = \left(\frac{n}{\sum_{1}^{n} (q_o - q_c)^2} \right)^{0.5} x \ Q_{op}$$ Equation 9.5

where q_o is the observed flow at each time step

q_c is the modelled flow at the time step

n is the number of time steps or observations

Q_{op} is the observed peak flow

Further details of the calibration and verification for each selected catchment is given in Appendix 8.

It is a matter of conjecture whether a mean value of initial loss should be used for verification, if no relationship is found between initial loss and catchment condition. Kuczera (pers. comm.) considers that initial loss is usually quite arbitrary depending on when you start the hyetograph in the simulation, and because of this arbitrary nature it should be a fitted parameter during verification. However it was considered that for this study mean values would be used. This represents the application of the model in a real situation where the model will be applied to predict a hydrograph. There will usually be no prior information on the runoff hydrograph.

Summarising the process of calibration and verification then, for each catchment;

- A minimum of twelve storm events were chosen from the period of record, and approximately half chosen for calibration.
- Each calibration event was applied to the RRR model using the PEST parameter estimation program. A least squares objective function is used to determine the level of fit.
- Once parameter values are determined for each calibration event, mean parameter values were determined using a weighting factor incorporating the level of fit and the absolute gauged peak flow for each event, unless a relationship was found relating parameter values to the physical catchment condition, as represented by the API or initial baseflow for each event.

- To each of the remaining selected events the RRR model was applied using either the mean parameter values or the derived relationship.
- The success of the verification was determined by the mean error in predicted peak flow and runoff volume.

9.3.3 The Effect of Data Inaccuracy

The data that is used in the verification of the RRR model is subject to inaccuracy. This arises for two reasons. First there is the inherent error in the data at site. Rainfall data is influenced by the location of the instrument and if the instrument is calibrated correctly. Flow data is subject to greater error, as the recorded information is flow depth, which must be converted to flow by the application of a rating table. The rating table is based on measured flows (gaugings), but is assumed to be stable over time, and is often extrapolated past the range of measured flows.

Second there is the issue of rainfall variability over the catchment. In the calibration and verification rainfall data from a single site is used as input to the model. This may not be representative of average catchment rainfall, depending on the rainfall distribution across the catchment both in space and in time.

The hydrological model provides a transformation of the input data to the output information, which is then compared with recorded information. To verify that the model is providing a robust transformation the effect of the inaccuracy in both the input data (rainfall) and recorded information (flows) must be considered.

Errors in rainfall data and rainfall variability across the catchment will mainly affect calibrated losses, but may also affect the apparent catchment lag. Dyer (1994) acknowledged that timing errors might exist in the rainfall data, either due to gross timing errors in the data, or by the input data being unrepresentative of catchment rainfall. The timing errors affect any objective function that compares the ordinates of observed and calculated hydrographs without due allowance. The procedure adopted by Dyer was to first translate the calculated hydrographs until the hydrograph centroids are aligned. The calculated hydrograph was then translated backwards and forwards in time to obtain the lowest value of the objective function. If the resultant time translation is unacceptable the modeller can either discard the event, or check the data for errors.

The procedure adopted was to first undertake a trial calibration of the RRR model for each event. If the shape of the hydrograph could only be matched if there was a time translation then it was assumed that there was a data error (either at site or due to variability). A time translation was introduced into the model as an extra parameter that was adjusted with all other parameters to minimise the objective function.

Of the catchments chosen for the verification of the RRR model only one had more than one pluviometer within the catchment. This catchment was Celia Creek. The other catchments had only one pluviometer, and it was assumed that this pluviometer was representative of catchment rainfall. The only other information available in some of the catchments was a daily read rainfall gauge. It was assumed that pluviometer data was accurate unless there was an indication that catchment runoff was getting close to or exceeding rainfall. In this case there are problems either with the rainfall data, or the rating curve. The pluviometer rainfall was compared with the daily read station to try and determine the cause of the error.

Data error may also have an effect on the level of fit achieved during the verification events with mean parameter values determined by calibration. In this case rainfall errors may have a major effect on the level of fit, particularly with regard to runoff volume and the peak flow. There will be a lesser effect on the shape and timing of the hydrograph.

Errors in the rating curve will have a lesser effect on the calibration and verification of the model, as the same consistent error will be present in all cases. However errors in the rating curve will affect the calibrated losses, and thus any comparison of losses across different catchments.

9.3.4 Torrens River at Mount Pleasant

The catchment of the Torrens River at Mount Pleasant lies within the Mount Lofty Ranges approximately 50km from Adelaide. The land use is predominantly grazing. Other physical data are as follows:

Area	26 km^2
Main Stream Length	9.0 km
S_e (equal area slope)	7.0 m/km
Average Annual Rainfall	677mm (at Mt. Pleasant township)

The catchment is served by a gauging station and pluviometer maintained by the Department of Environment, Heritage and Aboriginal Affairs (AW504512). The station is located at a stable natural rock bar, with minor concrete work. The pluviometer is located at the site of the gauging station. Data were obtained for both stream flow and rainfall from 1989 to 1997. The catchment is shown on Figure 9-16.

Figure 9-15 View of the Torrens Catchment

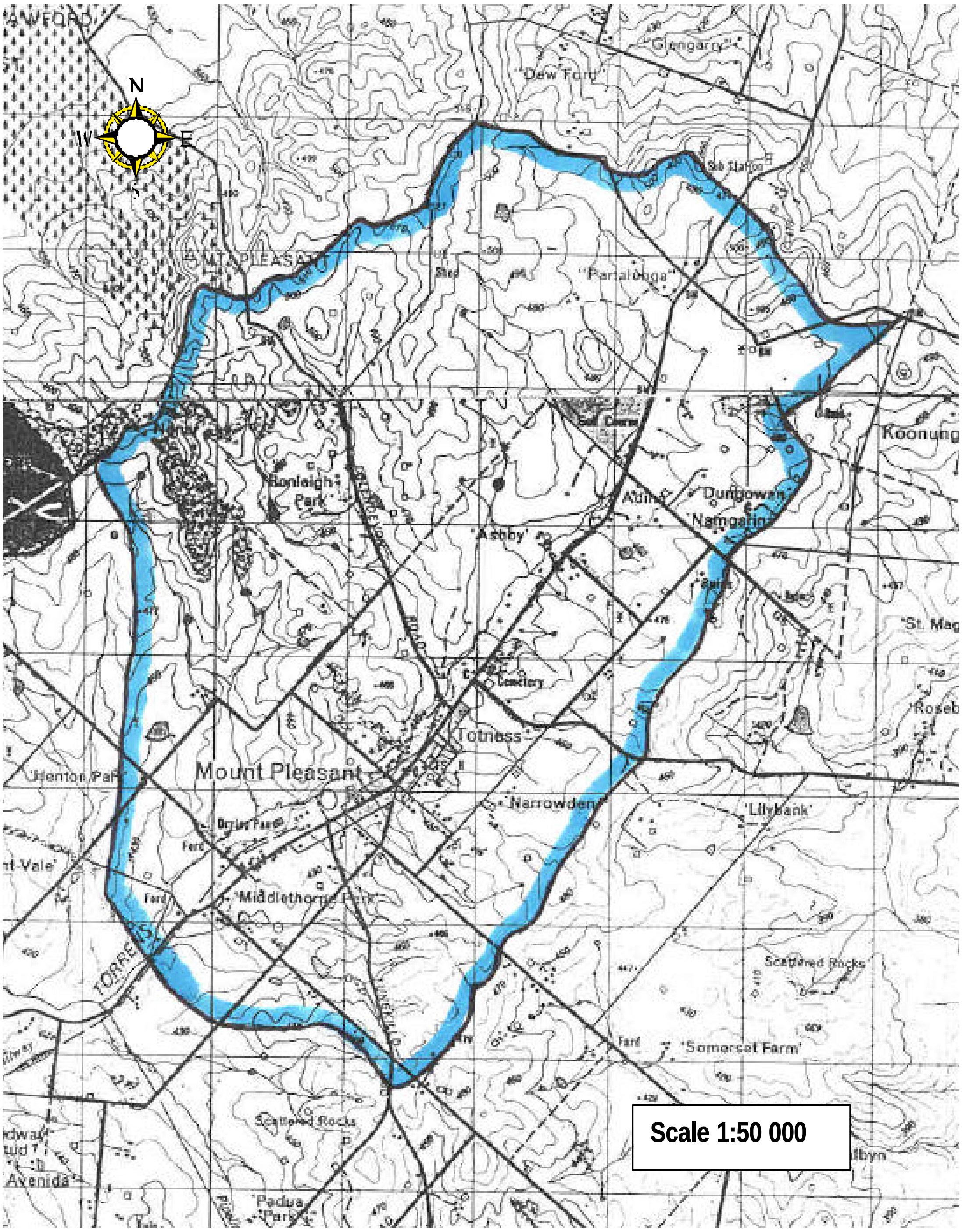


Figure 9-16 River Torrens Catchment

For the purposes of verification of the RRR model data were obtained for the periods of approximately one week containing the 12 largest flows. For each period the rainfall and stream flow data were in 15 minute time steps.

Six events from this data set were selected for calibration. The events selected included the event having the largest peak flow (29/08/92) and five other events selected at random. The largest event has an ARI of approximately 50 years, and all the rest are less than 10 years. Chapter 10 contains details of the flood frequency analysis.

It was assumed in the calibration that 3 runoff processes were occurring, being base, slow and fast flow. The PEST optimisation determined whether all processes were occurring. Table 9-11 shows the calibrated parameter values. In five events base and slow flow were found. In the other (29/08/92) slow and fast flow were found. This event had the highest peak flow.

Table 9-11 River Torrens Catchment RRR Calibrated Parameter Values

Event date	PL1	IL2 (mm)	PL2	IL3 (mm)	PL3	k_p1	k_p2	k (hrs)	MEAN ERROR (m^3/s)
30/07/89	0.85	6.50	0.12	*	*	0.75	0.29	0.221	0.51
29/08/92	*	15.0	0.12	52.4	0.25	*	0.39	0.268	3.20
23/09/92	0.83	6.36	0.37	*	*	2.68	0.48	0.262	0.33
18/07/96	0.81	14.4	0.58	*	*	1.07	0.36	0.087	0.86
03/08/96	0.55	4.48	0.38	*	*	1.54	0.41	0.384	0.97
28/09/96	0.78	14.6	0.25	*	*	0.61	0.49	0.287	0.92

The calibrated hydrographs are shown in Figure 9-17.It can be seen that a good fit could be achieved for all events. It was found by testing both loss models that the use of a proportional loss rather than a continuing loss gave a better fit to the recorded hydrograph.

The verification was carried out on six events, with the weighted mean values for the parameters as determined in the calibration. A relationship was found between the IL2 and the initial baseflow occurring at the start of the storm.

The parameters used for verification are given in Table 9-12

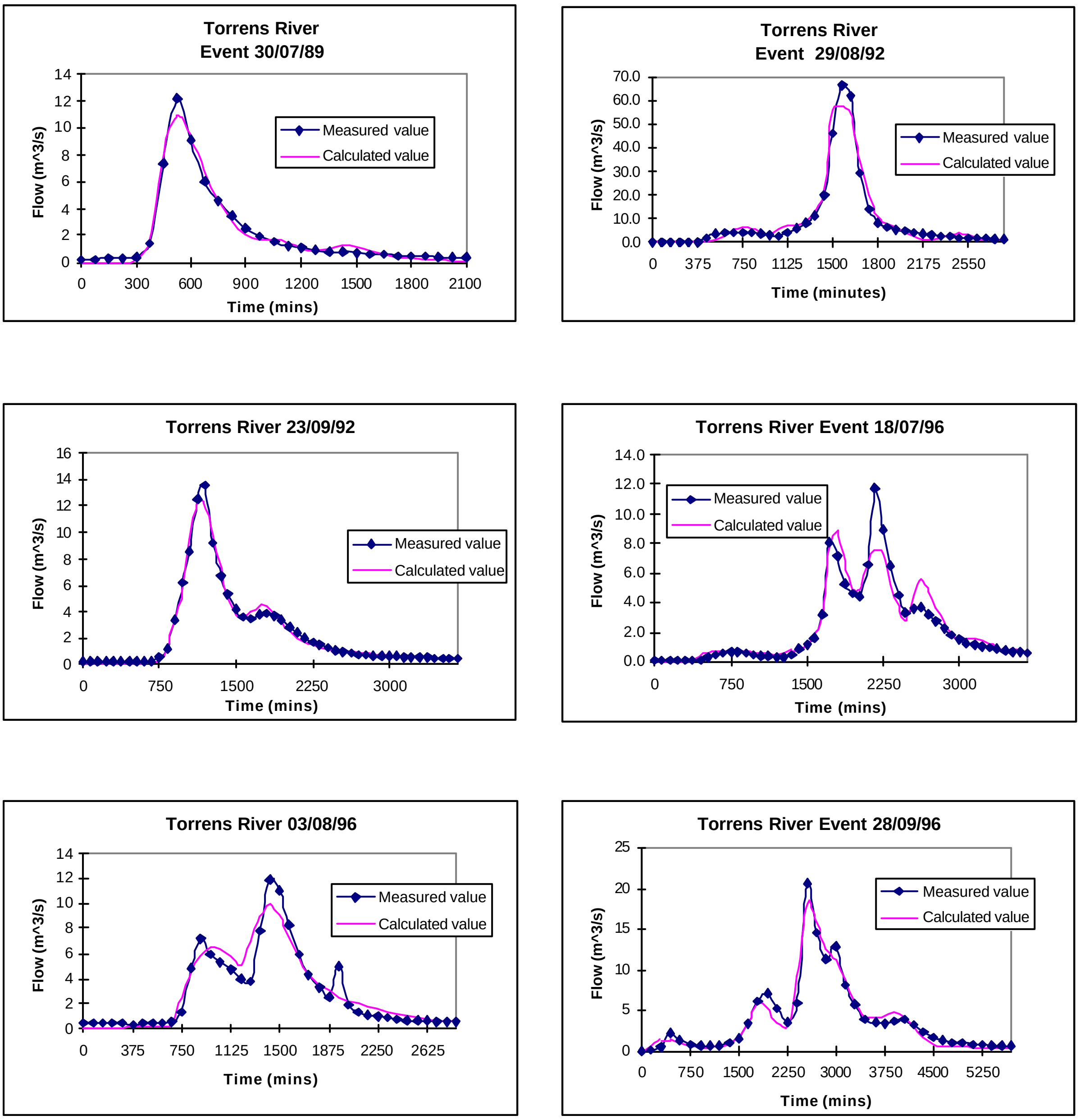


Figure 9-17 Torrens River Calibration Hydrographs

Table 9-12 River Torrens Verification Parameters

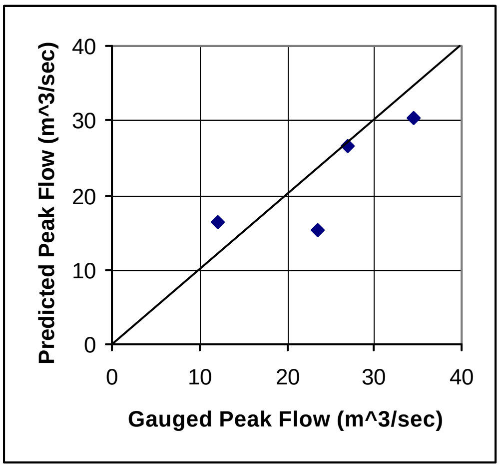
IL1(mm)	IL2(mm)	PL1	PL2	k	k_p1	k_p2
0.0	Derived from relationship	0.75	0.28	0.258	1.257	0.406

For the purpose of verification it was assumed that the process 3 would not occur. It is expected that this process will occur only rarely and there was insufficient evidence from the calibration runs as to the losses to be applied. All verification events had an ARI of less than 10 years.

The results of the verification process are given in Table 9-13.

Table 9-13 River Torrens Verification Results

Date	Rainfall (mm)	Gauged peak flow (m^3/s)	Predicted peak flow (m^3/s)	Predicted peak / gauged peak	Gauged volume (m^3)	Predicted volume (m^3)	Predicted volume / gauged volume
07/10/92	41.8	23.6	15.2	0.64	958 600	855 300	0.89
16/12/92	136.2	27.0	26.7	0.99	2 365 000	2 890 000	1.22
20/07/95	67.0	34.4	30.4	0.98	1 479 000	1 550 000	1.05
25/08/96	23.8	12.0	16.5	1.38	296 600	423 200	1.43
			mean	0.97			1.15



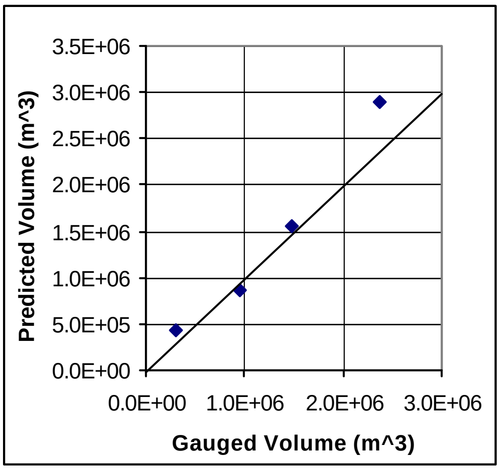


Figure 9-18 Torrens River RRR Verification Results

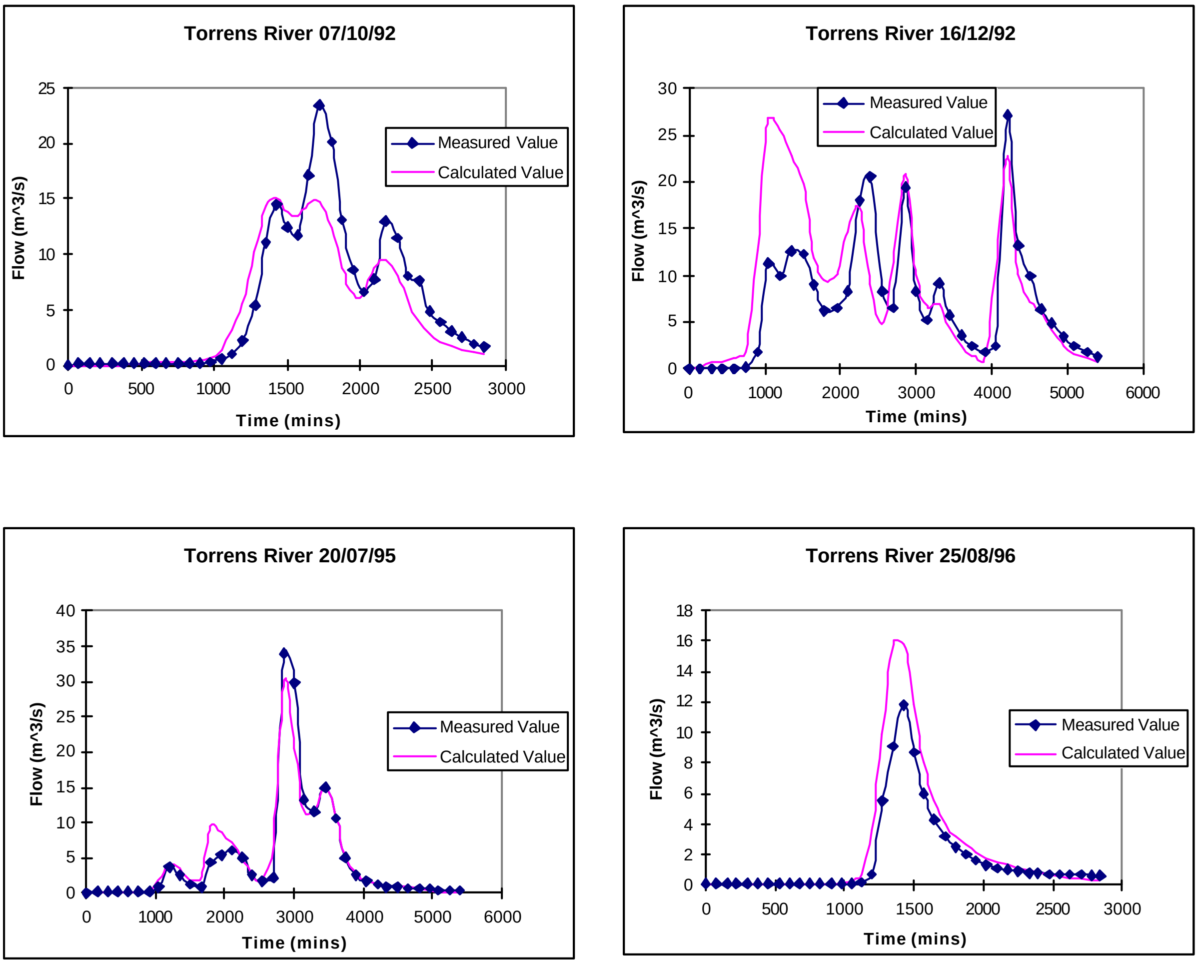


Figure 9-19 Torrens River Verification Hydrographs

The verification runs show good timing of peaks, although in some cases the peak flows are not well modelled. The event of 16/12/92 shows a good fit after the initial part of the storm, indicating that the IL2 was possibly not realistic.

9.3.5 Inverbrackie Creek

The second rural catchment used for the verification of the RRR model was the Inverbrackie Creek catchment, again in the Mount Lofty Ranges, approximately 26 km from Adelaide. The predominant land use is grazing, with some cropping and horticulture.

Physical data for the catchment are as follows:

Area	8.4 km^2
Main Stream Length	6.1 km
S_e (equal area slope)	15.6 m/km
Average Annual Rainfall	812mm (at Woodside township)

Figure 9-20 View of the Inverbrackie Creek Catchment

The catchment is served by a gauging station and pluviometer maintained by the Department for Environment, Heritage and Aboriginal Affairs (AW503508). The station is located at a natural rockbar, and has a concrete low flow section. One rating relationship exists covering the whole period of record comprising 78 gaugings to a flow of 4.79m^3/sec. A theoretical extension has been made to 24m^3/sec. The pluviometer is located at the site of the gauging station. Data are available for both stream flow and rainfall from 1989 to 1997.

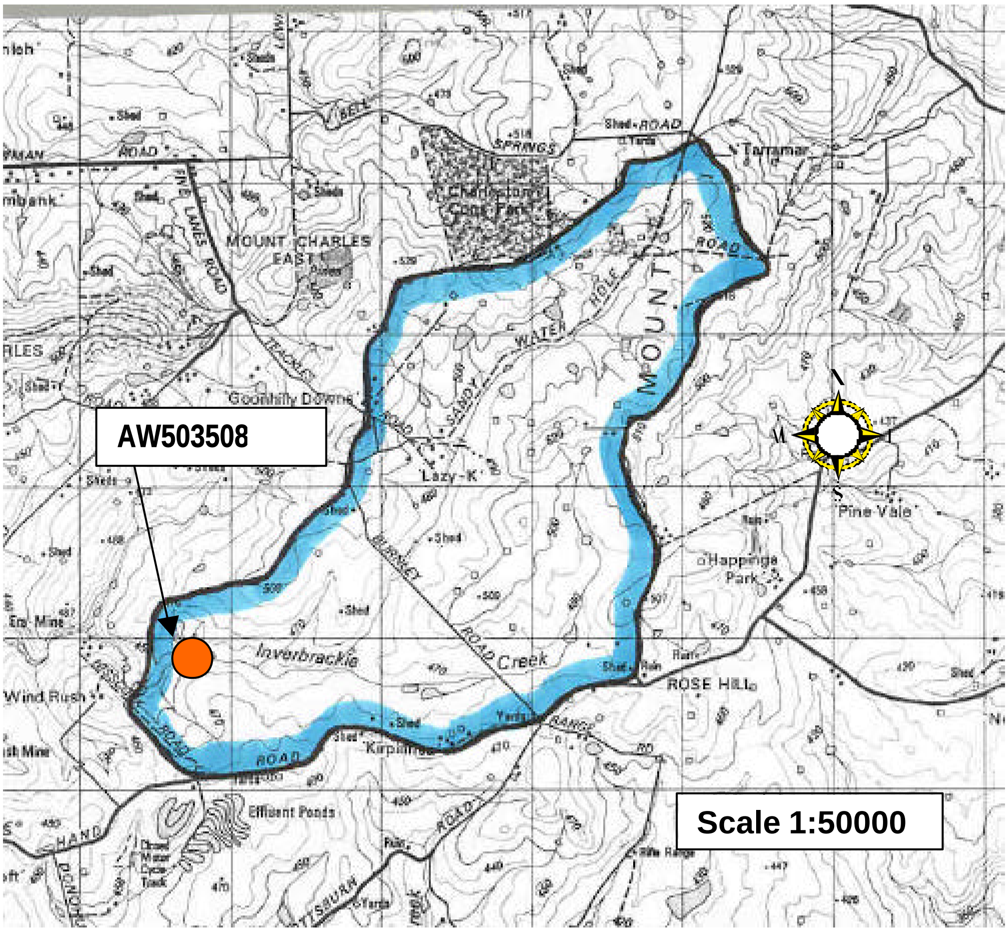


Figure 9-21 Inverbrackie Creek Catchment

The catchment is shown on Figure 9-21. Data were obtained for 13 highest flow events in the period of record, and calibration carried out on six of these using PEST. All events had an ARI of less than 10 years.

It was assumed in the calibration that three runoff processes were occurring. Table 9-14 shows the calibrated parameter values. In five of the six events two runoff processes were found, and in the other only baseflow was present. The process storage parameters k_p1 and k_p2 show considerable variation between events. The event of 29/08/92 showed zero proportional loss for the second process, which together with the runoff occurring from process 1 indicates that for some time during the event runoff was exceeding rainfall. Since this is not possible, it is most likely that catchment rainfall is not being represented by the pluviometer during at least part of the storm.

Table 9-14 Inverbrackie Creek RRR Model Calibrated Parameter Values

Event date	PL1	IL2 (mm)	PL2	IL3 (mm)	PL3	k_p1	k_p2	k (hrs)	MEAN ERROR (m^3/s)
14/07/87	0.87	22.7	0.37	*	*	0.84	0.31	0.168	0.204
05/06/88	0.73	21.0	0.69	*	*	0.62	0.12	0.299	0.147
19/08/90	0.71	2.5	0.44	*	*	1.88	0.37	0.139	0.094
04/07/90	0.59	*	*	*	*	0.62	*	0.213	0.164
29/08/92	0.82	14.8	0.00	*	*	2.31	0.40	0.231	0.524
28/09/96	0.60	23.0	0.45	*	*	0.89	0.34	0.146	0.266

The calibrated hydrographs are shown on Figure 9-22.

The fits achieved in the calibration were good, with multiple peaked storms modelling well. Again the use of a proportional loss gave a better result than the continuing loss. The parameters for verification were then determined. No relationship was found for the IL2. It was assumed that no process 3 (fast flow) was occurring, as there was no fast runoff found during the calibration events. In addition, all verification events had an ARI of less than 10 years, and fast runoff would not be expected.

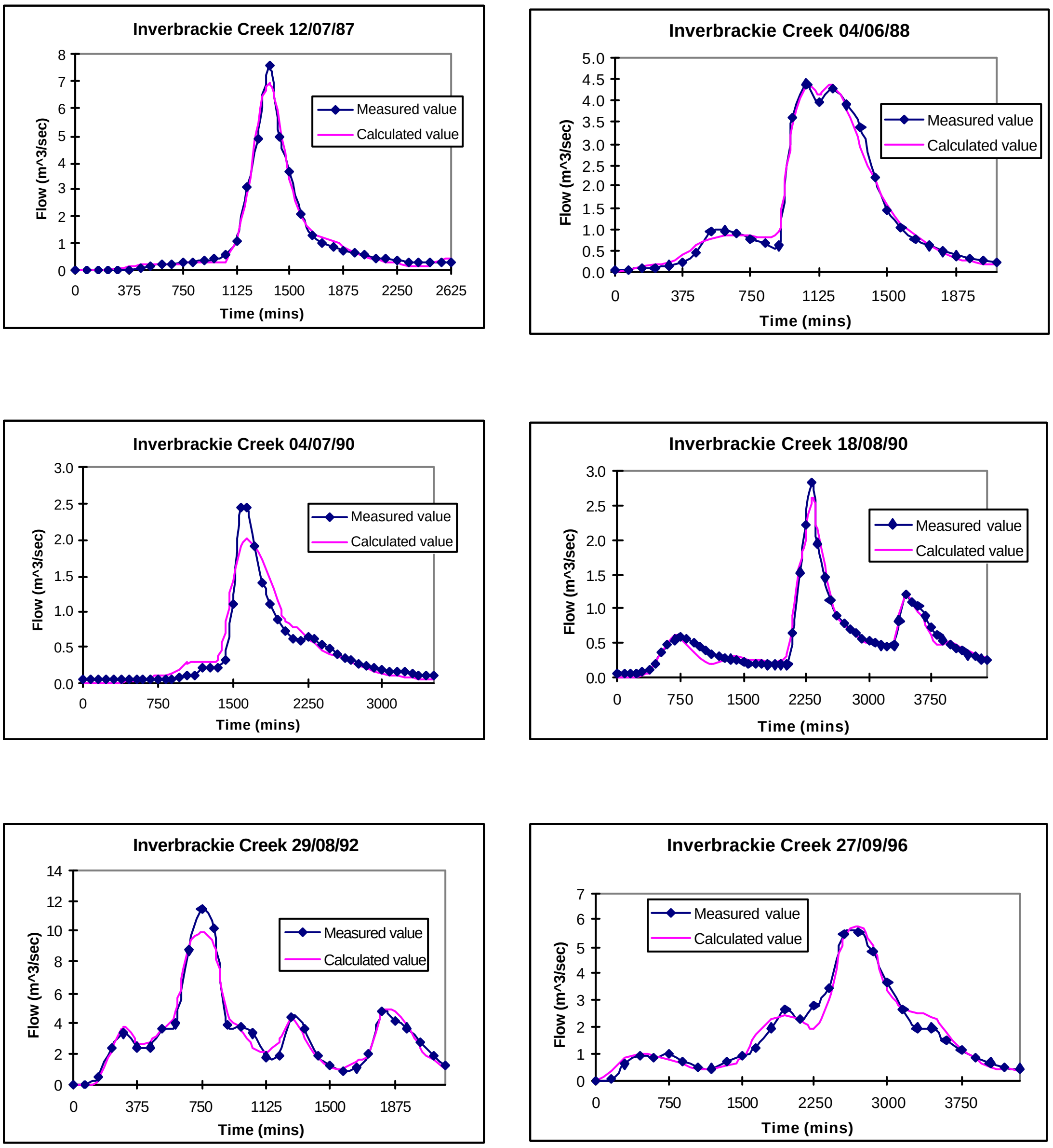


Figure 9-22 Inverbrackie Creek Calibration Hydrographs

Table 9-15 Inverbrackie Creek Verification Parameters

IL1(mm)	IL2	PL1	PL2	k	k_p1	k_p2
0.0	16.9	0.74	0.42	0.198	1.181	0.299

The results of the verification are given in Figure 9-23.

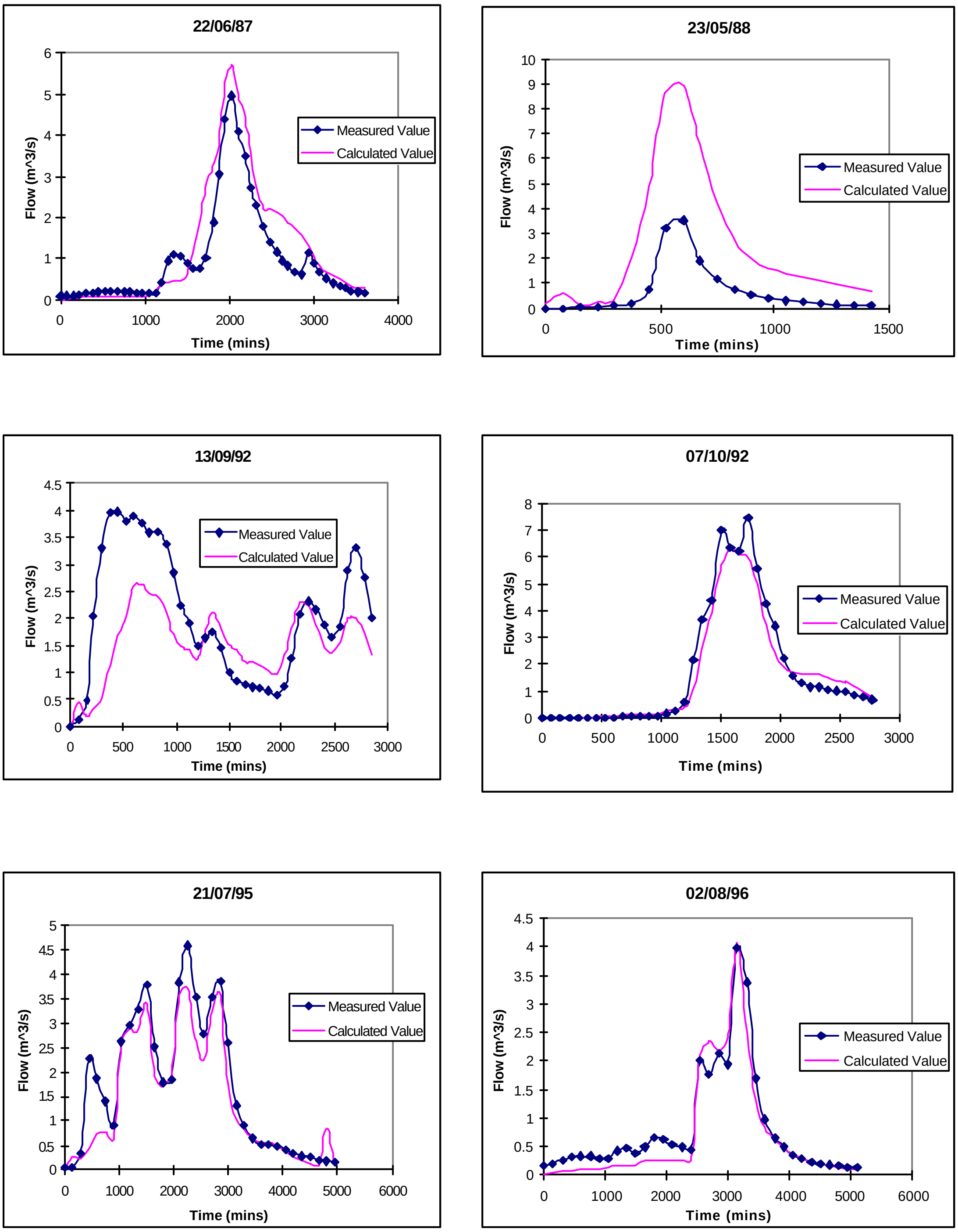


Figure 9-23 Inverbrackie Creek Verification Hydrographs

Table 9-16 Inverbrackie Creek Verification Results

Date	Rainfall (mm)	Gauged peak flow (m^3/s)	Predicted peak flow (m^3/s)	Predicted peak /gauged peak	Gauged volume (m^3)	Predicted volume (m^3)	Predicted volume/ gauged volume
07-10-92	55.4	7.69	6.36	0.83	311 500	298 000	0.96
14-09-92	53.2	4.01	2.71	0.67	356 200	319 300	0.90
22-06-87	56.2	5.20	5.76	1.11	221 600	308 600	1.40
21-07-95	70.4	4.60	3.98	0.87	513 100	416 700	0.81
23-05-88	45.5	3.89	9.29	2.39	61 700	231 300	3.75
02-08-96	40.7	4.07	4.07	1.00	251 100	206 000	0.82
			mean	1.14			1.44

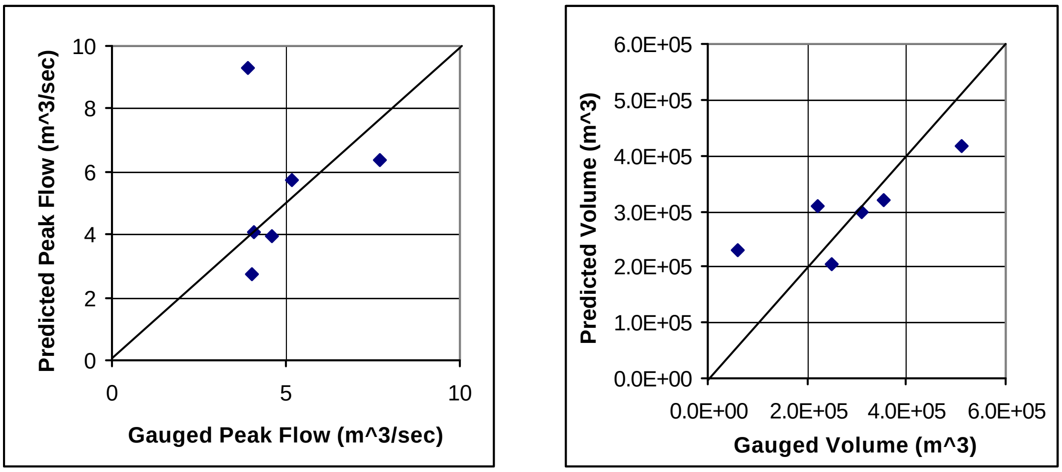


Figure 9-24 Inverbrackie Creek Verification Results

If the event of the 23/05/88 is ignored as an outlier, as both the peak flow and volume ratios are large then the mean ratio of predicted/gauged peak flows is 0.90, and the ratio of predicted/gauged volumes is 0.98. The gauged rainfall in the event of 23/05/88 may not have been representative of true catchment rainfall. All events show good correlation of the time of rise of the hydrographs, and multiple peaks are also modelled well. The event of 23/05/88 has the worst fit. The rainfall for this event may not have been representative of the catchment rainfall, but in the absence of any other gauges within the catchment it is not possible to confirm this.

9.3.6 Echunga Creek

The third rural catchment used for the verification of the RRR model was the Echunga Creek catchment, again in the Mount Lofty Ranges, approximately 26 km from Adelaide. The predominant land use is grazing, with some cropping and horticulture.

Physical data for the catchment are as follows:

Area	34.2km^2
Main Stream Length	13.5 km
S_e (equal area slope)	4.6 m/km
Average Annual Rainfall	808 mm (Echunga township)

Figure 9-25 View of the Echunga Creek Catchment

The catchment is shown on Figure 9-26.

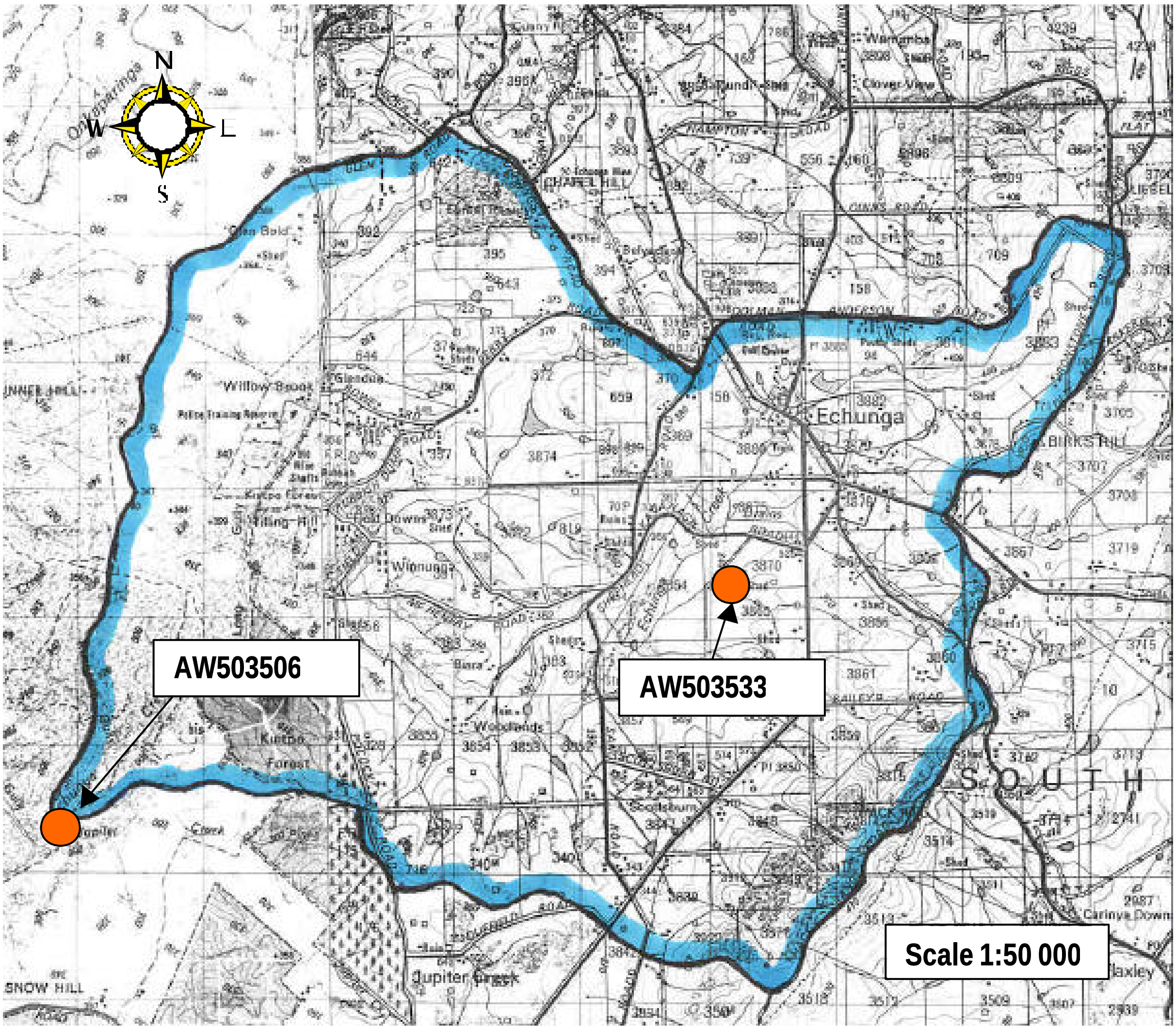


Figure 9-26 Echunga Creek Catchment

The catchment is served by a gauging station (AW503506) and pluviometer (AW503533) maintained by the Department for Water Resources. The station consists of a 90 deg V notch sharp edge concrete weir. One rating relationship exists covering the whole period of record comprising 85 gaugings to a flow of 18.82m^3/sec. A theoretical extension has been made to 45.4m^3/sec. The pluviometer is located close to the centroid of the catchment and is installed within a cleared compound which conforms to Bureau of Meteorology installation guidelines. Data are available for both stream flow and rainfall from 1989 to 1997.

Data were obtained for 13 events having the highest flows in the period of record, and calibration carried out with PEST. Of the six events chosen for calibration, one had an ARI of approximately 20 years (29/08/92). The rest had an ARI of less than 10 years.

It was assumed for the calibration that three runoff processes were occurring. In some cases the fit achieved during the calibration was very good, particularly for storms having multiple peaks. Table 9-17 shows the calibrated parameter values. One storm event had three processes, and one had only the slow flow. The others had baseflow and slow flow present. Fitted parameter values were reasonably consistent for all events, apart from the event of 29/08/92, which had a much lower proportional loss, indicating that a much higher proportion of rainfall appeared as runoff.

Table 9-17 Echunga Creek RRR Model Calibration Parameter Values

Event date	PL1	IL2 (mm)	PL2	IL3 (mm)	PL3	k_p1	k_p2	k (hrs)	MEAN ERROR (m^3/s)
10/09/89	0.84	16.8	0.79	*	*	1.81	0.432	0.267	0.214
04/07/90	0.96	5.2	0.72	*	*	1.80	0.327	0.246	0.361
14/08/90	0.94	5.1	0.65	*	*	2.35	0.530	0.518	0.214
29/08/92	*	17.0	0.18	*	*	*	0.467	0.263	2.07
17/12/92	0.81	1.8	0.79	*	*	3.46	0.324	0.289	0.967
20/07/95	0.80	7.1	0.75	25	0.82	1.10	0.119	0.371	1.07

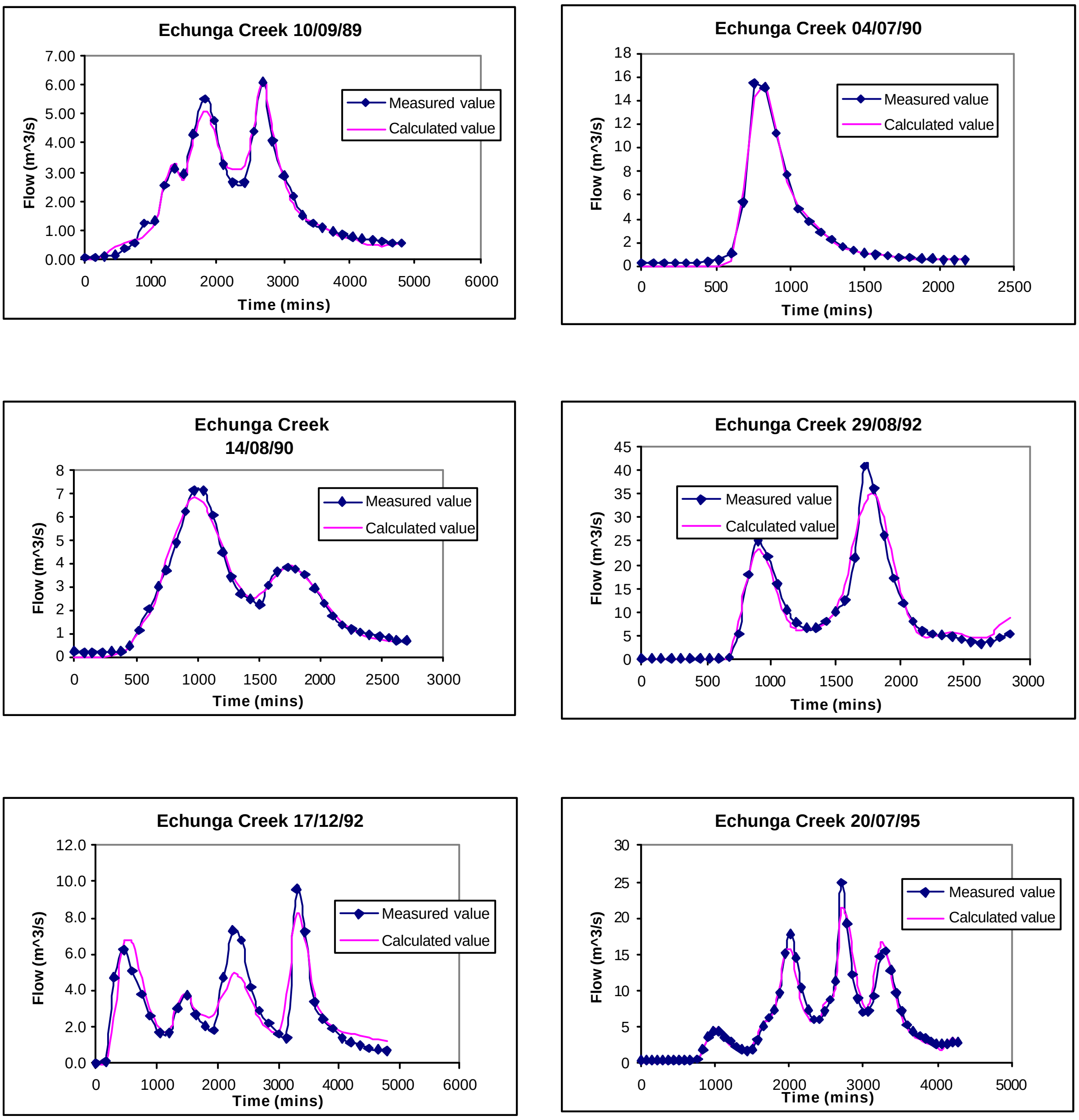


Figure 9-27 Echunga Creek Calibration Hydrographs

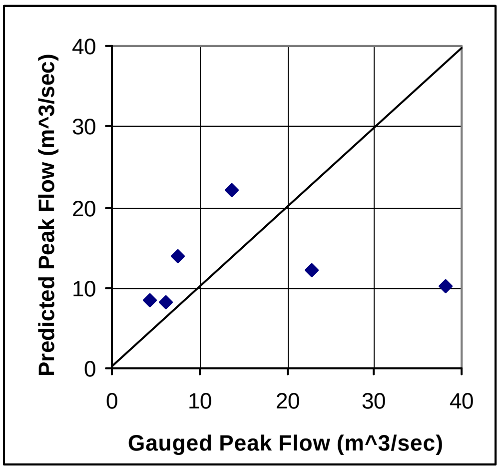
As with Inverbrackie Creek there was no discernible relationship between IL2 and API or initial baseflow, so verification was carried out using the weighted mean for all parameters, with the result given in Table 9-18. As there was insufficient evidence to determine loss parameters for fast flow, it was assumed that only two runoff processes were occurring. One event (30/07/96) had an ARI near 20 years, and all the rest were smaller than this. It would not be expected than fast runoff would occur for these frequent events.

Table 9-18 Echunga Creek Verification Parameters

IL1(mm)	IL2(mm)	PL1	PL2	k	k_p1	k_p2
0.0	8.7	0.89	0.73	0.329	1.945	0.375

Table 9-19 Echunga Creek RRR Verification Results

Date	Rainfall (mm)	Gauged Peak Flow (m^3/s)	Predicted Peak Flow (m^3/s)	Predicted Peak / Gauged Peak	Gauged volume (m^3)	Predicted volume (m^3)	Predicted volume / gauged volume
03-07-92	35.4	7.48	13.92	1.96	122 500	374 300	3.06
07-07-93	100.0	13.62	22.03	1.62	304 000	1 140 000	3.75
09-07-91	28.4	4.37	8.44	1.93	115 500	240 600	2.08
11-07-95	36.0	6.06	8.13	1.34	223 700	343 800	1.54
29-09-96	53.4	22.76	12.07	0.53	903 500	554 800	0.61
03-08-96	88.8	38.24	10.30	0.28	1 896 000	1 050 000	0.55
			mean	1.26			1.93



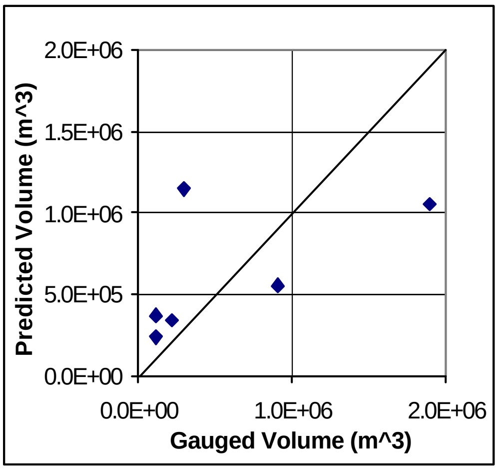


Figure 9-28 Echunga Creek Verification Results

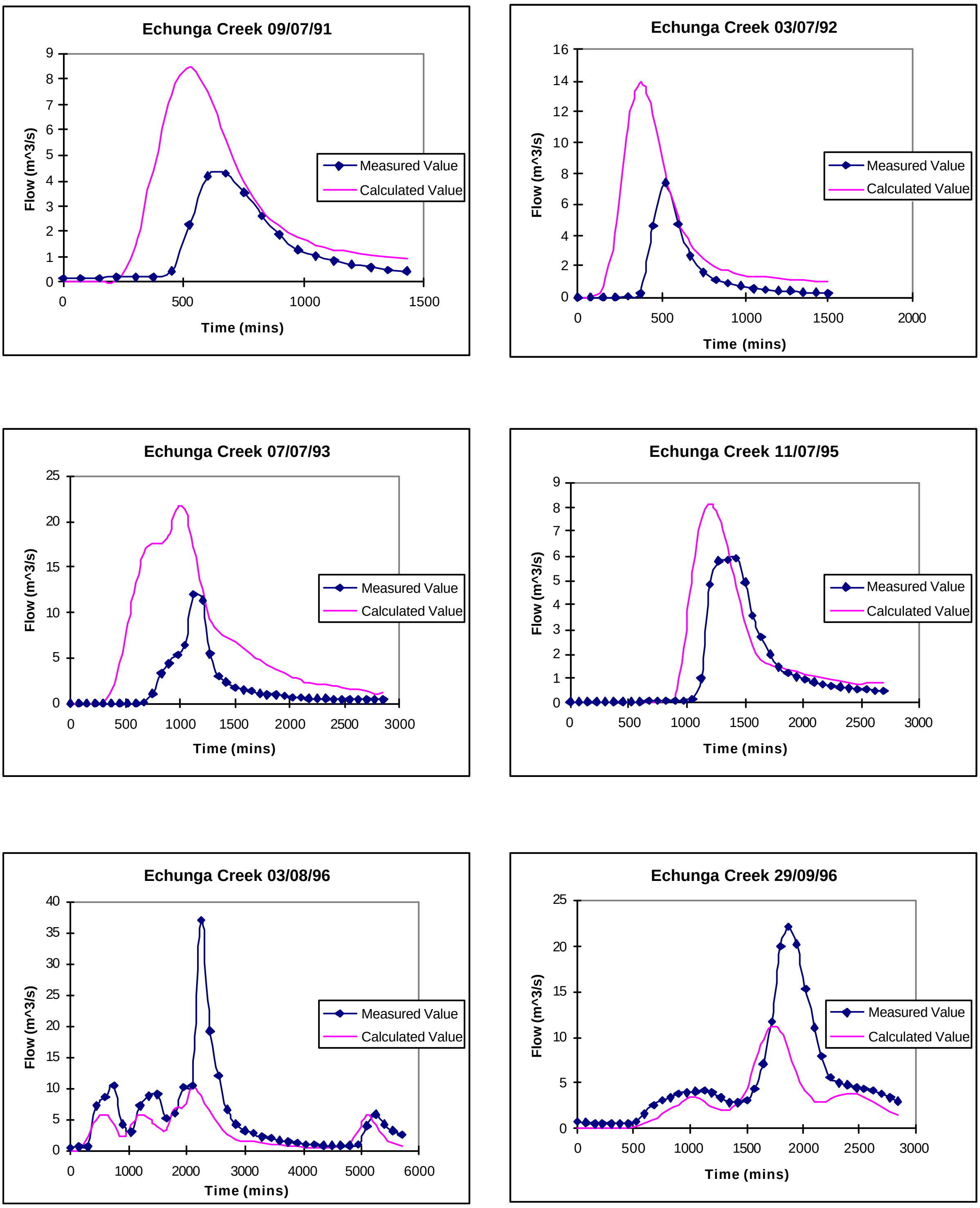


Figure 9-29 Echunga Creek Verification Hydrographs

The verification runs for Echunga Creek show considerable variation, particularly with regard to the predicted volume. This variation could be attributed to errors in the rainfall or the loss model.

It may be also that one calibration event, of 29/08/92 is biasing the losses. This event had a lower proportional loss than any other event.

9.3.7 Scott Creek

The fourth Adelaide Hills catchment used for verification was that of Scott Creek, which lies in an area of higher rainfall than the other catchments. Land use in the catchment consists of some grazing, and natural vegetation. Catchment details are as follows:

Area	26.8km^2
Main Stream Length	10.0 km
S_e (equal area slope)	19.5 m/km
Average Annual Rainfall	900 mm

The catchment has a gauging station with a pluviometer located at the catchment outlet (AW503502). The station consists of a concrete rectangular stepped weir with steel knife edge. A series of stage-discharge ratings apply to the entire period of record, with changes based on changes in cease to flow datum or weir profile modification. The most recent rating covers the record since the addition of the 90 degree V notch section on 06/04/1984, comprising 13 gaugings with a maximum flow gauged being 2.38m^3/sec. A theoretical extension has been made to 19.5m^3/sec. The tipping bucket pluviometer is installed in clear compound to BoM guidelines.

There is also a pluviometer maintained by the Bureau of Meteorology (BM023108) located at Longwood, at the upper end of the catchment. Data for the full period of record chosen for the calibration and verification was not available for BM023108. The catchment is shown on Figure 9-31.

Figure 9-30 View of the Scott Creek Catchment

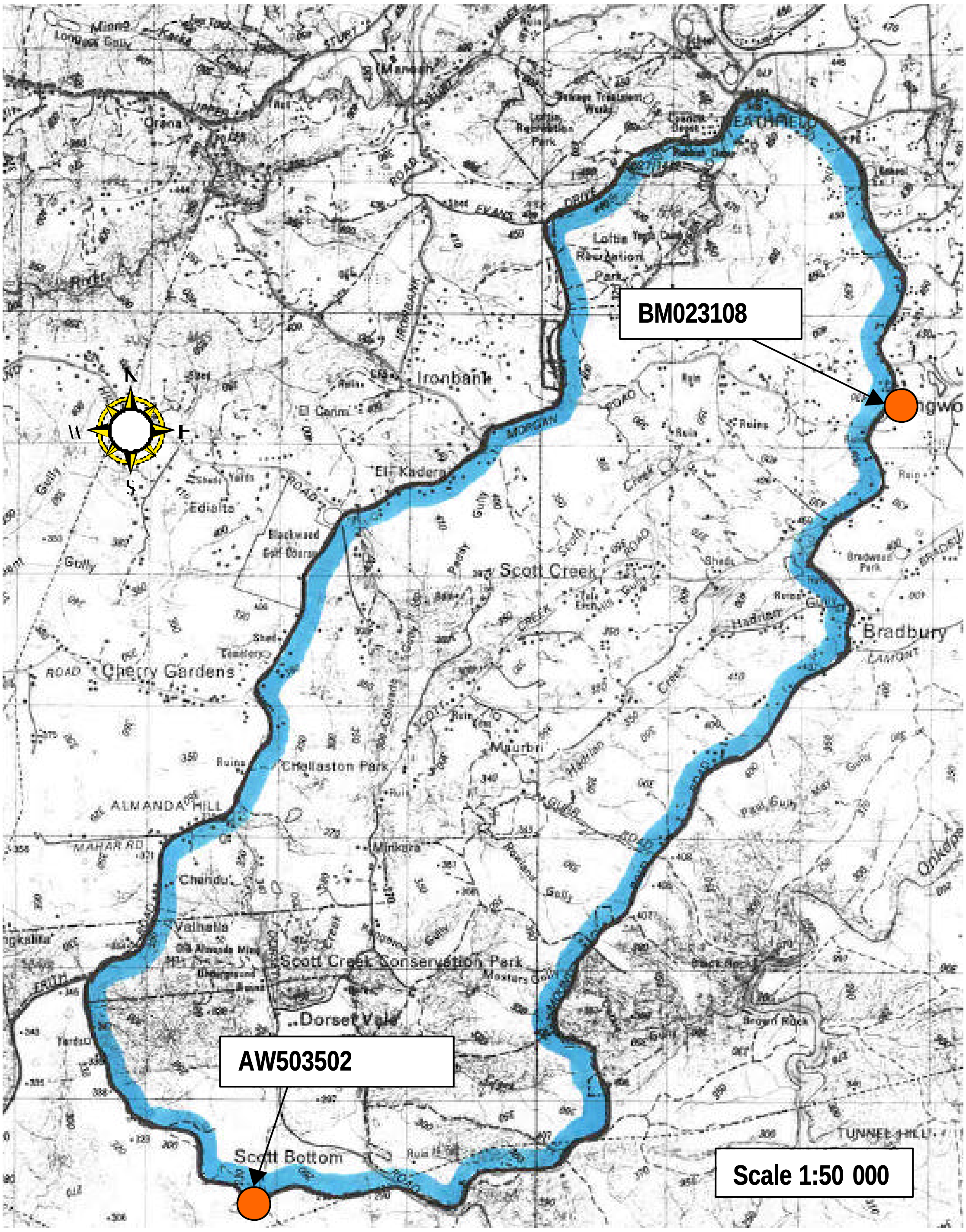


Figure 9-31 Scott Creek Catchment

Six events were chosen for calibration. One calibration event (02/08/96) had an ARI of approximately 10 years. All the rest were smaller events.

The mean error of estimate of the continuing loss versus the proportional loss varied, but the proportional loss case had the best fit, and the continuing loss case the worst fit, being the event of 02/08/96. This event had a better fit with only one contribution, being that of process 1 with a proportional loss. Two runoff processes were found for each event. Table 9-20 shows the calibrated parameter values. On event (11/07/95) showed a proportional loss of zero for the second process. This indicates that more runoff is occurring than rainfall. The pluviometer rainfall is possibly not representative of catchment rainfall for this event.

Table 9-20 Scott Creek RRR Model Calibrated Parameter Values

Event date	PL1	IL2 (mm)	PL2	IL3 (mm)	PL3	k_p1	k_p2	k (hrs)	MEAN ERROR (m^3/s)
14/09/91	0.84	10.8	0.73	*	*	1.3	0.391	0.075	0.099
16/12/92	0.85	9.61	0.78	*	*	2.55	0.459	0.433	0.329
05/07/93	0.99	40.0	0.85	*	*	2.0	0.567	0.226	0.320
11/07/95	0.74	21.5	0.0	*	*	1.04	0.323	0.292	0.241
20/07/95	0.52	9.13	0.86	*	*	2.43	0.297	0.173	0.774
02/08/96	0.72	43.1	0.74	*	*	1.66	0.535	0.147	0.399

There were no relationships discernible between IL2 and API. Therefore the weighted mean values of all parameters were used for the verification, as given in Table 9-21. As no process 3 (fast flow) was detected during calibration, it was assumed that it would not occur in the verification events. One event (29/08/92) had an ARI close to 10 years, and all other events were smaller than this.

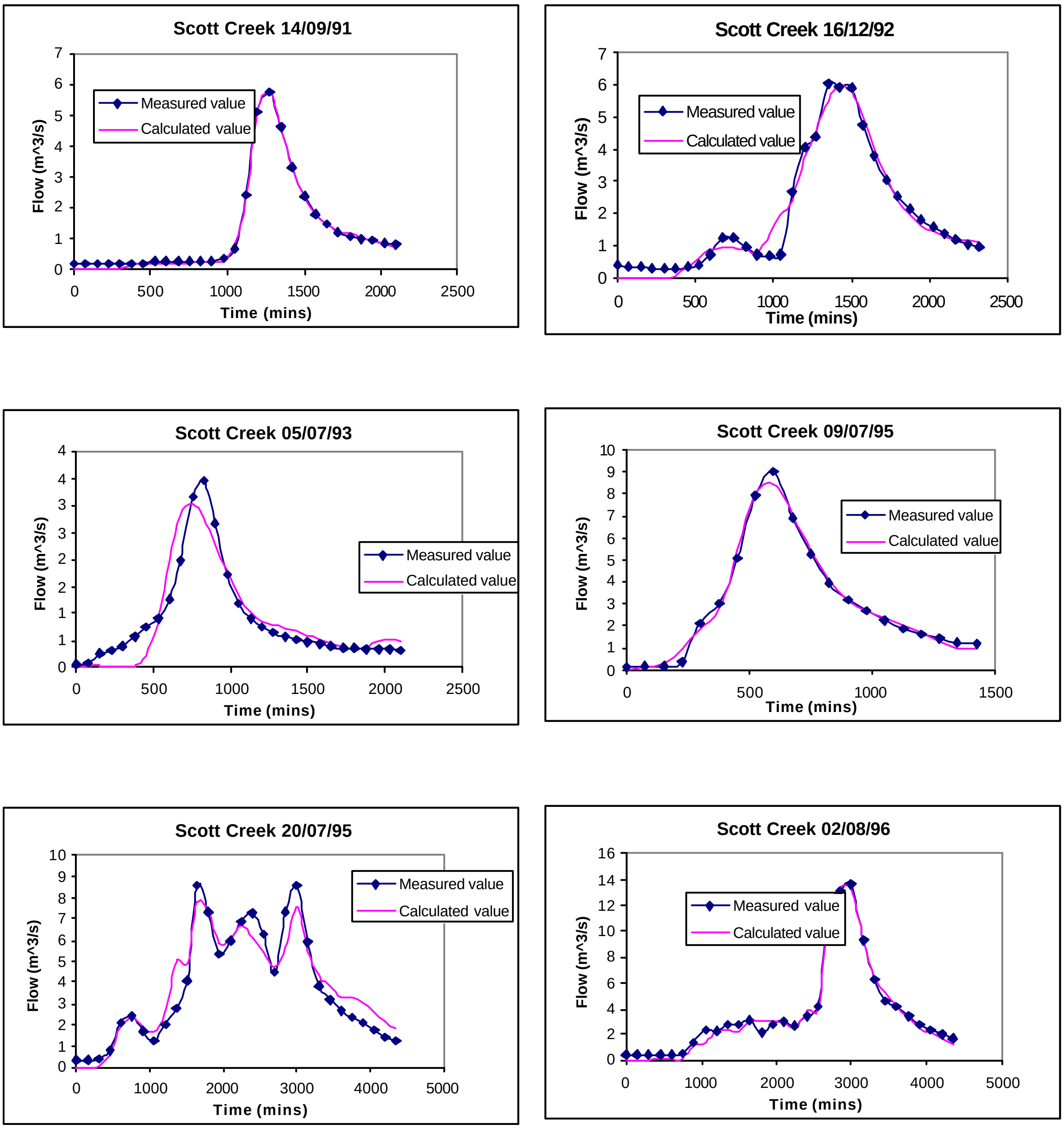


Figure 9-32 Scott Creek Calibration Hydrographs

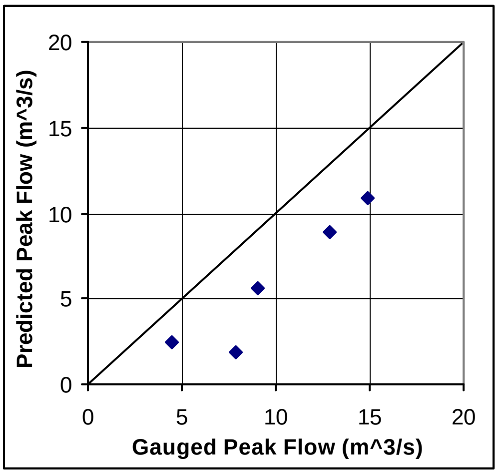
Table 9-21 Scott Creek Verification Parameters

IL1(mm)	IL2(mm)	PL1	PL2	k	k_p1	k_p2
0.0	21.6	0.78	0.76	0.234	0.873	0.241

Verification was carried out as before, with the result given in Table 9-22 and shown on Figure 9-33.

Table 9-22 Scott Creek RRR Verification Results

Date	Rainfall (mm)	Gauged Peak Flow (m^3/s)	Predicted Peak Flow (m^3/s)	Predicted Peak / Gauged Peak	Gauged volume (m^3)	Predicted volume (m^3)	Predicted volume / gauged volume
08-10-92	55.6	9.04	5.64	0.62	682 000	507 800	0.74
18-07-96	34.2	4.46	2.40	0.54	265 000	259 500	0.97
25-08-91	23.0	7.87	1.88	0.24	232 700	139 400	0.60
29-08-92	84.0	14.89	10.86	0.73	1 242 000	906 300	0.73
28-09-96	61.8	12.83	8.85	0.69	873 600	567 000	0.65
			Mean	0.56			0.74



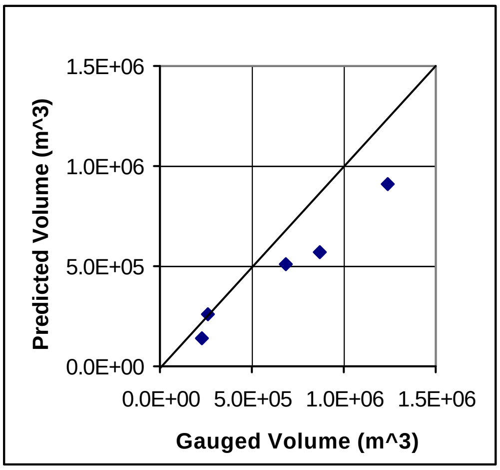


Figure 9-33 Scott Creek Verification Results - 1 Pluviometer

For four of the verification events rainfall data from the second pluviometer at Longwood (BM023108) was available. The verification for these events was carried out on a RRR model having the same global parameters, but two sub-catchments, with the results given in Table 9-23 and shown on Figure 9-34. The results are better than the single rainfall input, showing the benefit of having better data to use with the model. In general the RRR model was able to match the shape of the hydrographs well, but the volume is less well matched. With 2 rainfall inputs the predicted and measured volumes are closer, indicating that the problem lies with the rainfall input to the model, and not the model itself.

Table 9-23 Scott Creek RRR Verification Results (2 Pluviometers)

Date	Gauged peak flow (m^3/s)	Predicted peak flow (m^3/s)	Predicted peak /gauged peak	Gauged volume (m^3)	Predicted volume (m^3)	Predicted volume/ gauged volume
08-10-92	9.04	8.16	0.90	682 000	476 800	0.70
18-07-96	4.46	5.25	1.18	265 000	374 800	1.41
29-08-92	14.89	11.13	0.75	1 242 000	953 200	0.77
28-09-96	12.83	9.89	0.77	873 600	644 800	0.74
		Mean	0.90			0.90

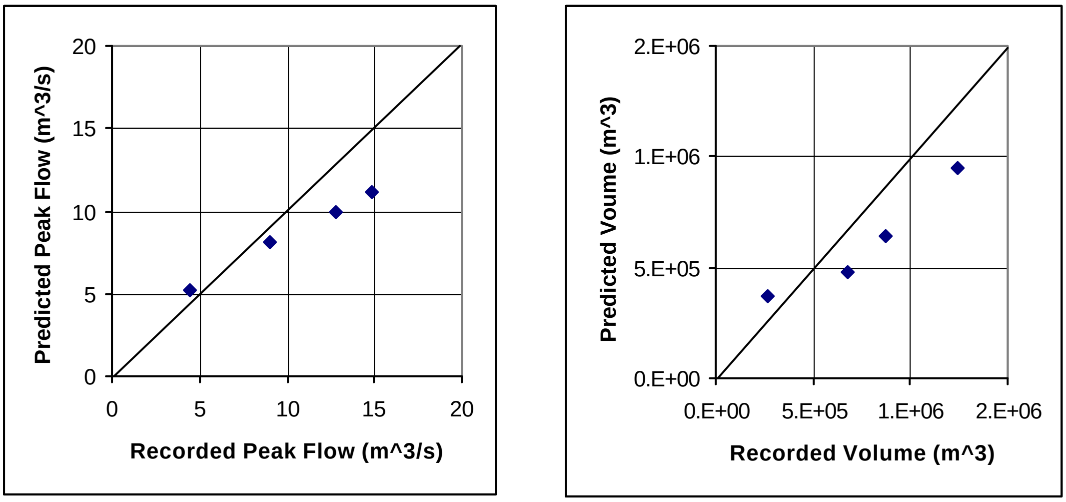


Figure 9-34 Scott Creek Verification Result - 2 Pluviometers

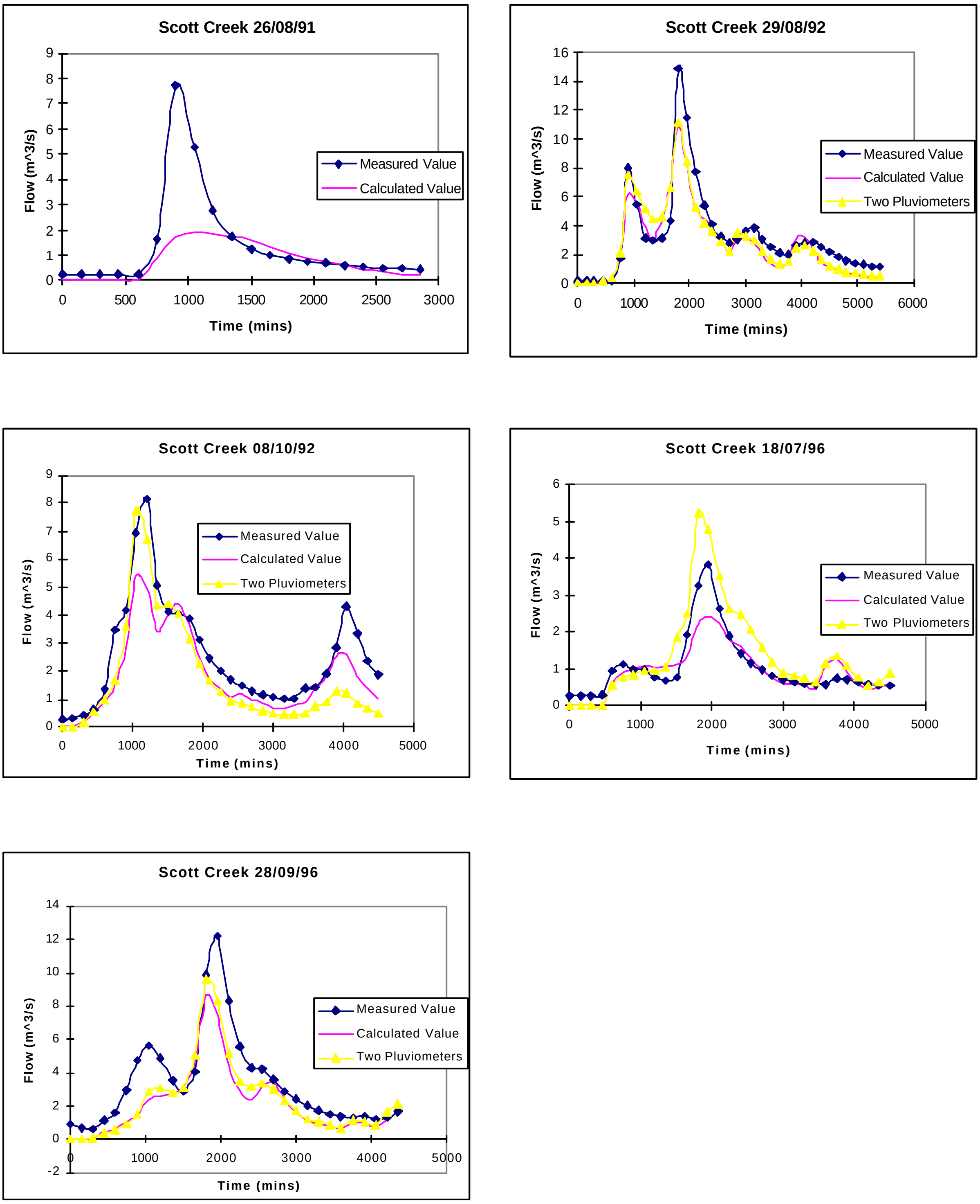


Figure 9-35 Scott Creek Verification Hydrographs

It is of note that in the event of 26/08/91 slow flow was not predicted, even though it has obviously occurred.

9.3.8 Celia Creek

The fifth rural catchment used for the verification of the RRR model was the Celia Creek catchment, situated in the Northern Territory approximately 100km south of Darwin. The catchment is shown on Figure 9-36. This catchment was chosen as it has a completely different climate form the Mount Lofty Ranges, and is therefore a good test of the applicability of the model to a different climate region. The catchment is predominantly in natural condition.

Physical data for the catchment are as follows:

Area	52.2 km^2
Main Stream Length	11.0 km
S_e (equal area slope)	2.5 m/km
Average Annual Rainfall	1340mm

The catchment is served by a gauging station (G8150151) and three pluviometers (R8150151, R8150205 and R8150332) maintained by the Department of Lands, Planning and the Environment. The pluviometers are located at the site of the gauging station, and just outside the upper end of the catchment. Data are available for both stream flow and rainfall from 1989 to 1998, but the gauging station was closed between August 1981 and February 1990. The control is sheet piling, with a “V” notch. The maximum flow gauged of 39 gaugings is 31m3/sec. Access during much of the wet season is poor, however the section is stable and the rating is reasonable.

Flood frequency analysis of the catchment was carried out as part of the design of the Alice Springs – Darwin railway, with the 5 year ARI flow being 102m^3/sec, and the 50 year ARI flow being 210m^3/sec (Weeks et al, 2002).

The catchment is substantially larger than the Mount Lofty Ranges catchments, and data from all pluviometers were used both in the calibration and verification of the model.

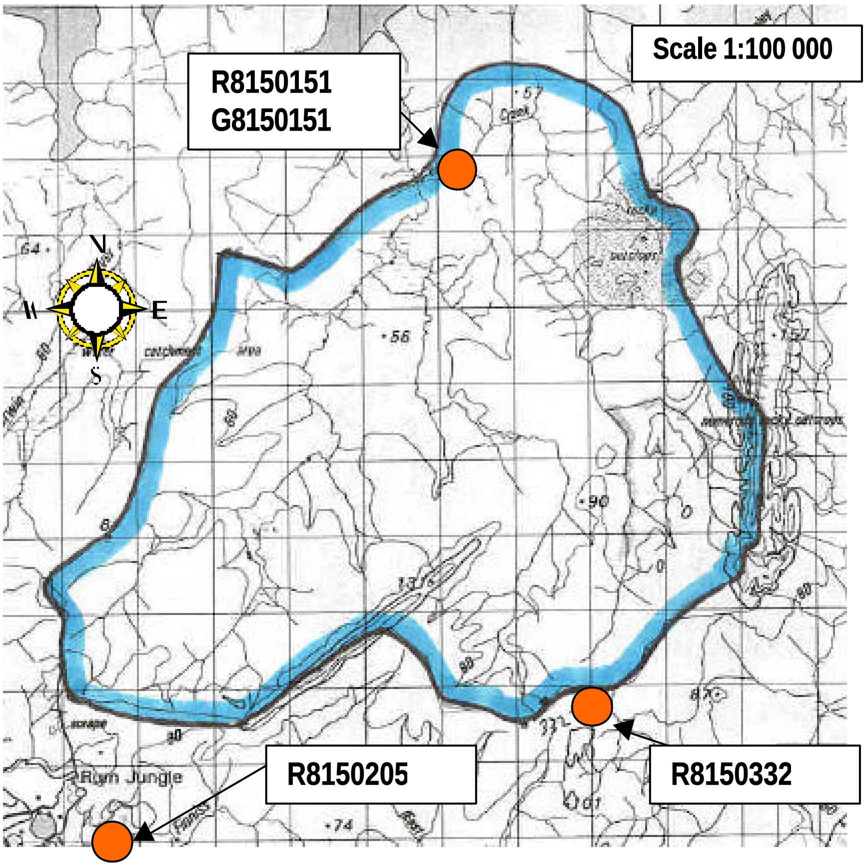


Figure 9-36 Celia Creek Catchment

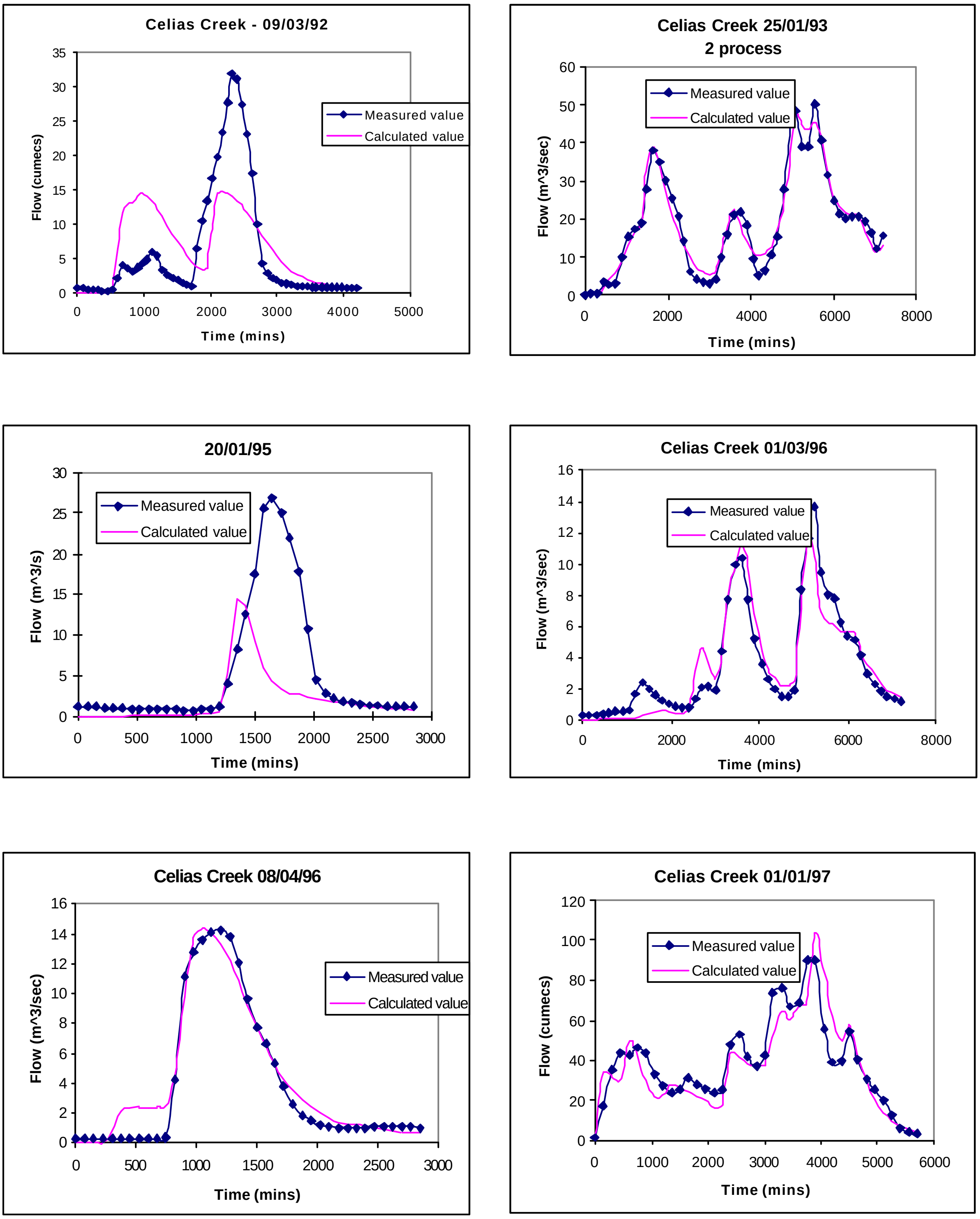
The calibration was first carried out using the Thiessen mean rainfall for the three pluviometers. However it became obvious that there were substantial differences between the rainfall patterns at the three stations. As a result it was decided to split the catchment into six sub-catchments, relating each to the nearest pluviometer. It was decided also to use more than the normal number of events for calibration, as this should result in a better definition of parameters.

In all cases it was assumed that three runoff processes were occurring, but it was found that only two were occurring. All flows used for calibration and verification were less than 5 year ARI. Table 9-24 shows the calibrated parameter values. The values of the process and channel lags are given for one sub-catchment only. Reasonable variation was found in all parameter values. This would be expected, given that the catchment is reasonably large, and the rainfall is not evenly distributed. This will affect both the loss and the storage parameters.

Table 9-24 Celia Creek RRR Model Calibrated Parameter Vaules (6 sub-catchment model)

Event date	PL1	IL2 (mm)	PL2	k1e	k2e	ke (hrs)	Mean Error (m^3/s)
10/03/92	0.63	>72.9	*	1.264	*	0.170	6.58
27/01/93	0.77	132.1	0.53	1.276	0.732	0.326	3.96
20/01/95	0.85	20.0	0.51	1.278	0.319	0.183	7.45
04/03/96	0.89	32.4	0.68	5.020	0.506	0.370	1.13
09/04/96	0.86	>76.3	*	0.960	*	0.169	1.00
01/01/97	0.59	0	0.76	1.390	0.597	0.011	10.3
30/01/97	0.0	29.5	0	0.823	0.620	0.340	3.84
19/02/97	0.16	27.8	0.99	3.63	2.04	0.109	4.95
01/03/97	0.57	29.6	0.65	1.440	0.304	0.046	3.31
15/01/98	0.97	45.8	0.57	0.693	0.977	0.239	1.12

Figure 9-37 shows the result of the calibration, using the six sub-catchments. Much of the error between the predicted and measured hydrograph could be attributed to the problems of the definition of rainfall.


Celias Creek - 09/03/92
Flow (cumecs)
Time (mins)
Measured value
Calculated value
Celias Creek 25/01/93
2 process
Flow (m^3/sec)
Time (mins)
Measured value
Calculated value
20/01/95
Flow (m^3/s)
Time (mins)
Measured value
Calculated value
Celias Creek 01/03/96
Flow (m^3/sec)
Time (mins)
Measured value
Calculated value
Celias Creek 08/04/96
Flow (m^3/sec)
Time (mins)
Measured value
Calculated value
Celias Creek 01/01/97
Flow (cumecs)
Time (mins)
Measured value
Calculated value

(Figure 9-37)

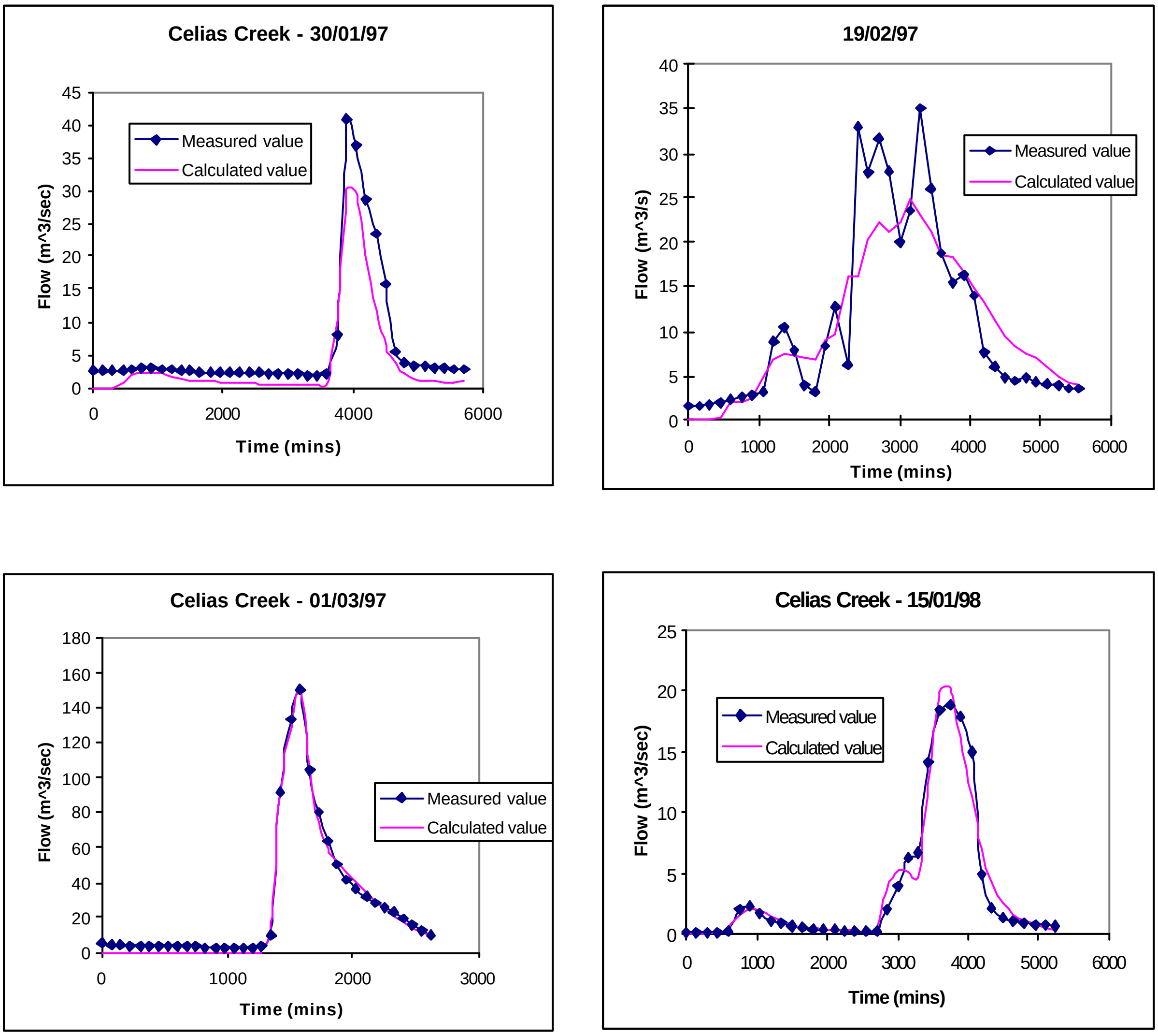


Figure 9-37 Celia Creek Calibration Hydrographs

Verification was carried out on six storm events. It was assumed in the verification that no fast runoff was occurring.

Table 9-25 Celia Creek Verification Parameters

IL1(mm)	IL2(mm)	PL1	PL2	ke	k1e	k2e
0.0	29.3	0.69	0.63	0.167	1.193	0.468

The results are given in Table 9-26:

Table 9-26 Celia Creek Verification Results

Date	Rainfalls (mm)	Gauged peak flow (m^3/s)	Predicted peak flow (m^3/s)	Predicted peak /gauged peak	Gauged volume (m^3)	Predicted volume (m^3)	Predicted volume/ gauged volume
13/03/92	55.1, 4.5, 13.8	20.3	16.8	0.83	719 000	619 900	0.96
25/02/93	83.8, 49.9, 82.1	18.2	14.9	0.82	1 774 200	1 814 800	1.02
25/02/94	204.3, 176.0, 198.7	68.9	52.5	0.96	7 419 900	6 304 500	0.85
04/03/94	80.8, 52.5, 66.0	58.3	30.2	0.52	2 672 700	1 777 000	0.66
30/01/96	80.3, 88.5, 10.9	18.8	16.7	0.89	1 572 100	2 045 200	1.30
20/01/97	192.0, 125.0, 157.5	43.6	54.0	1.24	6 011 800	5 021 900	0.83
			mean	0.84			0.92

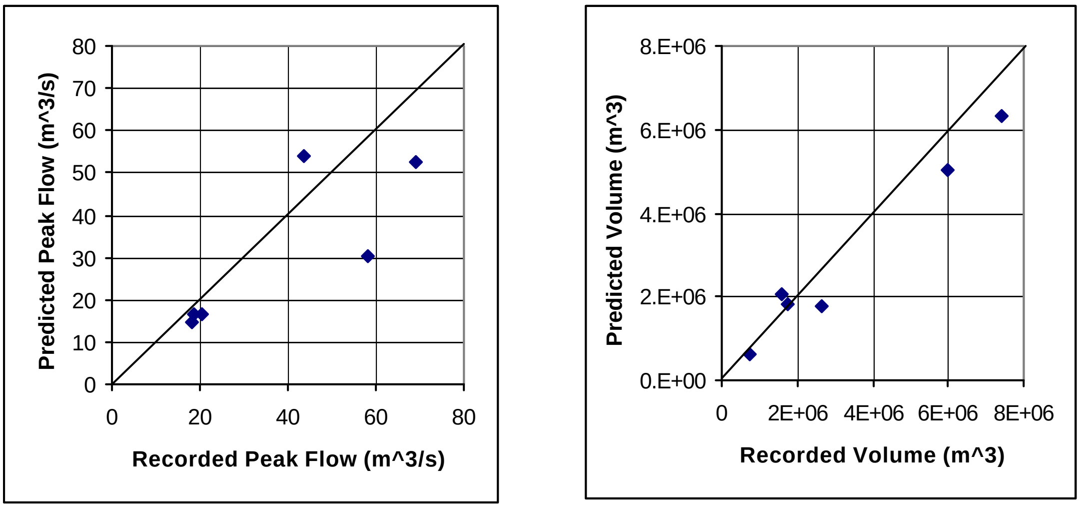


Figure 9-38 Celia Creek Verification Results

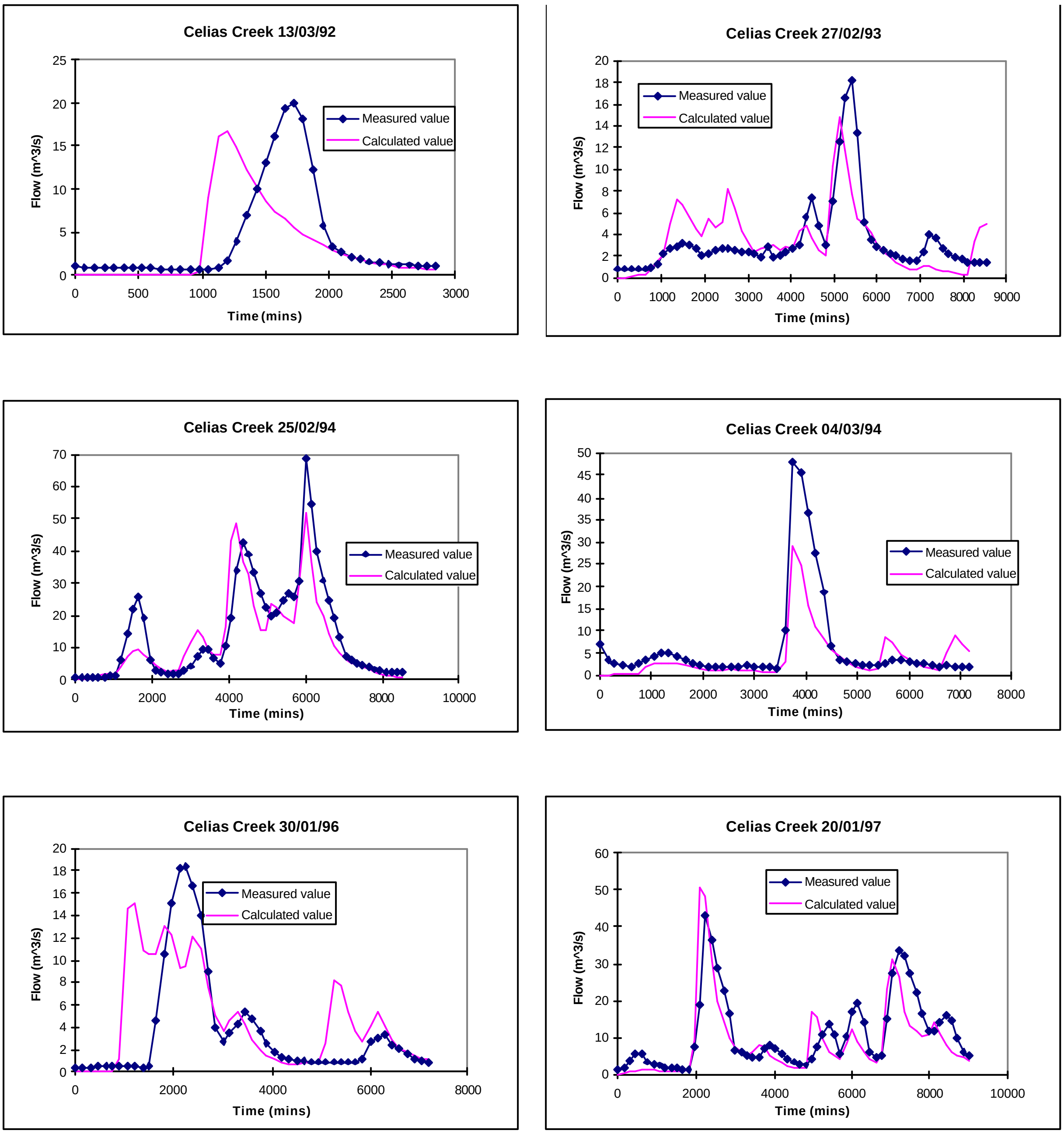


Figure 9-39 Celia Creek Verification Hydrographs

It can be seen that the verification results are mixed, with only some of the events being modelled reasonably well. The event of 13/03/92 is interesting, as the measured hydrograph is very different in shape to the predicted. This event has the most non-uniform flow, with the recorded rainfalls at the three pluviometers ranging from 4.5mm to 55.1mm. The general impression is that as the recorded rainfalls increase, and become more uniform the predicted hydrograph becomes closer to the recorded hydrograph. This would be expected if the rainfall information rather than the model is causing the difference between the recorded and predicted hydrograph.

9.3.9 Burra Creek

Burra Creek lies within state of New South Wales, approximately 30km south of Canberra. Land use is grazing, with natural vegetation in the upper parts of the catchment, particularly in the east. Its physical data are as follows:

Area	70 km^2
Main Stream Length	15.6 km
S_e	16 m/km
Average Annual Rainfall	660 mm

The catchment is served by a gauging station and pluviometer located at the catchment outlet (410774). The control is a concrete improved rock bar. One hundred and fifty nine gaugings have been carried out (to March 2002), with the highest gauging at 50.3m^3/sec. Calibration was carried out on seven runoff events, assuming three processes were occurring.

Figure 9-40 View of the Burra Creek Catchment

Table 9-27 Burra Creek RRR Model Calibrated Parameter Values

Event date	PL1	IL2 (mm)	PL2	IL3 (mm)	PL3	k_p1	k_p2	k (hrs)	Mean error (m^3/s)
04/06/88	0.86	20.6	0.75	> 44.2	n/a	2.06	0.430	0.201	0.603
05/07/88	0.81	19.3	0.64	47.8	0.59	2.32	0.677	0.083	0.803
14/03/89	0.91	59.6	0.81	105.5	0.74	2.51	0.246	0.226	1.130
09/04/89	0.73	10.1	0.73	> 37.2	n/a	2.06	0.466	0.260	0.829
11/06/91	0.93	18.9	0.76	> 50.6	n/a	3.10	0.551	0.248	0.348
09/01/92	0.94	11.4	0.81	> 45.6	n/a	1.85	0.531	0.284	0.774
12/04/94	0.96	45.9	0.74	> 62.3	n/a	1.04	0.276	0.279	1.203

Two runoff processes were found in five of the seven events, with three processes in the other two. Calibrated parameter values were reasonably consistent.

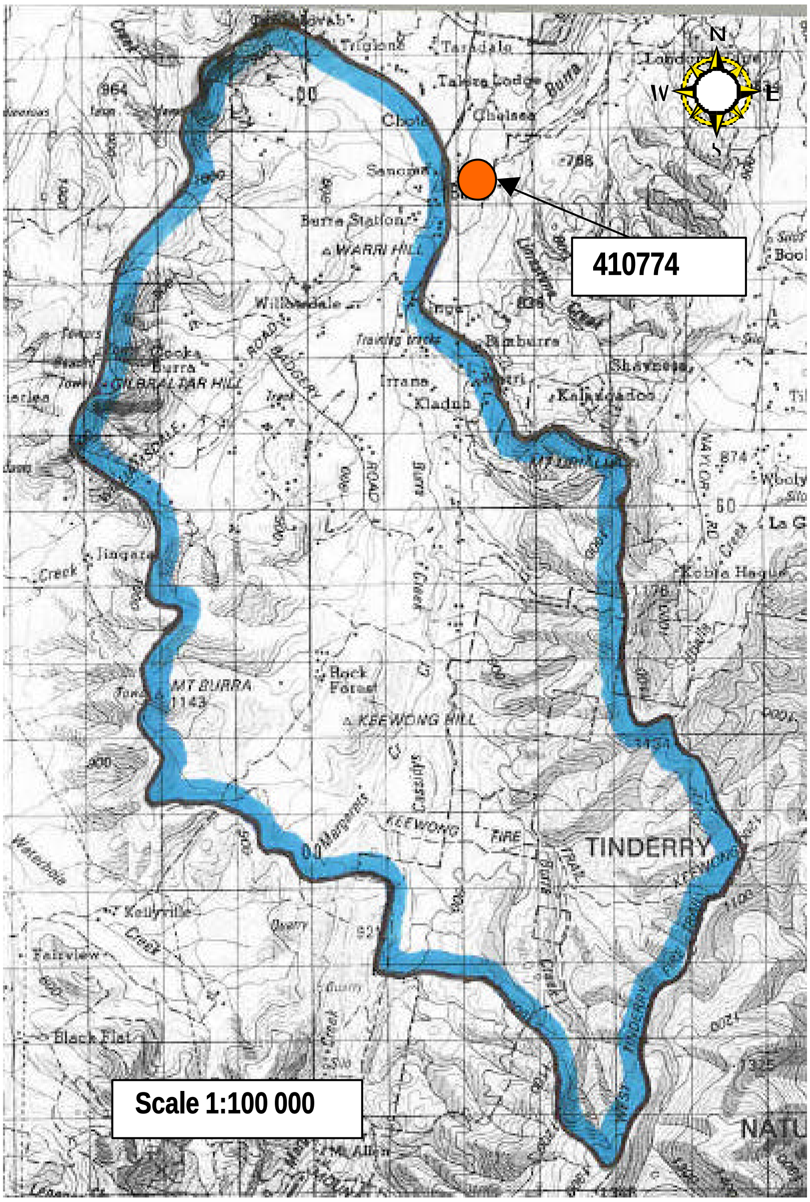
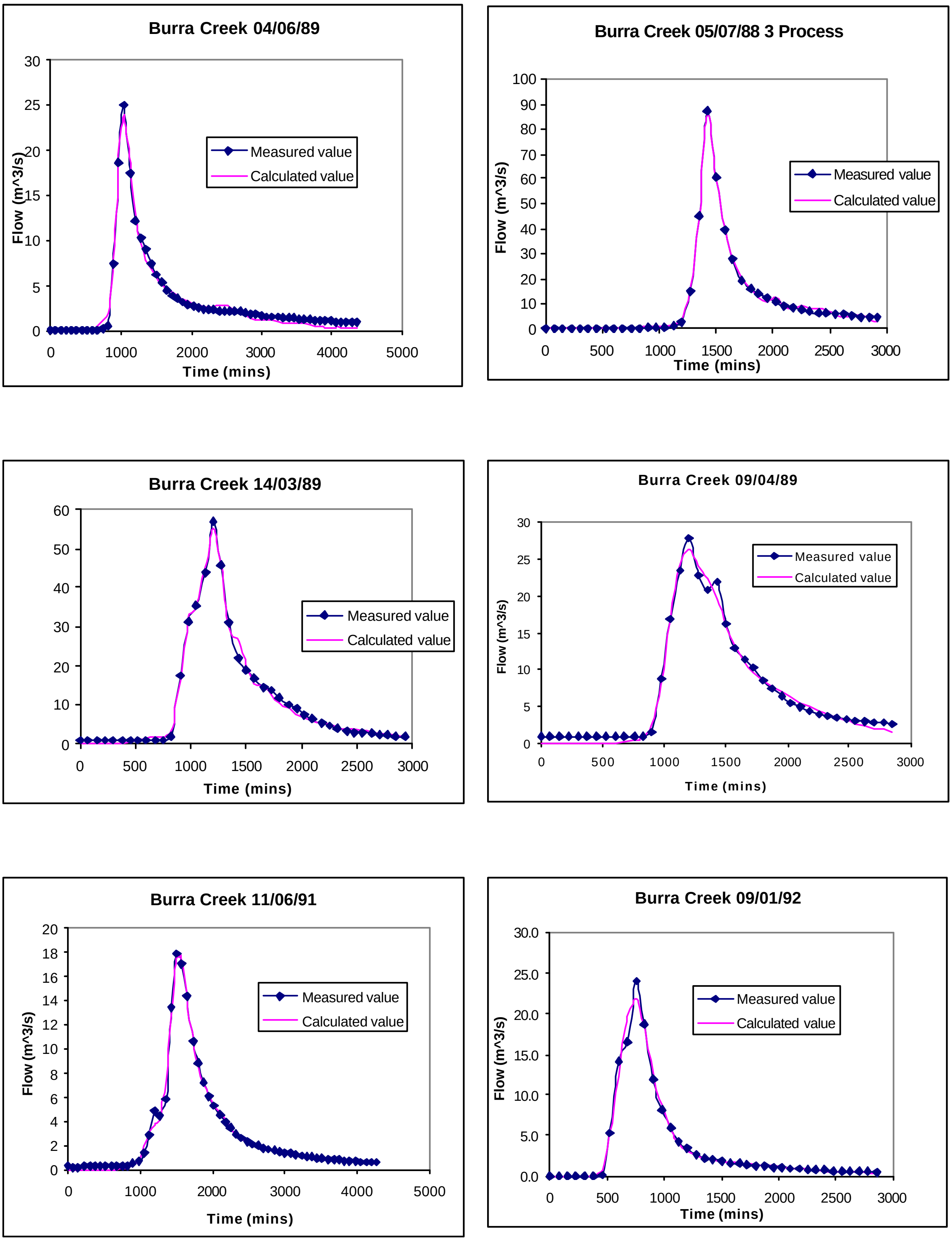


Figure 9-41 Burra Creek Catchment


Burra Creek 04/06/89
Flow (m^3/s)
Time (mins)
Measured value
Calculated value
Burra Creek 05/07/88 3 Process
Flow (m^3/s)
Time (mins)
Measured value
Calculated value
Burra Creek 14/03/89
Time (mins)
Measured value
Calculated value
Burra Creek 09/04/89
Flow (m^3/s)
Time (mins)
Measured value
Calculated value
Burra Creek 11/06/91
Flow (m^3/s)
Time (mins)
Measured value
Calculated value
Burra Creek 09/01/92
Flow (m^3/s)
Time (mins)
Measured value
Calculated value

(Figure 9-42)

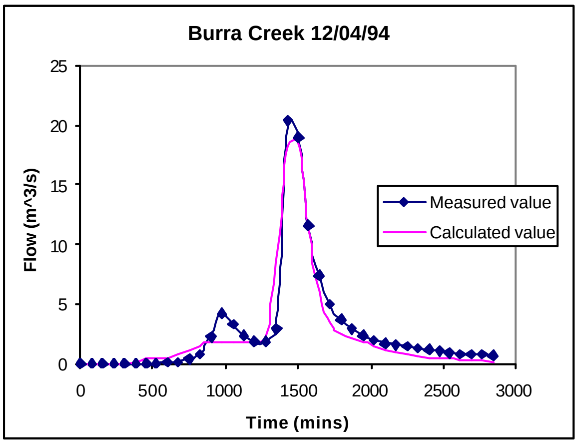


Figure 9-42 Burra Creek Calibration Hydrographs

Verification was carried out with weighted the mean parameter values in Table 9-28:

Table 9-28 Burra Creek Verification Parameters

IL1(mm)	PL1	IL2(mm)	PL2	IL3(mm)	PL3	k	k_p1	k_p2
0.0	0.86	25.2	0.73	66.1	0.64	0.191	1.92	0.470

The results are given in Table 9-29 and Figure 9-44.

Table 9-29 Burra Creek Verification Results

Date	Rainfall (mm)	Gauged peak flow (m^3/s)	Predicted peak flow (m^3/s)	Predicted peak /gauged peak	Gauged volume (m^3)	Predicted volume (m^3)	Predicted volume/ gauged volume
26/12/88	41.6	48.2	16.4	0.34	1 420 000	700 000	0.49
31/03/89	124.9	117.3	89.1	0.76	6 591 000	4 572 300	0.69
12/04/89	33.0	15.2	9.4	0.62	1 363 000	462 400	0.34
19/01/95	120.4	64.7	178.6	2.76	1 524 000	4 179 000	2.74
27/01/95	71.6	51.6	83.5	1.62	1 135 000	1 680 000	1.48
			mean	1.22			1.15

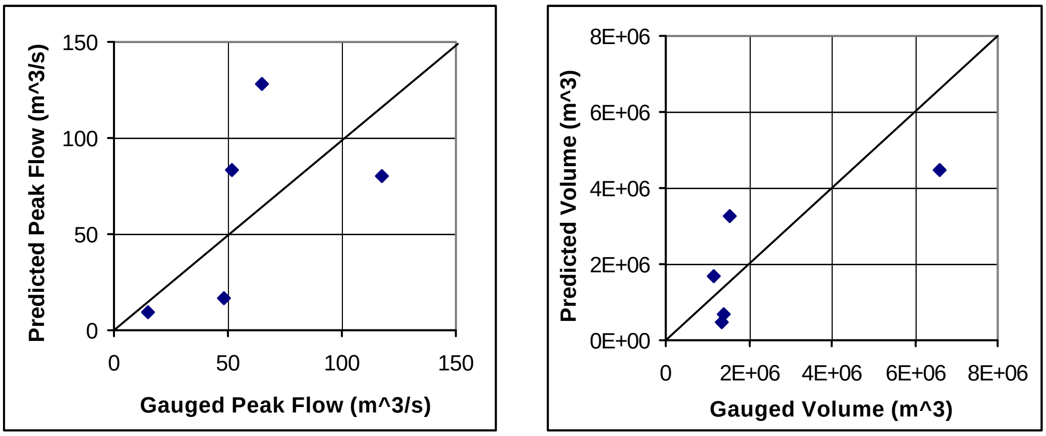


Figure 9-43 Burra Creek Verification Results

The event of 12/04/89 followed on directly after the event of 9/04/89. As another means of verification, the event of 12/04/89 was modelled using the calibrated parameters for 9/04/89, and assuming no IL2. The results are given in Table 9-30, and shown in Figure 9-45. It resulted in a good fit.

Table 9-30 Burra Creek Fit for 12/04/89 with Parameters From 9/09/89

Date	Gauged peak flow (m^3/s)	Predicted peak flow (m^3/s)	Predicted peak /gauged peak	Gauged volume (m^3)	Predicted volume (m^3)	Predicted volume/ gauged volume
12/04/89	15.2	15.6	1.03	1 363 000	1 300 000	0.95

The fit produced by the RRR model is generally not good, however as is the case in the other catchments the differences most probably are caused by the loss model, and the lack of definition of catchment rainfall by a single pluviometer. As it has an area of 70km^2 The Burra Creek catchment would be expected to have a large variation in rainfall across the catchment.

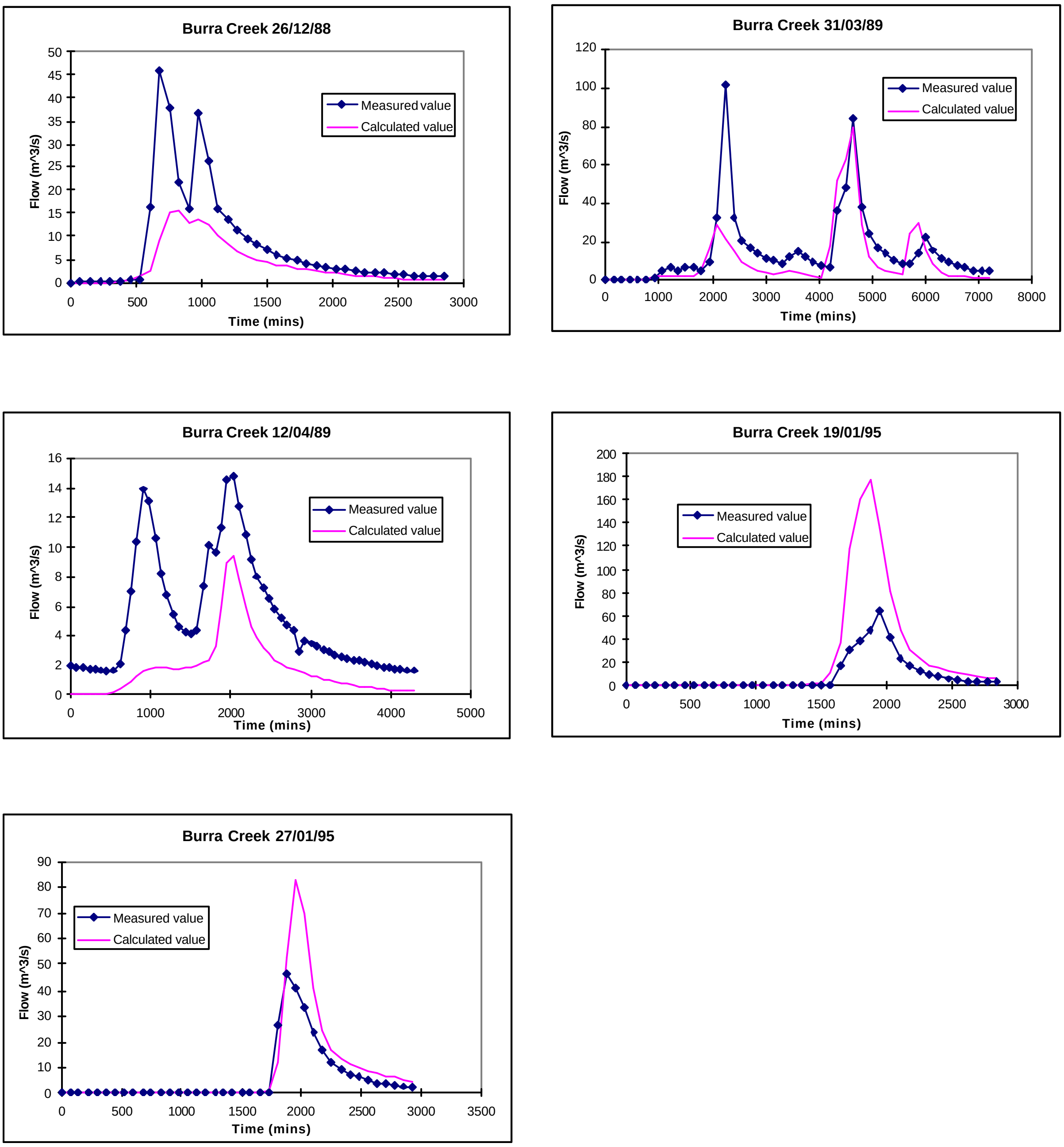


Figure 9-44 Burra Creek Verification Hydrographs

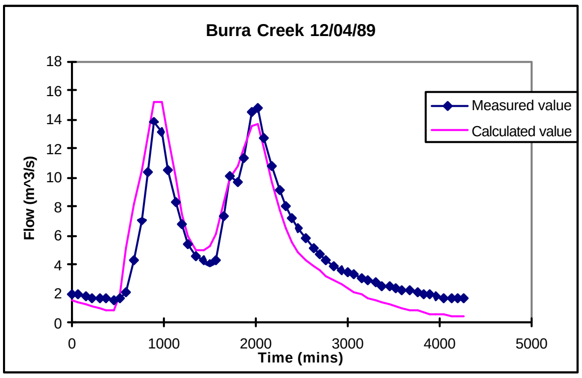


Figure 9-45 Burra Creek Verification 12/04/89 With Parameters from 09/04/89

9.3.10 Comparison With KINDOG and RORB

The KINDOG model (Kuczera, 2000) is a model that includes both baseflow and surface flow. Routing from hillsides is by linear (baseflow) and non-linear (surface flow) storages, and channels are modelled by kinematic wave. That catchment is subdivided like the RORB and WBNM model. The RORB model is a standard runoff routing model, which includes only one process.

For the verification using KINDOG and RORB the Inverbrackie Creek catchment was chosen.

Five parameters in KINDOG were used in calibration, being initial loss, continuing loss, Cg, Cs and Cr. The exponents γ and m were not used in calibration, as was recommended in the KINDOG notes (Kuczera, 2000). As these exponents were not calibrated in RRR this approach leads to a reasonable comparison.

Calibration was carried out in the case of RORB by the PEST parameter estimation program, applied to the models set up in the XP-RAFTS format. Unlike normal RORB calibration the initial and continuing losses were not linked to ensure continuity, they were adjusted in combination with the kc value to give the best overall hydrograph fit. This ensured that a similar objective function was used to fit all models.

In the case of the RORB model, baseflow was separated with a recursive digital filter. The baseflow was extracted using a recursive digital filter as described by Lyne (1979). This filter is built into the HYDSYS hydrological data archiving program. It has been widely used and accepted in Australia. Compared to graphical methods it is objective and reproducible.

Digital filters are used in signal processing and analysis. They can be used to separate an unwanted signal from a set of data. The digital filter has two main characteristics. The first of these is gain, which is the factor by which the original signal is multiplied by when passing through the filter. The second characteristic of the digital filter is that it can produce a shift in phase (Daniell and Hill, 1993).

The filter separates the total hydrograph into two components. The two components are assumed to be baseflow and the quick flow that is modelled by the runoff routing model.

The filter has the form of the equation;

$$f_k = a.f_{k-1} + \frac{(1+a)}{2}(y_k - y_{k-1}) \qquad \text{Equation 9.7}$$

where f_k is the filtered quick response at the k^{th} sampling instant

y_k is the total streamflow; and

a is the filter parameter

Two restrictions placed on the digital filter are that the separated streamflow is not negative or greater than the original streamflow.

After the forward pass filtering a reverse pass is applied to the slow flow starting from the end of the data. This is to nullify any phase distortions. It is also suggested that the data be passed through the filter again if the separated slow flow appears to be responding too quickly. The choice of filter parameter was not a critical factor.

Lyne and Hollick (1979) also discussed the use of a recursive digital filter. The filter and constraints were the same as those discussed by Lyne (1979). It is suggested that using a filter parameter between 0.75 and 0.9 can achieve a good separation of components. It was also recommended that a reverse pass filter be applied to nullify any phase distortions.

Application of the technique has also been discussed by O'Loughlin et al (1982) and Nathan and McMahon (1989). The recommended value of the filter parameter was 0.925.

The recursive digital filter was used to separate baseflow, using a filter parameter of 0.925, and three passes, as is used by default in HYBASE, the separation program within HYDSYS. The filter was incorporated into a spreadsheet to perform the separation on exactly the same data points that were used in the calibration of the RRR models.

A proportional loss model was used with the RORB model, as it was found that this model gave the best result

The same methodology was used for each model, being the calibration on six events, the selection of mean parameter values, and the application of these values to six independent events as verification.

Table 9-31 summarises the fit achieved by KINDOG on the six events used for calibration. The best fit is shown in bold.

Table 9-31 Comparison of RRR and KINDOG Calibration

Date	Mean Error (m3/s) RRR	Mean Error (m3/s) RORB	Mean Error (m3/s) KINDOG
14/07/87	**0.204**	0.327	0.348
05/06/88	**0.147**	0.295	0.256
04/07/90	0.164	0.159	**0.099**
19/08/90	**0.094**	0.223	0.210
29/08/92	**0.524**	1.357	1.654
28/09/96	**0.266**	0.441	0.344

The RRR model provided a better fit in all but one event. The RORB model had the greatest mean error, but this may be expected as the RORB model has less parameters, and models only one process. Figure 9-46 shows the calibration results.

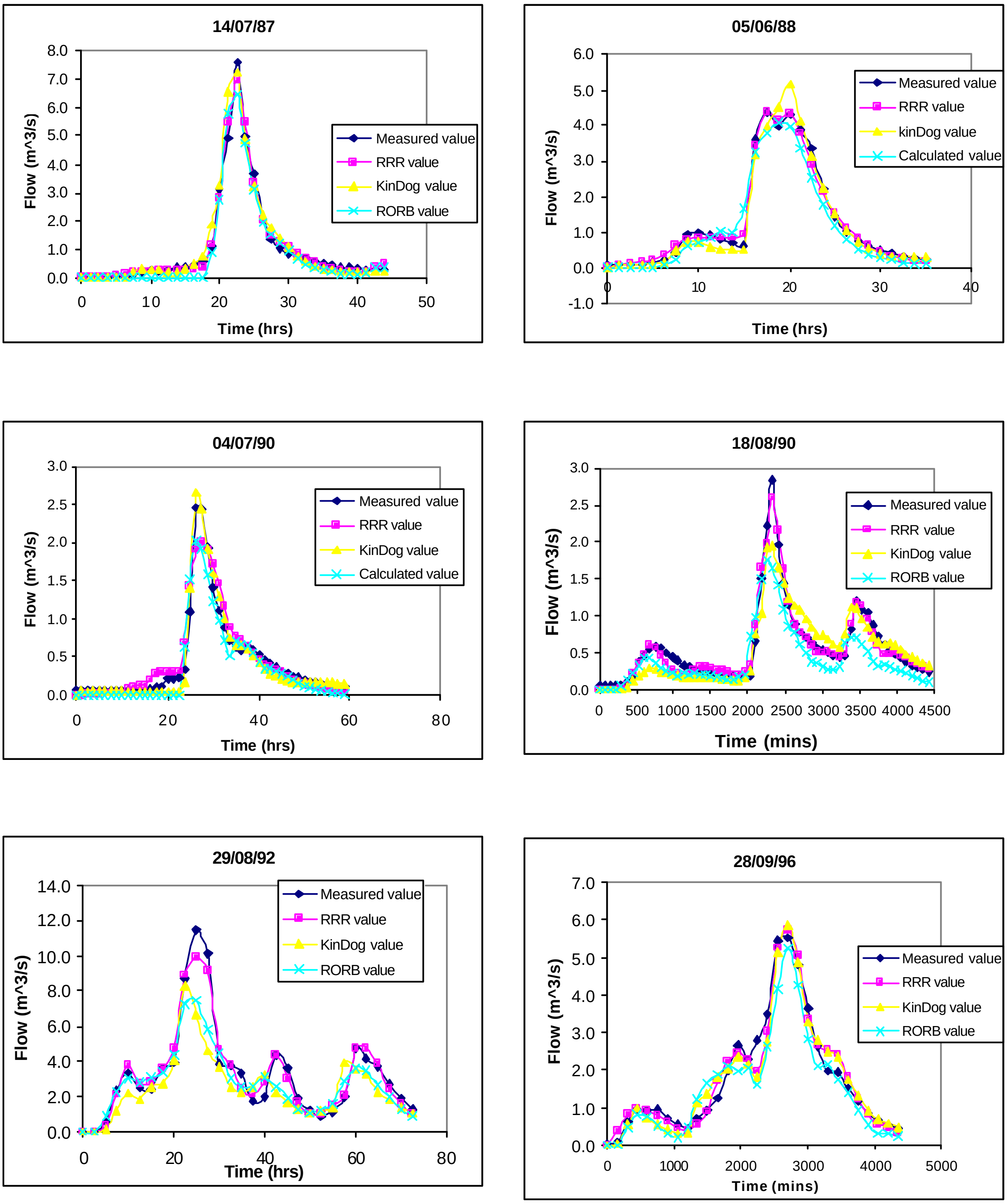


Figure 9-46 Inverbrackie Creek KINDOG and RORB Calibration Results

Table 9-32 gives the calibrated parameters for the KINDOG model.

Table 9-32 Calibration Parameters for the KINDOG Model

Date	Cs	Cg	IL (mm)	CL (mm/hr)	Cr
14/07/87	0.539	784	9.5	0.575	9.18
05/06/88	0.623	1041	0.5	0.925	7.02
04/07/90	0.714	1262	4.5	0.625	3.96
19/08/90	0.728	1190	0	0.050	5.82
29/08/92	0.714	839	9.0	0	9.24
28/09/96	0.616	1315	6.5	0.100	6.48

A relationship, shown on Figure 9-47 was found between the event API and initial loss for the calibration events. The relationship was applied to the verification events. For other parameters mean values were used as follows;

Cs = 0.65

Cg = 1258

Continuing Loss = 0.46mm/hr

Cr = 6.63

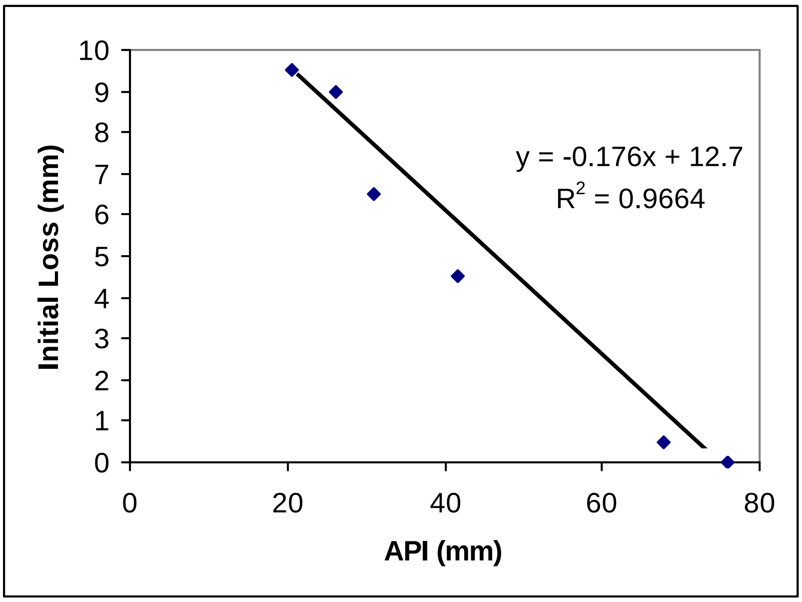


Figure 9-47 KINDOG API - Initial Loss Relationship

The calibrated parameters for RORB were as follows:

k_c = 5.0

IL = 12.75mm

Proportional loss = 0.33

Verification was carried out on the six independent events.

A problem arises with RORB that the model cannot predict baseflow, and thus a comparison cannot be made directly. To make some comparison two methods were used, first by adding the mean baseflow as determined during the calibration (1 in Table 9-33), and second by adding the baseflow separated from the total measured hydrograph, to give a total predicted flow (2 in Table 9-33). Figure 9-48 shows RORB with the addition of the mean baseflow ($0.28 m^3/sec$).

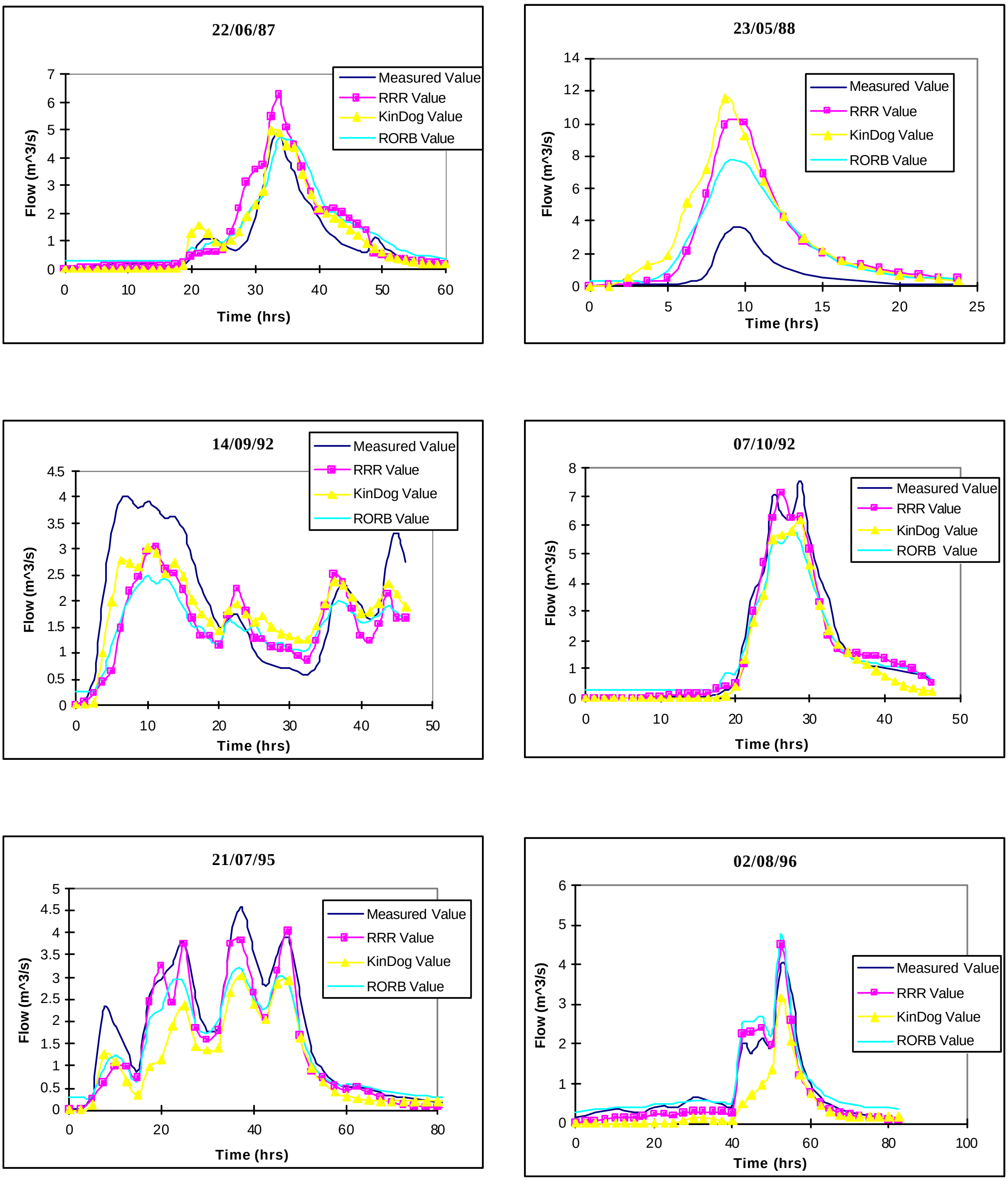


Figure 9-48 Inverbrackie Creek RRR, KINDOG and RORB Verification Results

Table 9-33 Summary of RRR, KINDOG and RORB Verification

Date	Mean Error (m3/s) RORB (1)	Mean Error (m3/s) RORB (2)	Mean Error (m^3/s) RRR	Mean Error (m^3/s) KINDOG
22/06/87	0.521	0.574	0.682	**0.394**
23/05/88	2.132	**1.970**	2.830	3.274
07/10/92	0.602	0.609	**0.442**	0.551
14/09/92	0.998	1.061	0.962	**0.710**
21/07/95	0.557	0.523	**0.501**	0.813
02/08/96	0.378	0.386	**0.269**	0.543

The RRR model performed best in three of the six events. For the other three events kinDog performed better in two and RORB in one.

It is also interesting to compare the predicted peak flows from the different models, as the peak flow is most often used for floodplain mapping, or the design of structures. The RRR model predicted the closest to the measured peak in four of the six events, with the other two being the RORB model with the baseflow extraction and kinDog.

Table 9-34 Peak Flow Verification Summary

Date	Measured Peak Flow (m3/s)	Peak Flow (m3/s) RORB (1)	Peak Flow (m3/s) RORB (2)	Peak Flow (m3/s) kinDog	Peak Flow (m3/s) RRR
22/06/87	4.95	4.74	4.81	**5.01**	6.32
23/05/88	3.53	7.52	**7.27**	11.56	10.56
07/10/92	7.50	5.80	5.82	6.18	**7.20**
14/09/92	3.98	2.48	2.73	3.02	**3.09**
21/07/95	4.58	3.20	3.45	3.02	**4.31**
02/08/96	3.99	3.34	3.36	3.16	**4.45**

9.3.11 The Influence of Model Complexity

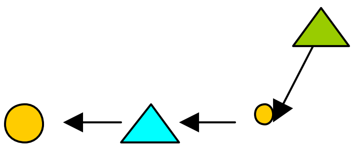
The RRR model has been developed with ten channel reaches and a series of ten storages to represent the hillside processes. This was on the basis of Laurenson (1964), who used a series of ten storages, and because the XP-RAFTS interface was used to run the model. RAFTS uses a series of ten storages, as it is based on Laurenson's model.

To support the use of ten channel reaches it was decided to investigate the performance of a series of models like RRR, but having less channel reaches.

The simplest model possible has one process storage and one channel storage. Inflow to the channel is assumed to occur at a dsitance of half the longest flow path length in the catchment. This will be termed model 1. The next simplest structure (model 2) has two inflows to the channel, at the downstream end of the channel reaches, as per RRR. Two process storages are also used, so that a better representation of the distributed nature of storage on the hillside is possible. Model 2 can be expanded by increasing the number of channel reaches and process storages . Three and five channel reaches were examined, and these are termed model 3 and model 5. The RRR model is equivalent to model 10.

Inverbrackie Creek was chosen for the calibration and verification of all models, using the same procedure as the RRR model. A direct comparison can then be made between all models.

Figure 9-49 to Figure 9-51 shows the models. In the figures only one set of process storages is shown contibuting to the channel. However in the calibration and verification of the models two sets of process storages were used, contributing to each channel input location.



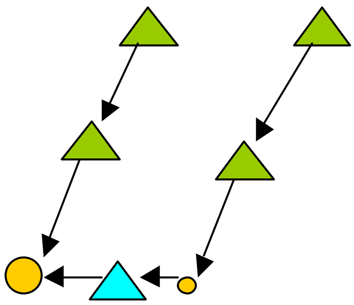


Figure 9-49 Model 1 (left) and Model 2

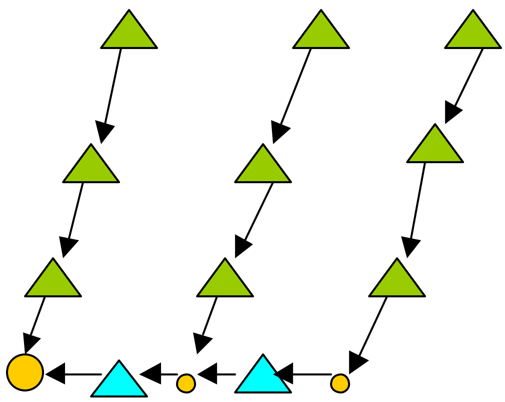


Figure 9-50 Model 3

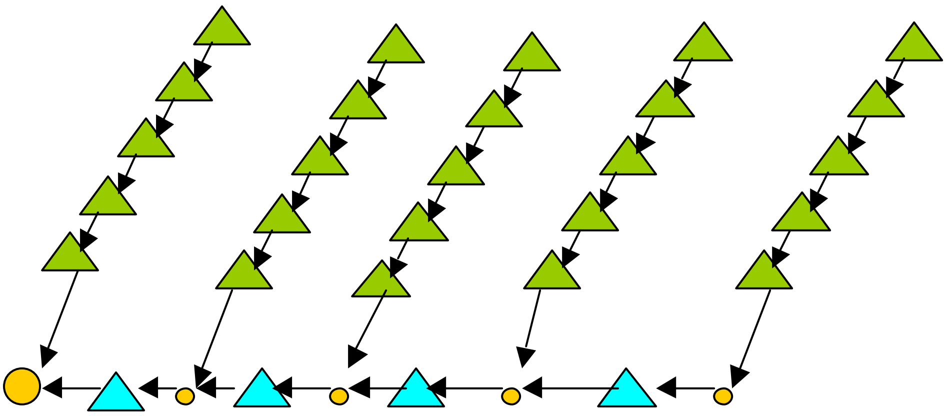


Figure 9-51 Model 5

All models were calibrated using PEST, and the mean error for each model and storm event shown on Table 9-35. The lowest error is shown in bold type.

Table 9-35 Mean Errors for Each Storm and Model

Model	05/06/1988	04/07/1990	14/07/1987	19/08/1990	28/09/1996	29/08/1992
1	0.171	**0.118**	0.266	0.162	0.288	0.680
2	0.226	0.213	0.381	0.219	0.336	0.873
3	0.195	0.248	0.275	0.188	0.302	0.747
5	0.163	0.151	0.229	0.160	0.280	0.596
10	**0.147**	0.164	**0.204**	**0.094**	**0.276**	**0.488**

It is of interest that model 10 (the RRR model) shows the best performance, but the simplest possible model (model 1) performs better in some cases than more complex models. The range of errors is not large.

The weighted mean parameter values are given in Table 9-36. As expected as more storages are introduced into the model the storage parameters kp_1, kp_2 and k reduce. The loss parameters remain reasonably consistent.

Table 9-36 Model Mean Parameter Values

Model	kp1	kp2	k	IL2	PL1	PL2
1	5.558	2.386	1.702	18.716	0.748	0.436
2	3.494	2.039	1.243	17.111	0.733	0.402
3	2.543	1.310	0.695	18.281	0.774	0.400
5	2.164	0.700	0.470	18.189	0.747	0.399
10	1.249	0.310	0.194	16.327	0.746	0.459

The mean parameter values for each model were then applied to the set of independent storm events, and mean errors calculated. Table 9-37 gives the results of the verification, with the lowest error shown in bold.

Table 9-37 Verification Mean Errors

Model	07/10/1992	14/09/1992	22/06/1987	21/07/1995	23/08/1988	02/08/1996
1	0.829	1.095	0.828	0.892	2.846	0.660
2	**0.346**	**0.908**	0.670	**0.486**	2.716	0.337
3	1.045	0.547	**0.547**	0.541	2.463	0.309
5	1.077	1.077	0.599	0.500	**2.452**	**0.261**
10	0.963	0.963	0.682	0.501	2.830	0.269

Since runoff routing models are often used in the estimation of peak flows it is also useful to examine the prediction of peak flows by all the models. Table 9-38 shows the results, with the closest to the actual peak in bold type.

Table 9-38 Verification Peak Flows

Model	07/10/1992	13/09/1992	18/06/1987	17/07/1995	23/08/1988	02/08/1996
1	6.30	2.57	**5.59**	3.82	**8.56**	3.93
2	6.75	2.91	6.01	4.06	9.28	**4.07**
3	6.46	2.58	5.73	3.89	8.87	3.96
5	6.39	2.63	5.69	3.92	8.81	4.00
10	**7.20**	**3.09**	6.32	**4.31**	10.56	4.45
Gauged	7.69	4.01	5.20	4.60	3.89	4.07

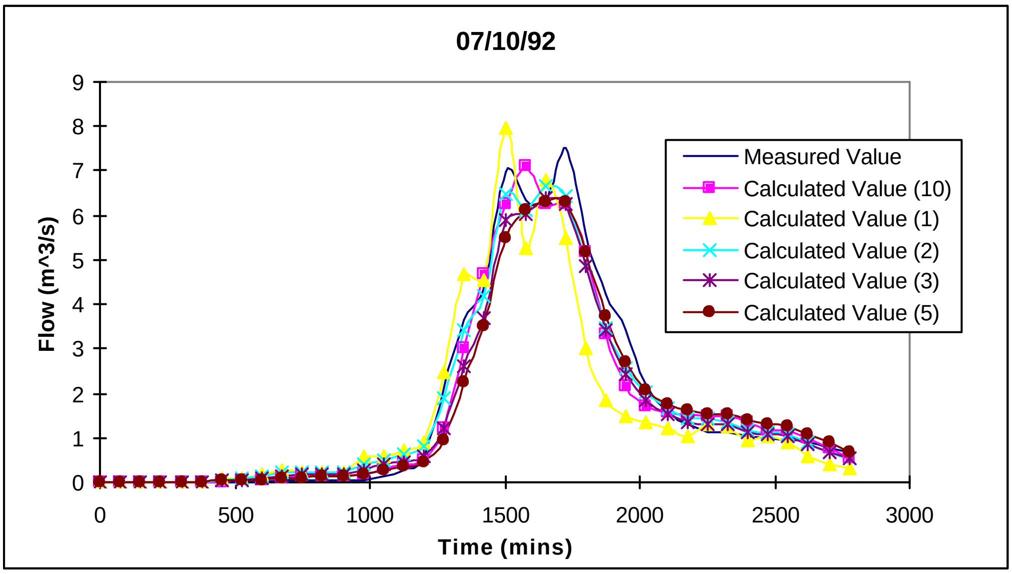


Figure 9-52 Event 7/10/92 - Effect of Model Complexity

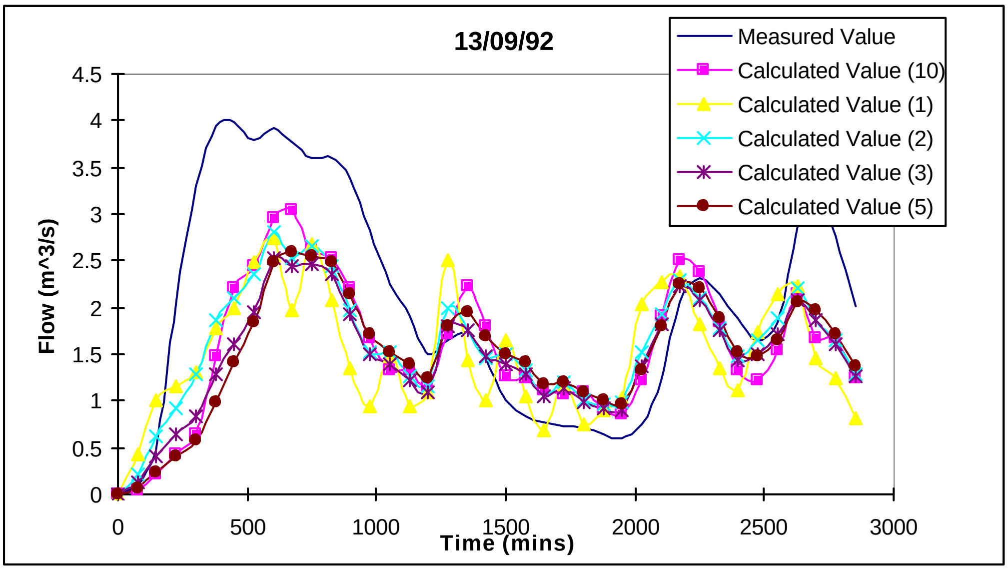


Figure 9-53 Event 13/09/92 - Effect of Model Complexity

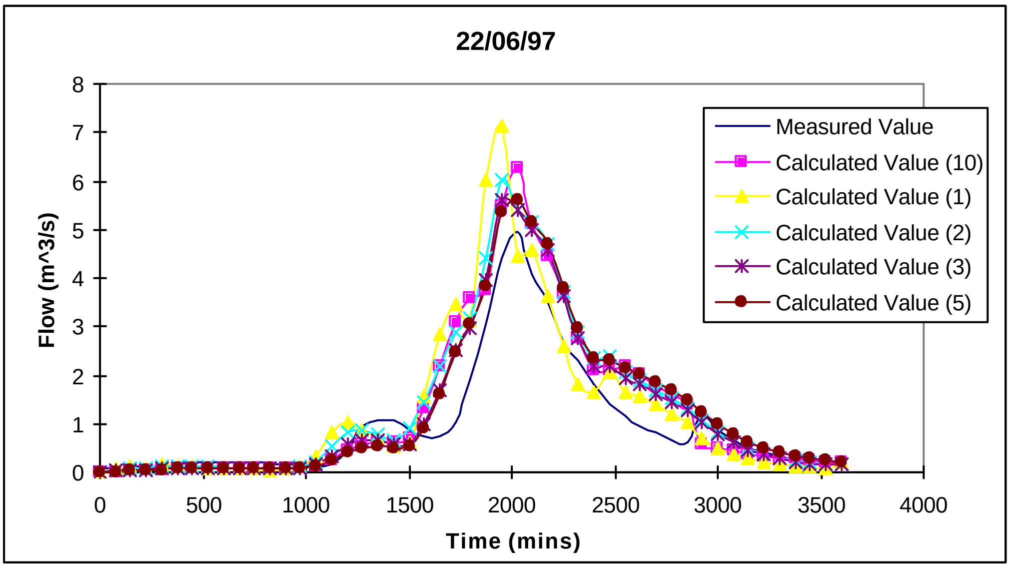


Figure 9-54 Event 22/06/87 - Effect of Model Complexity

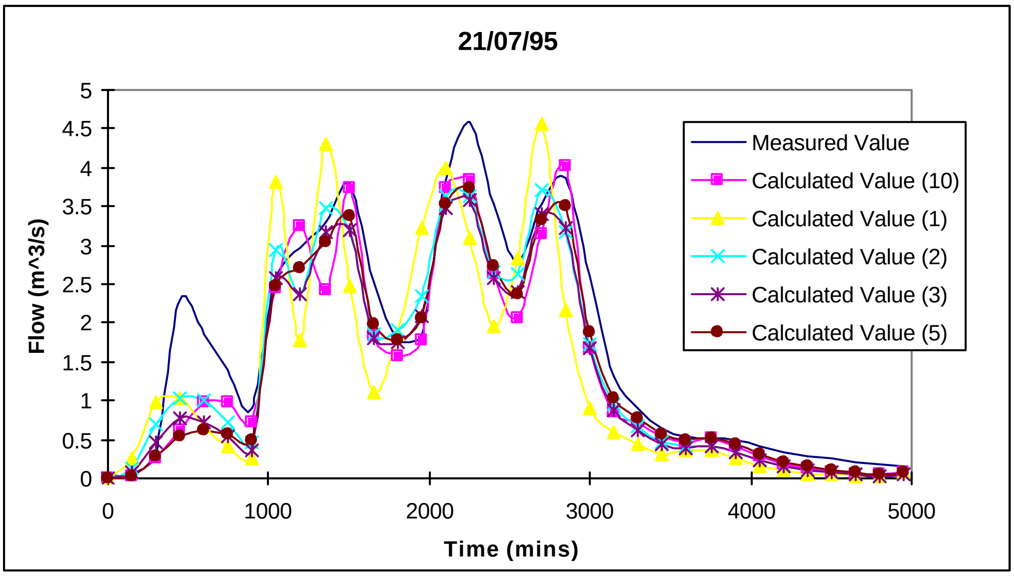


Figure 9-55 Event 21/07/95 - Effect of Model Complexity

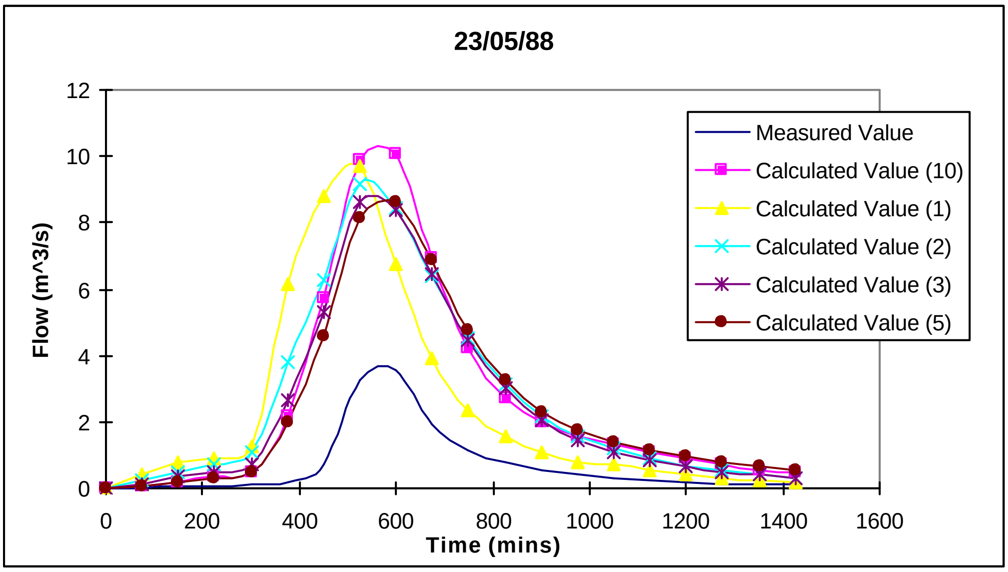


Figure 9-56 Event 23/05/88 - Effect of Model Complexity

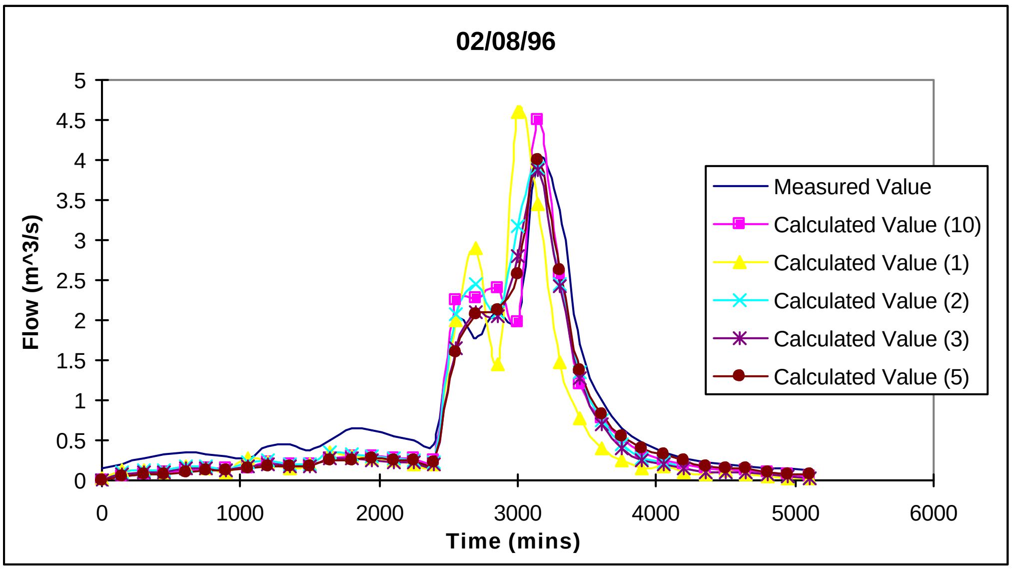


Figure 9-57 Event 02/08/96 - Effect of Model Complexity

The result is surprising in that there is no one model that consistently performs better than the others. The RRR model could be fitted better to gauged events, but in general is no better at prediction as indicated by the verification events. The RRR model shows the best fits during calibration, and can therefore be considered to be the best representation of the catchment in a mathematical form. However if the model is being used in the prediction of design flows a simpler model, with less storages may be adequate.

9.3.12 A Spreadsheet Model (KSSM)

The simplest model having one storage for each process and one channel storage has been developed as a spreadsheet model in Excel. Two processes only are assumed to occur. Three worksheets are used. The first contains the parameters, and a plot of the calculated hydrograph, and the gauged hydrograph, if available. The second contains the data entry and hydrograph calculation. The third worksheet does the non-linear runoff routing.

The parameter entry is shown as Figure 9-58. Included in this are suggested parameter values and a value of the sum of the square error between the predicted and gauged hydrograph ordinates. This is used for calibration, which can be done manually or by using the solver built into the spreadsheet. This solver is set up to minimise the sum of square errors, whilst maintaining the parameter values between reasonable limits.

KEMP'S SPREADSHEET MODEL

Catchment	Inverbrackie Creek
Start Time	06/05/1988 6:30

Catchment Loss Parameters		**Catchment Storage Parameters**		**Suggested Value**
IL Base Flow (mm)	0.0	Base Flow Lag Parameter	6.01	10.94
PL Base Flow	0.73	Slow Flow Lag Parameter	1.55	1.53
		Channel Lag Parameter (hrs)	1.42	0.85
IL Slow Flow (mm)	20.1			
PL Slow Flow	0.66	m	0.8	
Catchment Area (km^2)	8.4	**Sum of Square Errors**	3.142813	
		Time Step (hrs)	0.5	

Figure 9-58 Sample Parameter Entry for the Spreadsheet Model

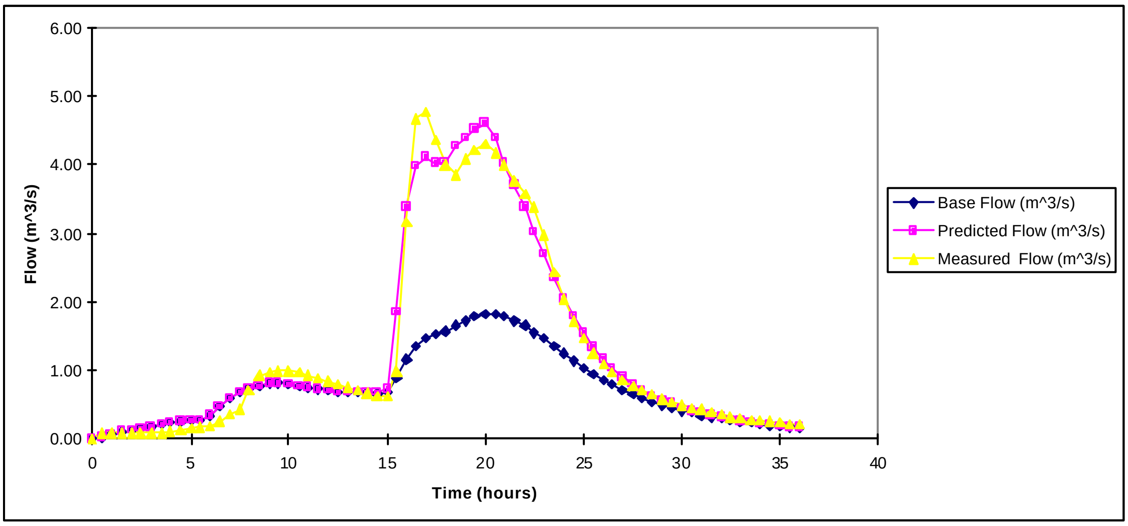


Figure 9-59 Sample Plotted Hydrographs from the Spreadsheet Model

The main advantage of the spreadsheet model is the ease of calibration. If calibration is done manually by altering the parameter values the hydrograph is replotted immediately any parameter is changed. The calibration using the solver is also straightforward and gives good results.

9.4 Summary of RRR Verification

The RRR model has been successfully applied to a variety of catchments. It was applied to three urban catchments in Adelaide, and six rural catchments. For four of the rural catchments rainfall information was available from only one pluviometer. This has meant that there are inaccuracies in the spatial distribution of the rainfall, both for calibration and verification.

For urban catchments in South Australia there has been a verification of the storage parameters of the model. Runoff from areas other than those directly connected to the drainage system was not common.

For the Jamison Park catchment in Sydney a loss model was derived from the calibration of the model's loss parameters and applied to the data set with acceptable results. This is not a true verification of the loss model, as the loss model would have to be applied to an independent set of data.

The RRR model in urban areas gives a similar level of performance to that of the ILSAX model. However the RRR model structure and function is much simpler. It is much easier to apply to large catchments, where flows in individual elements of the pipe or channel system are not required.

Verification was carried out on six rural catchments. The results in terms of the prediction of peak flows has been erratic, for example the ratios of predicted to actual flows for the verification events on Echunga Creek ranged from 0.26 to 1.84. The mean ratio for Scott Creek was 0.56, but this improved to 0.90 when rainfall information from a second pluviometer was included. If more rainfall data had been used both for calibration and verification on all catchments then the results would be expected to have been better.

However by visual inspection the general shape of the predicted hydrographs and the time to peak is good. A comparison with the performance of single sub-catchment RRR model on the Inverbrackie Creek catchment with RORB a more complicated model (KINDOG) has also shown that RRR performs better than similar models. Reducing the complexity of the RRR model by reducing the number of storages also reduces the level of fit that can be achieved during calibration runs, but this may not necessarily provide worse model performance in the prediction of hydrographs.

In most cases during calibration two processes were found to be occurring on the catchments examined, being baseflow and slow flow, being the flow component usually modeled by runoff routing models such as RORB, WBNM and RAFTS.

10. RRR Model Parameters and Catchment Characteristics

10.1 Introduction

One of the main objectives of this investigation was the examination of catchment lag and the relationship of this with the runoff processes. The use of a runoff routing model that includes more than one process should give a better indication of the factors that determine catchment response.

An investigation was performed to determine a relationship between RRR parameters and catchment characteristics in the Mount Lofty Ranges, and in particular the upper Onkaparinga River catchment, which was the focus of an investigation for the Onkaparinga Catchment Water Management Board. This investigation is still in progress.

The generalised storage parameters c_p1, c_p2 and v_c, and the initial and proportional losses are used in this study for comparison between catchments.

10.2 Mount Lofty Ranges Catchments Calibrations

The RRR model was calibrated on catchments where there were sufficient pluviometer and gauging data available.

The calibration of the RRR model was carried out using the parameter estimation program PEST. In all cases it was assumed that baseflow was occurring at the start of the event. For each catchment the largest six to seven storm events for the period of record were selected for calibration by PEST.

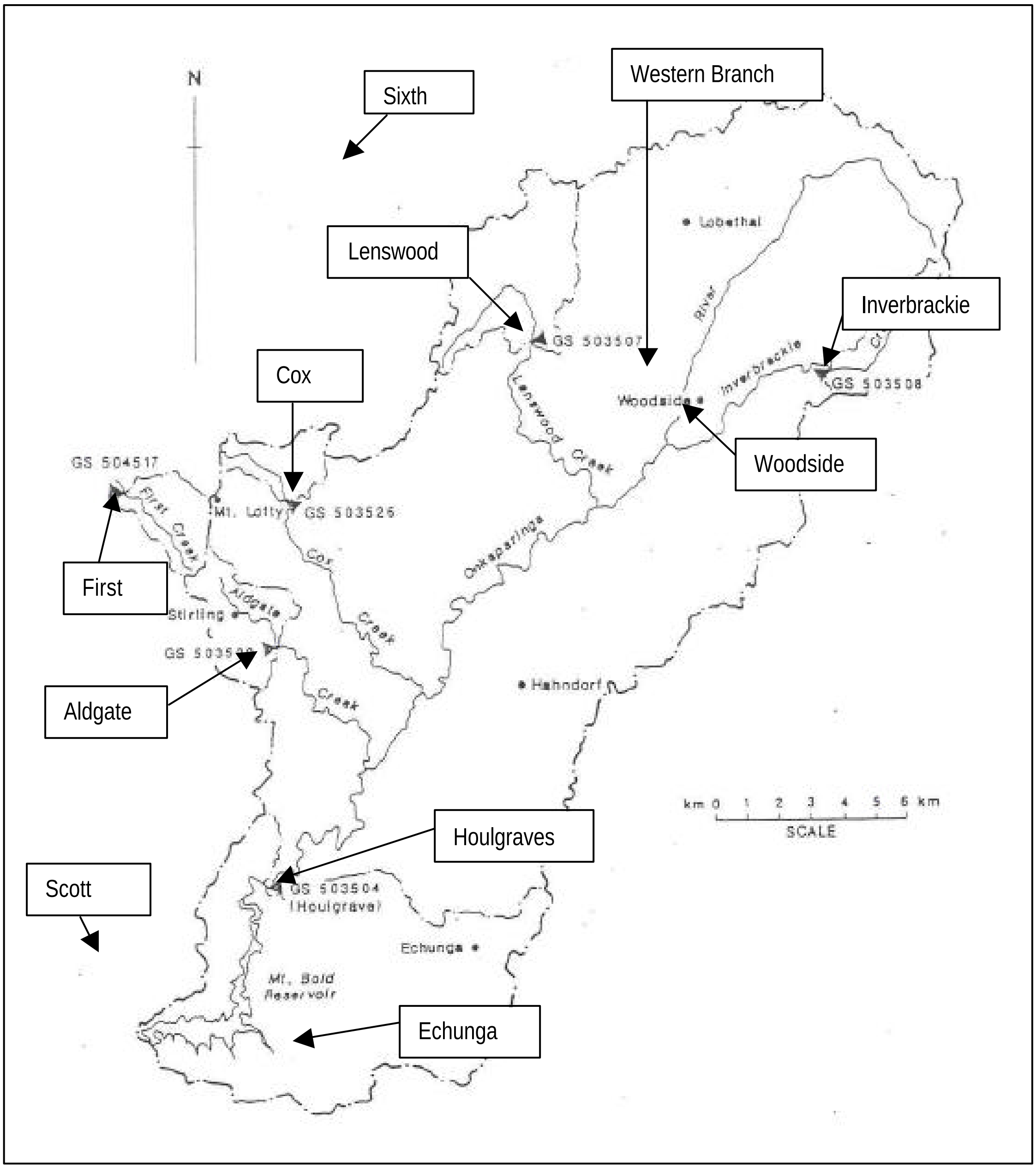


Figure 10-1 Mount Lofty Ranges Catchments

10.2.1 Cox Creek

The Cox Creek catchment has a catchment area of 4.3km^2. It is located in the higher rainfall portion of the Onkaparinga River catchment, with an annual rainfall of approximately 1090mm/annum (Uraidla). Land use is dominated by horticulture, particularly viticulture. The underlying rock is predominantly sandstone.

Six events were chosen for the calibration of the model for this catchment. Flow data is for the Cox Creek station (AW503526). The station consists of a stable, regular profile weir (using precast caps) to gauge height 1.62 metres, (3.24m^3/sec). One rating relationship exists covering the whole period of record comprising 56 gaugings to a flow of 2.97m^3/sec. A theoretical extension has been made to 14.3m^3/sec. Rainfall data at either Vince (AW503524) or Sutton (AW503525) stations was used, depending on availability. Table 10-1 gives the results of the calibration.

Table 10-1 Cox Creek RRR Calibration Results

Event Start Date	PL1	IL2 (mm)	PL2	k	k_p1	k_p2
24/08/1983	0.82	8.80	0.80	0.036	0.589	0.0376
07/09/1983	0.79	3.39	0.74	0.133	0.396	0.0108
16/08/1984	0.90	6.05	0.80	0.089	0.524	0.1125
01/07/1986	0.77	0	0.84	0.072	1.354	0.0572
01/08/1986	0.74	10.3	0.75	0.093	1.073	0.0884
23/06/1987	0.73	0	0.58	0.308	0.874	0.0361
Mean	0.82	5.58	0.76	0.112	0.676	0.0660

10.2.2 Lenswood Creek

The Lenswood Creek catchment has an area of 16.5km^2, and an annual average rainfall of 1030mm (Lenswood). The catchment land use is mainly horticulture, but a substantial amount of native vegetation remains. The predominant rock types are siltstones and shales.

Six events were chosen for the calibration of the model. Flow data at the Lenswood Creek gauging station (AW503507) was used, along with rainfall data from the Stringybark pluviometer (BM023865), which is located on the northern boundary of the catchment. This pluviometer was chosen as it had a longer period of record than the Lenswood Creek pluviometer (AW503507), and it is situated such that it probably best represents catchment rainfall. The gauging station consists of a concrete V crump weir installed to replace a previous natural, control created by tree log in stream channel. One rating relationship exists for the record since installation of the V crump weir (1978), comprising 63 gaugings to a flow of 8.24m^3/sec. A theoretical extension has been made to 50m^3/sec.

Table 10-2 Lenswood Creek RRR Calibration Results

Event Start Date	PL1	IL2 (mm)	PL2	k	k_p1	k_p2
02/07/1995	0.73	6.38	0.65	0.066	2.263	0.368
21/07/1995	0.53	4.32	0.66	0.180	1.418	0.172
03/08/1996	0.47	4.22	0.36	0.141	2.190	0.445
28/09/1996	0.75	38.6	0	0.126	1.564	0.391
27/07/1998	0.84	26.5	0.73	0.187	1.975	0.231
07/09/2000	0.61	12.1	0.46	0.111	3.304	0.48
Mean	0.68	17.28	0.58	0.131	2.134	0.357

10.2.3 Aldgate Creek

The Aldgate Creek catchment is situated in the high rainfall area of the Onkaparinga catchment, with an average annual rainfall of 1190mm (Stirling). The catchment has a substantial amount of residential and commercial development within it, and for this reason the catchment was modelled with the assumption of 10% impervious area, based on an inspection of the catchment and planning zones. A previous study (Kinhill Engineers, 1993) assessed the impervious percentage as 9.1%. The impervious area was assumed to have an initial loss of 1mm, and zero continuing loss.

Seven storm events were modelled. Only two of these (22/05/1999 and 07/09/2000) had rainfall data available from a pluviometer at the gauging station (AW503509). For the other events pluviometer data from Mount Lofty was used (AW504552). As this station is outside the catchment and in an area having steep rainfall gradients is was expected that it would be more difficult achieving a reasonable fit for most events.

At the gauging station an irregular weir profile possibly contributes to some scatter in stage – discharge relationship between 0.4 and 4m^3/sec. One rating relationship exists covering the whole period of record comprising 102 gaugings to a flow of 8.2m^3/sec. A theoretical extension has been made to 26m^3/sec.

Table 10-3 gives the result of the calibrations.

Table 10-3 Aldgate Creek RRR Calibration Results

Event Start Date	PL1	IL2 (mm)	PL2	IL3 (mm)	PL3	k	kp1	kp2
07/09/2000	0.80	20.16	0.52	#	#	0.259	2.634	0.207
22/05/1999	0.73	18.13	0.87	#	#	0.242	5.712	0.129
22/09/1998	0.91	5.07	0.90	#	#	0.254	2.600	0.111
01/08/1995	0.63	0.00	0.68	#	#	0.190	0.823	0.152
22/05/1988	#	31.59	0.24	#	#	0.200	#	0.236
21/06/1987	#	20.27	0.45	#	#	0.062	#	0.284
01/07/1986	0.79	26.07	0.81	85.0	0.63	0.585	3.007	0.106
Mean	0.75	15.63	0.60			0.235	2.425	0.180

Note #: No contribution was found from this process.

10.2.4 Western Branch

The catchment to the Western Branch gauging station (AW503906) includes the town of Lobethal and it has a catchment area of 24.2km^2. Average annual rainfall is approximately 890mm (Lobethal). The major rock type within the catchment is quartzite.

Six events from the period of record were chosen for calibration, using pluviometer data from the Lobethal station (BM023862). The Lobethal station was chosen as it was closest to the catchment centroid. Table 10-4 gives the results of the calibration. Baseflow was present in only one event modelled.

Table 10-4 Western Branch RRR Calibration Results

Event Start Date	PL1	IL2 (mm)	PL2	k	kp1	kp2
03/08/1996	0.90	4.28	0.66	0.277	1.225	0.298
28/09/1996	#	14.3	0.63	0.345	#	0.462
27/07/1998	#	28.85	0.78	0.317	#	0.387
07/08/1999	#	19	0.82	0.264	#	0.400
15/09/1999	#	25.13	0.78	0.249	#	0.374
07/09/2000	#	9.16	0.70	0.299	#	0.413
Mean	0.90	18.04	0.73	0.292	1.225	0.391

Note #: No contribution was found from this process.

10.2.5 Woodside Weir

Six events were chosen for calibration at the Woodside Weir on the Onkaparinga River (AW503903). The station consists of a low profile concrete V notch weir. The catchment area to this point is 51.9km^2, and the average annual rainfall is 812mm (Woodside). Rainfall data from the Lobethal pluviometer (BM023862) was used, as this pluviometer is closer to the catchment centroid than the pluviometer at AW503903.

Table 10-5 gives the result of the calibration. Baseflow was present in two of the six events. In these two events the relative contribution of the second contribution (slow flow) was much greater.

Table 10-5 Woodside Weir RRR Calibration Results

Event Start Date	PL1	IL2 (mm)	PL2	k	kp1	kp2
21/07/1995	0.92	9.12	0.62	0.328	2.933	0.479
03/08/1996	#	7.58	0.59	0.109	#	0.358
26/08/1996	0.80	11.32	0.36	0.267	1.536	0.387
28/09/1996	#	13.65	0.52	0.309	#	0.556
27/07/1998	#	21.73	0.79	0.383	#	0.465
07/09/2000	#	5.96	0.68	0.382	#	0.672
Mean	0.85	13.39	0.68	0.347	2.092	0.567

Note #: No contribution was found from this process.

10.2.6 First Creek

The First Creek catchment is situated in the hills face zone of the Mount Lofty Ranges, to the east of Adelaide. It is a steep catchment, and is substantially in natural condition, with most of the catchment being contained within the Cleland Conservation Park. It has a catchment area of 4.89km^2. The underlying rock is mainly quartzite.

Rainfall data from a gauge at the upper end of the catchment was used (AW504552).

Baseflow was present in all modelled events, but there was no evidence of fast runoff.

Table 10-6 First Creek RRR Calibration Results

Event Start Date	PL1	IL2 (mm)	PL2	k	kp1	kp2
30/06/1986	0.75	91.6	0.88	0.390	2.466	0.480
01/08/1986	0.65	30.6	0.74	0.891	3.594	0.656
21/06/1987	0.73	28.6	0.89	0.136	5.954	0.587
14/07/1987	0.53	19.47	0.93	0.026	4.524	0.815
14/08/1990	0.76	21.79	0.83	0.081	2.892	0.411
29/08/1992	0.62	13.57	0.90	0.038	8.040	0.769
14/09/1992	0.60	61.15	0.76	0.010	3.855	0.490
Mean	0.66	39.25	0.84	0.347	3.365	0.660

Note #: No contribution was found from this process.

10.2.7 Sixth Creek

The Sixth Creek catchment is a steep catchment in the high rainfall area of the Mount Lofty Ranges. There is a substantial amount of natural vegetation. It has a catchment area of 43.8km^2.

Table 10-7 Sixth Creek RRR Calibration Results

Event Start Date	PL1	IL2 (mm)	PL2	k	kp1	kp2
21/06/1987	0.88	41.48	0.68	0.207	13.45	0.848
15/09/1991	0.54	37.70	0.60	0.357	2.175	0.768
29/08/1992	0.59	37.60	0.63	0.256	2.886	0.502
07/10/1990	0.52	16.27	0.57	0.263	8.077	1.308
17/12/1992	0.75	11.13	0.88	0.302	2.598	0.461
28/09/1996	0.62	29.61	0.60	0.497	3.396	0.680
Mean	0.63	28.92	0.65	0.329	4.829	0.763

10.3 Correlation of Storage Parameters with Catchment Area, Mainstream Length and Equal Area Slope

From the above calibrations generalised parameters were derived so that storage parameters could be compared across the catchments. Table 10-8 summarises fitted storage parameters, together with basic catchment physical data, including catchment area, mainstream length, and equal area slope (Se).

Table 10-8 Mount Lofty Ranges RRR Storage Parameter Summary

Catchment	Area (km^2)	length (km)	Se (m/km)	c_p1	c_p2	v_c (m/sec)
Cox	4.27	3.1	33.9	0.505	0.049	0.77
First	4.89	3.6	72.2	3.080	0.424	0.47
Aldgate	7.96	4.6	33.3	1.602	0.119	0.54
Inverbrackie	8.27	6.1	15.6	0.770	0.195	0.86
Lenswood	16.84	6.7	18.7	1.215	0.203	1.38
Western Branch	24.2	8.8	19.9	0.648	0.207	0.78
Torrens	25.95	9.0	7.0	0.655	0.212	1.00
Scott	26.54	10.0	19.5	0.806	0.216	1.96
Echunga	34.05	13.5	4.6	0.960	0.185	1.14
Sixth	43.83	16.8	19.7	2.267	0.358	1.42
Woodside Weir	51.9	15.5	8.1	0.949	0.257	1.24

A correlation matrix was developed to find any relationships between the catchment parameters and the RRR storage parameters.

Table 10-9 Correlation Matrix for RRR Storage Parameters

	Area (km^2)	length (km)	Se (m/km)	c_p1	c_p2	v_c
Area (km^2)	1.00					
length (km)	0.97	1.00				
Se (m/km)	-0.62	-0.62	1.00			
c_p1	-0.09	-0.03	0.72	1.00		
c_p2	0.31	0.34	0.37	0.78	1.00	
v_c	0.59	0.60	-0.54	-0.25	0.08	1.00

As expected a strong relationship is found for catchment area and mainstream length. For the Mount Lofty Ranges catchments there is also a relationship between catchment area and the equal area slope.

There is a low correlation between process storage parameters and catchment area. This supports the use of the generalised storage parameters cp_1 and cp_2, which were derived to minimise the effect of catchment area caused by the non-linearity of the lag relationship. Figure 10-2 shows scatter diagrams for the characteristic storage parameters.

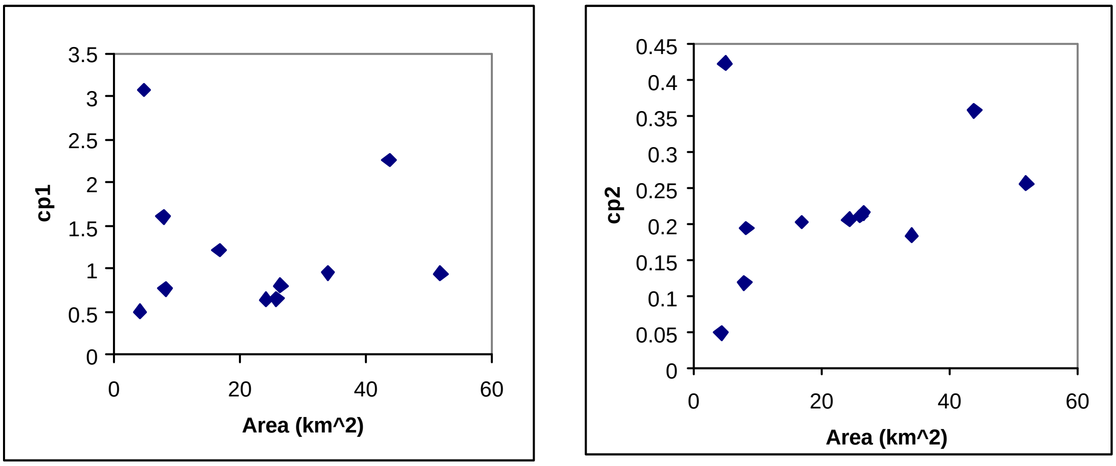


Figure 10-2 Correlation of Characteristic Storage Parameters with Catchment Area

There is some evidence of a relationship between the characteristic velocity v_c and the catchment characteristics. Scatter diagrams were produced for area and slope with v_c.

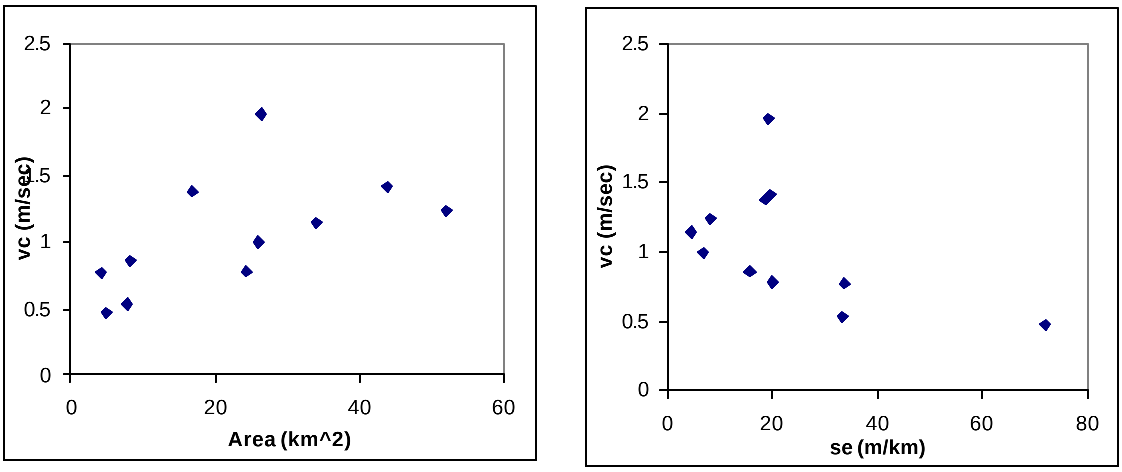


Figure 10-3 Correlation of Characteristic Velocity with Catchment Area and Equal Area Slope

The relationships are:

$$v_c = 0.016\,A + 0.689 \qquad (r^2 = 0.35)$$ Equation 10.1

And:

$$v_c = 1.335 - 0.0124\,s_e \qquad (r^2 = 0.28)$$ Equation10.2

The finding of increasing characteristic velocity with increasing catchment area and mainstream length is supported by Pilgrim (1982), who found that as a result of tracing studies flood velocities trend to increase slightly in a downstream direction throughout most catchments, despite decreasing slopes. Changes in hydraulic roughness and cross-sectional shape more than compensate for the effects of reduction of slope.

There is also a correlation of the two process storage parameters, c_p1 and c_p2, indicating that process lag for both processes are determined by similar factors. Figure 10-3 shows a scatter diagram of c_p1 and c_p2.

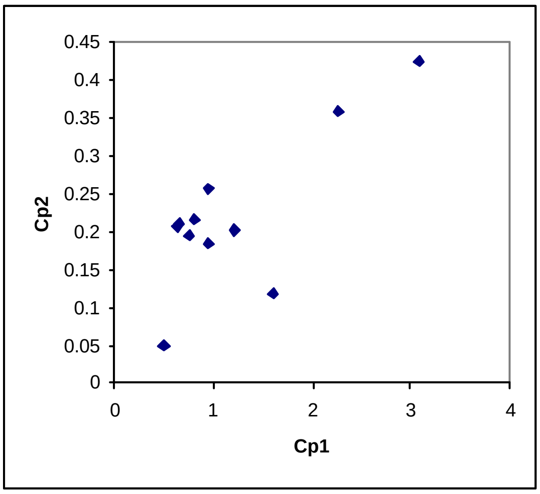


Figure 10-4 Correlation of cp1 and cp2

10.4 Correlation with Other Catchment Characteristics

The relationship of the RRR model storage and loss parameters with other catchment characteristics was examined.

Characteristics were obtained from the Department for Water Resources in South Australia, where they were derived as part of a study into stream flow characteristics of Mount Lofty Ranges catchments (McMurray, 1996). Table 10-10 to Table 10-16, taken from McMurray summarise the parameters examined. The parameters shown in bold were used for statistical analysis. Many other characteristics were rejected for statistical analysis due to their correlation with other characteristics.

Table 10-10 Catchment Characteristics Determined for the Mount Lofty Ranges Catchments – Land Use

Code	Units	Description
		LAND USES
		Percentage of each catchment area covered by the following land uses:
Lfo	%	Forest (Plantation Forest).
Lnv	%	Native Vegetation (Native Veg, Native Trust Reserve, NPWS).
Ler	%	Extensive Rural (Crops, Dairy, Dairy/Vegetables, Grazing, Horse, Recreation, Rural Living, Veg/Grazing Rotation).
Lir	%	Intensive Rural (Berry, Flori/Berry/Hort, Orchard, Vine, Winery). NOTE - The values of Lir are significant in a small number of catchments only.
Lur	%	Urban. NOTE - The values of Lur are significant in a small number of catchments only.
Lla	%	Lakes (Lake, Dam, Effluent Pond). NOTE - The values for Lla are all very low and were not be included in the statistical analysis.
%AC	%	Percentage of catchment area accounted for in the above categories.

Table 10-11 Catchment Characteristics Determined for the Mount Lofty Ranges Catchments - Soils

Code	Units	Description
		SOIL
		Percentage of each catchment area covered by soil with the following properties:
Swl	%	Soil prone to waterlogging or with minor potential to water logging
Spd	%	Soil with poor drainage (Drainage category = 5 or 7 or 8)
Shw	%	Soil with high root zone water holding capacity (Rootzonewhc category = 1)
Slw	%	Soil with low root zone water holding capacity (Rootzonewhc category = 4 or 5)
Ssh	%	Soil described as very shallow, shallow or moderately shallow.
Sde	%	Soil described as moderately deep, deep or very deep.
Sco	%	Soil with coarse texture (S, LS) or moderately coarse texture (SL, FSL).
Sfi	%	Soil with moderately fine texture (CL, SCL, SiCL) or fine texture (SC, SiC, C).
%AC	%	Percentage of catchment area accounted for in the above categories.

Table 10-12 Catchment Characteristics Determined for the Mount Lofty Ranges Catchments - Geology

Code	Units	Description
		GEOLOGY - Age Groups & Fault Lines
		Percentage of each catchment area underlaid by rock of the following geological age group:
Gqu	%	Quaternary
Gte	%	Tertiary
Gcf	%	Carboniferous
Gca	%	Cambrian
Glp	%	Late Pre-Cambrian
Gep	%	Early Pre-Cambrian
Got	%	Other
%AC	%	Percentage of catchment area accounted for in the above categories.
Gfl	km	Total length of Fault Lines in each catchment.
Gfa	km/ km^2	Total length of fault lines normalised by area (total fault line length divided by catchment area).

Table 10-13 Catchment Characteristics Determined for the Mount Lofty Ranges Catchments - Rainfall and Farm Dams

Code	Units	Description
		RAIN
Ram	mm	Mean Annual Rainfall (temporal and spatial mean).
Rat	GL	Total Mean Annual Rain input (temporal mean).
Rwm	mm	Mean Winter Rainfall (temporal and spatial mean).
Rwt	GL	Total Mean Winter Rain input (temporal mean).
		FARM DAMS
Fml	ML	Total estimated capacity of farm dams in catchment.
Fde	ML/km^2	Farm dams normalised by area, or Density of farm dams (total farm dam volume in ML divided by catchment area in km^2). NOTE - There is a strong linear correlation between Fde and Fwr. Fde is the only farm dams characteristic recommended for the statistical analysis.
Fwr	ML/GL	Farm dams normalised by rain input as a surrogate to runoff (total farm dam volume in ML divided by Rwt in GL).

Table 10-14 Catchment Characteristics Determined for the Mount Lofty Ranges Catchments - Topographic

Code	Units	Description
		TOPOGRAPHIC
Tsm	degrees	Average slope of catchment. NOTE - There are correlations (not all linear) between Sme and many of the following. Tsm (prefered) or Ts2 are the only slope characteristic recommended for the statistical analysis.
Ts1	%	Percentage catchment area with slope > 5°
Ts2	%	Percentage catchment area with slope > 10°
Ts3	%	Percentage catchment area with slope > 15°
Ts4	%	Percentage catchment area with slope > 20°
Ts5	%	Percentage catchment area with slope > 25°
Ts6	%	Percentage catchment area with slope > 30°
Ten	m	Minimum Elevation
Tex	m	Maximum Elevation
Tem	m	Mean Elevation
Tes	m	Standard Deviation of the Elevation

Table 10-15 Catchment Characteristics Determined for the Mount Lofty Ranges Catchments – Stream, Physical and Hillslope Connectivity

Code	Units	Description
		STREAMS
Wl1	km	Total Length of Strahler First Order Streams and above.
Wl3	km	Total Length of Strahler Third Order Streams and above.
Wd1	km/km²	Density of First Order Streams (total length / catchment area). NOTE - This is the only stream length characteristic recommended for the statistical analysis.
Wd3	km/km²	Density of Third Order Streams (total length / catchment area).
		PHYSICAL
Par	km²	Catchment Area
Ppe	km	Catchment Perimeter
Pap	km²/km	Shape (Perimeter in km/Area in km²)
		HCC - Hillslope-Channel Connectivity
		The percentage of the stream network length that is “connected” to hillslopes of 10° or greater. NOTE - HCC is strongly correlated to Tsm and Ts2. Therefore, it is recommended that HCC is not included in the statistical analysis.
H11	%	HCC for Strahler First Order Streams and above (one-cell method).
H19	%	HCC for Strahler First Order Streams and above (nine-cell method).
H31	%	HCC for Strahler Third Order Streams and above (one-cell method).
H39	%	HCC for Strahler Third Order Streams and above (nine-cell method).

Table 10-16 Catchment Characteristics Determined for the Mount Lofty Ranges Catchments - Groundwater

Code	Units	Description
		GROUNDWATER
		Percentage of each catchment area with the following groundwater recharge or discharge characteristic:
Npr	%	Peak Recharge.
Nre	%	Recharge.
Nsh	%	Depth to Water Table < 2m.
Ndi	%	Discharge.
%AC	%	Percentage of catchment area accounted for in the above.
Ntr	%	All Recharge Areas (sum of Npr and Nre).
Nts	%	All Discharge or Potential Discharge Areas (sum of Nsh and Ndi).

10.4.1 Storage Parameters

A correlation analysis was carried out using an Excel spreadsheet for the RRR storage parameters and catchment characteristics. Data from McMurray was available for only nine of the 11 catchments examined. It was determined that some of the characteristics, particularly related to geology had little correlation. Catchment geology was therefore not further examined.

Table 10-17 gives a summary of the correlations.

Table 10-17 Correlation of RRR Storage Parameters with Winter Runoff, Soil and Topographical Characteristics

	c_p1	c_p2	v_c	Rwm	Tsm	Tem	Swl	Spd	Shw	Slw	Ssh	Sde	Sco	Sfi
c_p1	1.00													
c_p2	0.82	1.00												
v_c	-0.30	0.06	1.00											
Rwm	0.37	-0.10	-0.12	1.00										
Tsm	0.80	0.73	0.17	0.51	1.00									
Tem	0.28	-0.01	-0.73	0.20	0.05	1.00								
Swl	-0.42	-0.03	0.50	-0.46	-0.27	-0.54	1.00							
Spd	-0.16	-0.11	0.06	-0.17	-0.30	-0.62	0.00	1.00						
Shw	-0.66	-0.62	0.12	-0.13	-0.57	0.01	0.25	0.16	1.00					
Slw	0.78	0.74	-0.25	0.26	0.65	0.16	-0.11	-0.19	-0.73	1.00				
Ssh	-0.20	-0.57	0.05	0.70	0.03	-0.13	-0.09	-0.04	0.02	-0.23	1.00			
Sde	0.68	0.69	0.14	0.35	0.78	0.17	-0.34	-0.17	-0.24	0.58	-0.36	1.00		
Sco	0.45	0.04	-0.45	0.72	0.38	0.11	-0.33	0.12	-0.41	0.53	0.56	0.09	1.00	
Sfi	-0.21	0.02	0.73	0.11	0.19	-0.61	0.59	-0.13	-0.20	0.18	0.30	0.01	0.01	1.00

The correlations for storage parameters with a coefficient of greater than 0.5 are highlighted. It can be seen that the main determinants of the process storage parameters are Tsm (Average catchment slope), Shw (Percentage of soil with high root zone water holding capacity), Slw (Percentage of catchment with low root zone water holding capacity), and Sde (Percentage of catchment with moderately deep, deep or very deep soil).

The relationships can be explained qualitatively as follows:

- Process storage lag increases with Tsm, the average slope of the catchment. This is counter intuitive, but may be related to the fact that other catchment characteristics are related to slope in these catchments. For example there is a correlation (coefficient = 0.89) between average slope and the percentage of native vegetation in the catchment, and average slope and soil depth (coefficient = 0.78);
- Process storage lag is related to the root zone holding capacity of the soil, increasing as the water holding capacity decreases; and
- Process storage lag increases with soil depth. This is expected, as the flow path length to the channel system will increase with increasing soil depths.

And, it can be added as a result of the finding on non-linearity that;

- Process storage lag decreases with increasing movement of water through the hillside to the channel system.

From the investigation it can be stated that soil depth and the root zone water holding capacity are the two main factors that affect catchment process lag. The presence of native vegetation in the catchment increases both catchment response times. However in the data set used the percentage of native vegetation in the catchment is related to both the percentage of soils with low root zone water holding capacity (coefficient = 0.90) and the percentage of the catchment with soils described as deep (coefficient = 0.73). The percentage of native vegetation may not be an independent variable.

Similarly correlation with land use, groundwater recharge or discharge characteristics, farm dam density and stream density can be examined, as shown in Table 10-18.

Table 10-18 Correlation of RRR Storage Parameters with Land Use, Groundwater State, Farm Dam Density and Stream Density

	c_p1	c_p2	v_c	Lfo	Lnv	Ler	Lir	Lur	Npr	Nre	Nsh	Ndi	Fde	Wd1
c_p1	1.00													
c_p2	0.82	1.00												
v_c	-0.30	0.06	1.00											
Lfo	0.09	0.11	0.39	1.00										
Lnv	0.81	0.79	0.04	0.03	1.00									
Ler	-0.69	-0.32	0.18	-0.10	-0.71	1.00								
Lir	-0.17	-0.54	-0.13	0.20	-0.30	-0.43	1.00							
Lur	0.13	-0.36	-0.48	-0.28	-0.18	-0.29	0.36	1.00						
Npr	-0.40	0.09	-0.16	-0.17	-0.49	0.79	-0.55	-0.29	1.00					
Nre	-0.03	-0.23	-0.46	-0.51	-0.47	0.12	0.20	0.28	-0.31	1.00				
Nsh	-0.42	0.11	0.40	-0.09	0.06	0.55	-0.69	-0.34	0.67	-0.67	1.00			
Ndi	0.73	0.02	-0.30	0.33	0.32	-0.76	0.47	0.70	-0.47	-0.07	-0.51	1.00		
Fde	-0.53	-0.26	0.08	0.05	-0.65	0.76	-0.13	-0.29	0.79	-0.12	0.50	-0.63	1.00	
Wd1	0.65	0.80	0.21	0.10	0.89	-0.48	-0.35	-0.53	-0.33	-0.29	0.13	-0.27	-0.38	1.00

The characteristics that are related to c_p1 and c_p2 are Lnv (Percentage of catchment with native vegetation), and Wd1 (density of first order streams). However the density of first order streams is related to the percentage of native vegetation, the catchment average slope (coefficient = 0.78), and the soil depth (coefficient = 0.86). The stream density is possibly acting as a substitute for the other physical characteristics.

In summary, the analysis has shown that the soil depth and the root zone water holding capacity of the soil are the main determinants of process storage parameters. The root zone water holding capacity of a soil is influenced mainly be soil type, with clay soils having a higher water holding capacity than sandy soils. As soils become sandier the storage parameters increase, indicating that catchment lag increases. This could be explained as the lower permeability of clay soils reducing infiltration, and increasing the amount of water that is moved laterally to the channel system. The overall lag of the hillsides in the catchment will reduce as the percentage of soil that encourages lateral transmission increases.

The presence of native vegetation on the catchment also has an effect, increasing the process storage lag over that expected for other land uses.

The process lags for base and slow flow are related, which is not surprising since they are both governed by the two main determining factors, being root zone water holding capacity and soil depth.

10.4.2 Losses

The calibrated losses and catchment characteristics were also examined, as shown in Table 10-19 and Table 10-20.

Table 10-19 Correlation of RRR Loss Parameters with Winter Runoff, Soil and Topographical Characteristics

	IL2 (mm)	PL1	PL2	Rwm	Roff	Tsm	Tem	Swl	Spd	Shw	Slw	Ssh	Sde	Sco	Sfi
IL2 (mm)	1.00														
PL1	-0.12	1.00													
PL2	0.20	-0.13	1.00												
Rwm	0.19	0.52	0.30	1.00											
Roff	-0.07	0.33	0.03	0.84	1.00										
Tsm	0.83	0.09	0.36	0.51	0.24	1.00									
Tem	0.16	-0.24	-0.19	0.20	0.57	0.05	1.00								
Swl	-0.17	-0.52	0.38	-0.46	-0.60	-0.27	-0.54	1.00							
Spd	-0.35	0.41	0.13	-0.17	-0.37	-0.30	-0.62	0.00	1.00						
Shw	-0.76	-0.23	0.05	-0.13	0.08	-0.57	0.01	0.25	0.16	1.00					
Slw	0.86	0.04	0.44	0.26	-0.04	0.65	0.16	-0.11	-0.19	-0.73	1.00				
Ssh	-0.33	0.49	0.14	0.70	0.66	0.03	-0.13	-0.09	-0.04	0.02	-0.23	1.00			
Sde	0.73	0.07	0.23	0.35	0.11	0.78	0.17	-0.34	-0.17	-0.24	0.58	-0.36	1.00		
Sco	0.23	0.48	0.54	0.72	0.54	0.38	0.11	-0.33	0.12	-0.41	0.53	0.56	0.09	1.00	
Sfi	0.13	0.18	0.25	0.11	-0.24	0.19	-0.61	0.59	-0.13	-0.20	0.18	0.30	0.01	0.01	1.00

It can be seen that the initial loss for the second process IL2 is strongly correlated with Tsm (catchment average slope), Shw (percentage of soil with high root zone holding capacity), Slw (percentage of soil with low root zone water holding capacity) and Sde (soils described as moderately deep, deep or very deep). These variables are the same as those affecting the process storage lag parameters.

Catchment losses can be explained qualitatively as follows:

- Initial loss increases with increasing catchment average slope. As with process lag this is counter intuitive. It may occur because particular catchment characteristics are related to slope. For example there is a correlation (coefficient = 0.89) between average slope and the percentage of native vegetation in the catchment, and average slope and soil depth (coefficient = 0.78);
- Initial loss increases as the root zone water holding capacity decreases. This would indicate that the occurrence of slow runoff is related to processes that occur in the root zone; and

- As the soil depth increases, so does the initial loss for the second process (slow runoff). Again this indicates that slow runoff is related to the root zone, and can occur only when the water store within the soil becomes saturated.

From the investigation it can be stated that soil depth and the root zone water holding capacity are the two main factors that affect the initial loss for the slow flow process. The presence of native vegetation in the catchment increases both catchment response times, and the initial loss. As before in the data set used the percentage of native vegetation in the catchment is related to both the percentage of soils with low root zone water holding capacity (coefficient = 0.9) and the percentage of the catchment with soils described as deep (coefficient = 0.73). The percentage of native vegetation may not be an independent variable.

The increase in initial loss with reducing water holding capacity of the root zone can be explained if it is considered that water holding capacity is related to soil type, with sandy soils having lower water holding capacity. Thus the initial loss increases as soils become sandier. This is as expected.

There are no consistent variables correlated with the proportional losses. This may be due in part to the variability of the calibrated proportional losses between events, leading to variability in mean proportional loss. It may be also that there are other factors that affect the proportional loss displayed by the catchment.

Table 10-20 Correlation of RRR Loss Parameters with Land Use, Groundwater State, Farm Dam Density and Stream Density

	IL2 (mm)	PL1	PL2	Lfo	Lnv	Ler	Lir	Lur	Npr	Nre	Nsh	Ndi	Fde	Wd1
IL2 (mm)	1.00													
PL1	-0.12	1.00												
PL2	0.20	-0.13	1.00											
Lfo	-0.08	0.21	0.02	1.00										
Lnv	0.88	0.09	0.57	0.03	1.00									
Ler	-0.55	-0.25	-0.44	-0.10	-0.71	1.00								
Lir	-0.36	0.19	-0.13	0.20	-0.30	-0.43	1.00							
Lur	-0.12	0.14	-0.14	-0.28	-0.18	-0.29	0.36	1.00						
Npr	-0.39	-0.66	0.23	-0.17	-0.49	0.79	-0.55	-0.29	1.00					
Nre	0.07	0.13	-0.89	-0.51	-0.47	0.12	0.20	0.28	-0.31	1.00				
Nsh	-0.23	-0.20	0.57	-0.09	0.06	0.55	-0.69	-0.34	0.67	-0.67	1.00			
Ndi	0.27	0.09	0.10	0.33	0.32	-0.76	0.47	0.70	-0.47	-0.07	-0.51	1.00		
Fde	-0.56	-0.34	-0.20	0.05	-0.65	0.76	-0.13	-0.29	0.79	-0.12	0.50	-0.63	1.00	
Wd1	0.80	0.14	0.39	0.10	0.89	-0.48	-0.35	-0.53	-0.33	-0.29	0.13	-0.27	-0.38	1.00

It can be seen in Table 10-20 that the initial loss IL2 is correlated to with the percentage of natural vegetation within the catchment, increasing as the percentage of natural vegetation increases. There is also a correlation with the stream density Wd1. However as before the density of first order streams is related to the percentage of native vegetation, the catchment average slope (coefficient = 0.78), and the soil depth (coefficient = 0.86). The stream density is possibly acting as a surrogate for the other physical characteristics that affect initial loss.

There is no consistent correlation for both proportional losses.

10.5 Comparison of RRR Flows and Flood Frequency Analysis

The mean RRR parameter values, as used in the verification runs were used with design rainfalls, and the results compared with flood frequency analysis flows for the 10 and 100 year Average Recurrence Interval.

Stations in the Onkaparinga and adjacent catchments were used, where the length of record was sufficient to allow flood frequency analysis. The stations used are shown in Table 10-21.

Table 10-21 Stations for Flood Frequency Analysis

Station Name	Number	Years of Record
Scott Creek	AW503502	31
Echunga Creek	AW503506	27
Lenswood Creek	AW503507	28
Inverbrackie Creek	AW503508	28
Aldgate Creek	AW503509	28
Cox Creek	AW503527	25
Torrens River	AW504512	29
First Creek	AW504517	25
Sixth Creek	AW504523	24

For each station, the annual maximum flow series were determined. For periods of missing record a comparison was made with adjacent stations, and the year discounted if there was not certainty that the annual maximum would not have occurred during the period.

Table 10-22 Annual Maximum Flows (m^3/sec) used in Flood Frequency Analysis (Onkaparinga Catchment)

Year	Scott	Echunga	Lenswood	Inverbrackie	Aldgate	Cox
1970	7.3	n/a	n/a	n/a	n/a	n/a
1971	10.8	n/a	n/a	n/a	n/a	n/a
1972	5.5	n/a	8.5	4.0	2.7	n/a
1973	10.6	n/a	25.0	7.4	9.7	n/a
1974	8.3	n/a	6.1	5.8	5.4	n/a
1975	5.8	12.5	5.2	9.6	6.6	n/a
1976	1.3	12.3	2.4	1.7	3.5	1.7
1977	0.5	5.0	2.4	7.4	7.3	3.4
1978	6.1	11.3	10.8	6.2	6.4	7.5
1979	8.6	17.8	15.7	4.5	10.5	14.5
1980	7.4	4.1	5.7	0.5	8.2	4.9
1981	18.3	22.1	48.4	20.7	23.0	6.2
1982	1.9	0.5	1.2	0.007	3.3	2.6
1983	8.8	9.3	19.2	4.3	6.6	4.4
1984	8.9	14.3	8.8	3.7	4.8	4.2
1985	5.4	7.2	5.8	2.3	3.7	2.9
1986	12.3	8.6	17.5	2.5	6.6	5.8
1987	15.8	30.3	16.4	8.0	8.6	5.4
1988	5.0	16.7	10.4	5.3	16.9	5.6
1989	7.8	6.2	n/a	3.1	n/a	n/a
1990	4.1	17.0	n/a	3.0	n/a	n/a
1991	7.9	8.4	n/a	1.5	n/a	n/a
1992	15.0	44.2	n/a	18.1	n/a	n/a
1993	3.6	13.9	n/a	0.9	n/a	n/a
1994	1.5	3.3	n/a	0.019	3.5	n/a
1995	10.2	27.6	12.9	4.7	6.3	4.6
1996	15.4	41.7	15.1	6.3	6.9	4.1
1997	5.0	5.8	2.2	0.5	8.6	2.9
1998	5.9	6.6	9.3	1.0	8.8	3.4
1999	2.8	3.4	5.1	0.4	10.0	3.5
2000	8.5	17.4	9.8	8.4	8.2	6.6

n/a indicates that the year was not available or used for analysis.

0.5 Flow censored (not used) - low flow

14.5 Flow censored – high outlier

Table 10-23 Annual Maximum Flows (m^3/sec) used in Flood Frequency Analysis (Torrens Catchment)

year	Torrens	First	Sixth
1973	14.44	n/a	n/a
1974	25.26	n/a	n/a
1975	21.12	n/a	n/a
1976	0.095	n/a	n/a
1977	0.167	0.109	n/a
1978	9.28	0.929	25.07
1979	6.37	2.804	38.00
1980	4.93	0.727	4.93
1981	24.36	1.803	24.36
1982	0.036	0.146	n/a
1983	22.85	10.14	15.70
1984	6.83	0.781	10.07
1985	2.06	n/a	11.43
1986	4.07	1.338	17.03
1987	9.21	1.066	27.26
1988	15.31	0.488	12.14
1989	12.34	0.413	n/a
1990	4.36	0.727	n/a
1991	9.01	0.719	27.12
1992	67.64	1.012	81.7
1993	1.49	n/a	5.14
1994	0.41	0.146	2.61
1995	34.54	n/a	28.36
1996	14.83	n/a	17.73
1997	0.596	0.245	5.05
1998	3.184	0.211	9.98
1999	2.23	0.621	8.19
2000	14.63	0.608	15.01
2001	12.75	n/a	11.13

n/a indicates that the year was not available or used for analysis.

10.14 Flow censored – high outlier

For all stations but for the River Torrens at Mt Pleasant the log-normal frequency distribution was used. This distribution fitted the recorded values in most cases. The use of a log-normal distribution was also confirmed to be a reasonable distribution by the application of the continuous simulation to the Gauging station at Houlgraves on the Onkaparinga River. The continuous simulation was carried out as part of the study for the Onkaparinga Catchment Water Management Board.

In some cases low flows were censored from the data set if these would influence unduly the frequency distribution for the less common flows. Flows were censored subjectively on the basis that emphasis was given to fitting the higher end of the frequency distribution. Because of the variability of South Australian rivers there are often years of very low or even zero flows. Klemeš (1986) describes examples where constraints imposed by data points at the low end of the range of observed values cause a systematic deviation of the fitted distribution from the high range of values.

A log-Pearson III distribution was used for the River Torrens at Mount Pleasant, as many of the gauged flows fell outside the 10% and 90% confidence limits for the log-normal distribution.

The high flow in Cox Creek is double any other flow recorded at the station. Comment was sought from Robin Leaney, Senior Hydrological Information Officer of the Department for Water Resources, who confirmed that there were no indications that the flow was in error. John Harrison of the Adelaide Hills Council was also contacted. However there are no records in Council indicating that significant flooding had occurred. Examination of the records also showed that the recorded level at the gauging station was only 200mm above the second highest flow. There were no pluviometers near the catchment in 1979, but the 24 hour rainfall at Uraidla, within the catchment was only 37mm, which is less than 1 year ARI. Over the 3 days leading up to and including the event 74mm of rain fell, approximately 1 in 1 year ARI. The flow was censored from the record for frequency analysis, as it can be assumed to be an outlier.

The high flow in First Creek in 1983 is more than double any other flow, and occurred shortly after a bushfire burnt the catchment. It was thus assumed to be an outlier and rejected as the catchment was not in the same condition as all the other years.

Table 10-24 gives the result of the flood frequency analysis. Appendix 9 contains plots of the frequency distributions.

Table 10-24 Results of Flood Frequency Analysis

Station	Area (km^2)	Q10 (m^3/sec)	Q20 (m^3/sec)	Q50 (m^3/sec)	Q100 (m^3/sec)
Cox	4.3	6.65	7.49	8.55	9.35
Aldgate	8.0	13.2	15.9	19.7	22.6
Inverbrackie	8.4	12.3	16.2	22.0	27.0
Lenswood	16.5	25.9	35.8	51.6	65.9
Scott	26.8	15.6	20.0	26.4	31.7
Echunga	34.2	30.6	40.1	54.6	66.9
Torrens	26.0	35.3	48.9	65.9	77.3
First	4.9	1.80	2.51	3.64	4.66
Sixth	43.8	40.1	54.0	75.5	94.4

For the comparison, weighted mean values of the RRR model storage parameters and proportional losses were used, together with zero initial loss for process 1 (baseflow) and the weighted mean value of the initial loss for the second process. The initial loss for the third process was set at 100mm, with the proportional loss consistent with the proportional losses for the other two processes.

One problem with the prediction of flows is that the initial and proportional loss for process 3 (fast flow) is not usually determined from calibration, as the process rarely occurs. It was found only in three calibration events on the Mount Lofty Ranges Catchments. In most cases PL3 must be estimated. From the calibrations undertaken that show runoff from process 3, the proportional loss is generally of the same order as that of process 1 and 2. Table 10-25 gives a summary of the proportional losses. It is noted that the estimated proportional loss for process 3 sometimes leads to more runoff occurring than rainfall during that part of the storm where 3 runoff processes are occurring. These losses can be reviewed in the derivation of design losses.

Table 10-25 Proportional Losses Assumed for Comparison

Catchment	PL1	PL2	PL3	% Runoff with 3 processes operating
Cox	0.82	0.76	0.80 (estimated)	62%
Aldgate	0.75	0.60	0.65 (from 1 calibration)	100%
Inverbrackie	0.74	0.42	0.70 (estimated)	114%
Lenswood	0.68	0.58	0.60 (estimated)	114%
Scott	0.78	0.76	0.75 (estimated)	71%
Echunga	0.89	0.72	0.82 (from 1 calibration)	47%
Torrens	0.25	0.75	0.28 (estimated)	128%
First	0.66	0.84	Not used	
Sixth	0.63	0.65	0.63 (estimated)	109%

The initial loss of process 3 is also unknown, but 100mm was assumed for initial comparison.

Table 10-26 and Figure 10-5 give the comparison, and shows no significant bias towards over or under estimating flows. However there are some differences between the model and flood frequency flows, particularly significant being the Echunga Creek catchment.

Table 10-26 Comparison of Flood Frequency and Calibrated RRR Model

Catchment	Q10 RRR model (m^3/sec)	Q10 flood frequency (m^3/sec)	Q100 RRR model (m^3/sec)	Q100 flood frequency (m^3/sec)
Cox	5.7	6.7	9.3	9.4
Aldgate	12.3	13.2	21.5	22.6
Inverbrackie	13.2	12.3	22.9	27.0
Lenswood	24.2	25.9	61.3	65.9
Scott	18.5	15.6	31.3	31.7
Echunga	26.0	30.6	42.6	66.9
Torrens	47.3	35.3	78.7	77.3
First	1.4	1.8	2.8	4.7
Sixth	32.8	40.1	89.9	94.4

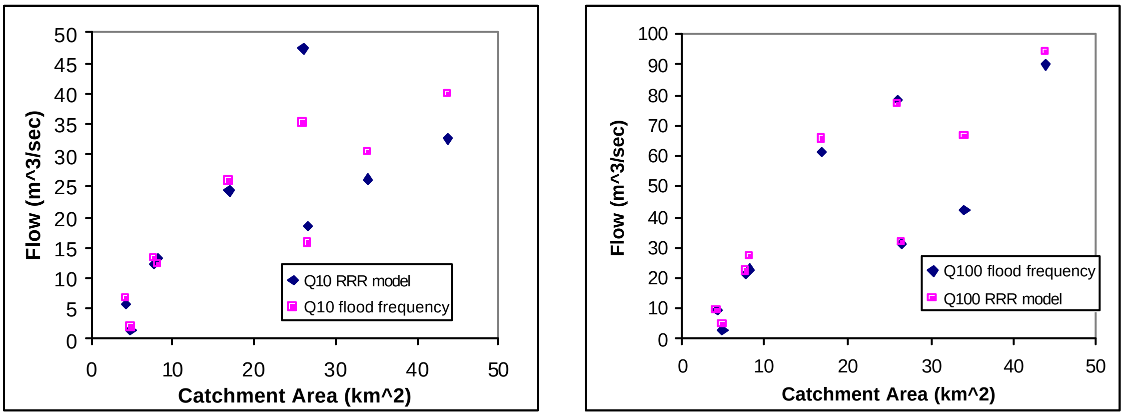


Figure 10-5 Comparison of Calibrated RRR Model and Flood Frequency Flows

10.6 Derivation of Design Losses and Correlation with Catchment Characteristics

It is necessary to determine design losses because design storms represent bursts within longer duration storm events. Calibrated loss may not be truly representative of mean catchment conditions, to be used with design rainfalls. It is thus considered legitimate to vary the losses determined in the calibration to obtain design losses.

Another issue is whether the flow predicted by the RRR model or the flow from the station flood frequency analysis is more representative of the true flow for each recurrence interval. As the station flood frequency flow is based on recorded data it was decided to adjust the RRR model parameters to match the flood frequency analysis flow, where this was possible while keeping to reasonable parameter limits.

The calibrated losses for the 6 catchments that had flood frequency analysis flows were adjusted so that the RRR model matched the flood frequency analysis flows. This was done as follows:

- The PL2 was adjusted so that the 10 year ARI flows matched. This was done as it was assumed that no fast flow occurred at this ARI, based on the calibration events.

- The PL3 and IL3 were then adjusted to give good agreement with the 100 year ARI flow. The IL3 was kept at 100mm, and PL3 adjusted. If the total runoff volume reached 100% of the rainfall with all 3 processes occurring, the IL3 was adjusted.

Table 10-27 gives the results of the RRR model parameter adjustment for the 9 catchments having flood frequency analysis available. Note that it was assumed that no process 3 (fast flow) occurs in the First Creek catchment.

Table 10-27 RRR Model Design Loss Parameters – Catchments with Frequency Analysis

Catchment	IL2 (mm)	IL3 (mm)	PL1	PL2	PL3
Cox	5.6	100	0.82	0.76	0.80
Aldgate	15.6	100	0.75	0.55	0.85
Inverbrackie	16.9	50	0.74	0.46	0.80
Lenswood	17.3	70	0.68	0.55	0.77
Scott	21.6	96	0.78	0.80	0.75
Echunga	8.7	90	0.89	0.67	0.44
First	39.3	n/a	0.66	0.73	n/a
Torrens	11.5	40	0.75	0.48	0.77
Sixth	28.9	100	0.63	0.65	0.63

The First Creek catchment had the majority of flow for the calibrated events from baseflow, so it was assumed initially that no fast runoff would occur on the catchment. This was confirmed when design flows were determined, as no fast runoff was required with design rainfalls to match the at station flood frequency analysis.

Analysis was then carried out into the correlation between design loss parameters and catchment characteristics. Table 10-28 and Table 10-29 show the results.

Table 10-28 Correlation of RRR Design Loss Parameters with Winter Runoff, Soil and Topographical Characteristics.

	IL2 (mm)	IL3 (mm)	PL1	PL2	PL3	Rwm	Roff	Tsm	Tem	Swl	Spd	Shw	Slw	Ssh	Sde	Sco	Sfi	Lfo
IL2 (mm)	1.00																	
IL3 (mm)	0.16	1.00																
PL1	-0.77	0.14	1.00															
PL2	0.23	0.77	0.19	1.00														
PL3	0.01	-0.22	-0.36	-0.28	1.00													
Rwm	0.19	0.87	-0.10	0.61	0.18	1.00												
Roff	-0.07	0.59	-0.13	0.24	0.46	0.84	1.00											
Tsm	0.83	0.60	-0.67	0.56	-0.14	0.51	0.24	1.00										
Tem	0.16	-0.25	-0.48	-0.33	0.72	0.20	0.57	0.05	1.00									
Swl	-0.17	-0.33	0.25	0.04	-0.02	-0.46	-0.60	-0.27	-0.54	1.00								
Spd	-0.35	0.15	0.67	0.13	-0.88	-0.17	-0.37	-0.30	-0.62	0.00	1.00							
Shw	-0.76	-0.31	0.40	-0.23	0.05	-0.13	0.08	-0.57	0.01	0.25	0.16	1.00						
Slw	0.86	0.38	-0.42	0.43	0.04	0.26	-0.04	0.65	0.16	-0.11	-0.19	-0.73	1.00					
Ssh	-0.33	0.85	0.36	0.47	0.19	0.70	0.66	0.03	-0.13	-0.09	-0.04	0.02	-0.23	1.00				
Sde	0.73	0.20	-0.63	0.34	-0.16	0.35	0.11	0.78	0.17	-0.34	-0.17	-0.24	0.58	-0.36	1.00			
Sco	0.23	0.84	0.17	0.66	-0.08	0.72	0.54	0.38	0.11	-0.33	0.12	-0.41	0.53	0.56	0.09	1.00		
Sfi	0.13	0.27	0.14	0.53	0.06	0.11	-0.24	0.19	-0.61	0.59	-0.13	-0.20	0.18	0.30	0.01	0.01	1.00	
Lfo	-0.08	0.45	-0.02	0.19	-0.73	0.12	0.04	0.32	-0.38	-0.24	0.44	0.14	-0.35	0.11	0.22	-0.07	-0.18	1.00

Table 10-29 Correlation of RRR Loss Parameters with Land Use, Groundwater State, Farm Dam Density and Stream Density

	IL2 (mm)	IL3 (mm)	PL1	PL2	PL3	Lfo	Lnv	Ler	Lir	Lur	Npr	Nre	Nsh	Ndi	Fde	Wd1
IL2 (mm)	1.00															
IL3 (mm)	0.16	1.00														
PL1	-0.77	0.14	1.00													
PL2	0.23	0.77	0.19	1.00												
PL3	0.01	-0.22	-0.36	-0.28	1.00											
Lfo	-0.08	0.45	-0.02	0.19	-0.73	1.00										
Lnv	0.88	0.65	-0.47	0.65	-0.29	0.03	1.00									
Ler	-0.55	-0.80	0.45	-0.59	-0.20	-0.10	-0.71	1.00								
Lir	-0.36	0.35	-0.05	0.02	0.42	0.20	-0.30	-0.43	1.00							
Lur	-0.12	0.36	0.05	-0.16	0.42	-0.28	-0.18	-0.29	0.36	1.00						
Npr	-0.39	-0.71	0.28	-0.45	-0.25	-0.17	-0.49	0.79	-0.55	-0.29	1.00					
Nre	0.07	-0.32	-0.31	-0.56	0.63	-0.51	-0.47	0.12	0.20	0.28	-0.31	1.00				
Nsh	-0.23	-0.21	0.57	0.20	-0.50	-0.09	0.06	0.55	-0.69	-0.34	0.67	-0.67	1.00			
Ndi	0.27	0.67	-0.32	0.15	0.12	0.33	0.32	-0.76	0.47	0.70	-0.47	-0.07	-0.51	1.00		
Fde	-0.56	-0.73	0.39	-0.59	-0.31	0.05	-0.65	0.76	-0.13	-0.29	0.79	-0.12	0.50	-0.63	1.00	
Wd1	0.80	0.22	-0.43	0.54	-0.45	0.10	0.89	-0.48	-0.35	-0.53	-0.33	-0.29	0.13	-0.27	-0.38	1.00

As before, there are no consistent factors that correlate with design proportional losses.

10.7 Summary

The factors that affect RRR storage and loss parameters have been examined in regard to a group of catchments within the Mount Lofty Ranges of South Australia.

It has been determined that there is some correlation of the characteristic channel velocity v_c with catchment area and equal area slope. This confirms the finding of Pilgrim (1982) that flood velocities tended to increase in a downstream direction throughout most catchments.

It has been found that for both process storage lag and the initial loss for the second runoff process (IL2) that conclusions can be made as to the factors that govern the parameters as follows:

- Process storage lag and IL2 is related to the root zone holding capacity of the soil, increasing as the water holding capacity decreases.
- Both process storage lag and IL2 increases with soil depth. The flow path length to the channel system will increase with increasing soil depths, leading to a longer response time.
- Process storage lag and IL2 increases with Tsm, the average slope of the catchment. This is counter intuitive, but may be because other catchment characteristics are related to slope in the data set. For example there is a correlation (coefficient = 0.89) between average slope and the percentage of native vegetation in the catchment, and average slope and soil depth (coefficient = 0.78), both of which influence process lag and IL2.
- Response time decreases with increased flows through the hillside flow paths.

From the investigation it can be stated that soil depth and the root zone water holding capacity are the two main factors that affect catchment process lag and the initial loss for the slow flow component.

The root zone water holding capacity is related to soil type, with the holding capacity reducing as the soil becomes sandier. Process lag increases as soils become sandier, indicating that more infiltration is occurring rather than runoff from at or near the surface, which would have a faster response time. The initial loss for the slow flow increases as the soil becomes sandier, as expected.

The presence of native vegetation in the catchment increases both catchment response times, and the initial loss. However in the data set used the percentage of native vegetation in the catchment is related to both the percentage of soils with low root zone water holding capacity (coefficient = 0.9) and the percentage of the catchment with soils described as deep (coefficient = 0.73). The percentage of native vegetation may not be an independent variable.

The RRR model with design rainfalls and calibrated losses gives an unbiased estimate of flows as determined by at station flood frequency analysis.

As it becomes easier to simulate complex problems, modellers must ask: "Is a more complex model necessarily a better model?" In many cases the answer is likely to be "no".

Mary Anderson (1999)

11. Application of the RRR model

11.1 Introduction

During the course of the development of the RRR model the opportunity arose to use the RRR model for the review of the flood hydrology of two catchments within the Adelaide urban area. The reviews were carried out at the time when the model was being verified on the rural catchments in the Mount Lofty Ranges, and before the storage and loss parameters were finalised.

Also during the development of the model a large flood occurred at Olary, 400km north east of Adelaide on the road to Broken Hill. The RRR model was used in the modelling of the event, to gain some understanding of the runoff processes that were occurring.

This chapter summarises the reports produced for Keswick and Brownhill Creeks, and the paper produced on the Olary Flood (Daniell et al, 1998). The Olary flood paper is reproduced in full in Appendix 11.

11.2 Keswick Creek

The Flood Warning Consultative Committee (FWCC) in South Australia endorsed a pilot study into flood risk management in a flood prone area of the western suburbs of Adelaide. The study forms a research program currently under way at the University of Adelaide into the development of a flash flood warning system for Brownhill and Keswick Creeks, and is reported by Wright and Daniell (1998). As part of the program a review of the hydrology of Keswick Creek was included.

Flood mapping of Keswick Creek was carried out in 1984 by WBCM Consultants (WBCM, 1984) and the resultant maps show significant areas of the western suburbs are at risk of flood damage.

Figure 11-1 Keswick Creek at Goodwood Road, October 1997

The WBCM study used the limited information that was available at the time. No flow data was available for the catchment, and the nearest pluviometers were located at Kent Town and at the Waite Institute at Urrbrae, both outside the catchment. The study report drew attention to the lack of event data for the catchment and proposed that further work be done to confirm the extent and frequency of flooding.

Further flow and rainfall data are now available within the catchment. Figure 11-2 shows the catchment and the location of the stations maintained by the Bureau of Meteorology.

The review was carried out as part of the development of the RRR model, as a case study in the application of the model to an urban area.

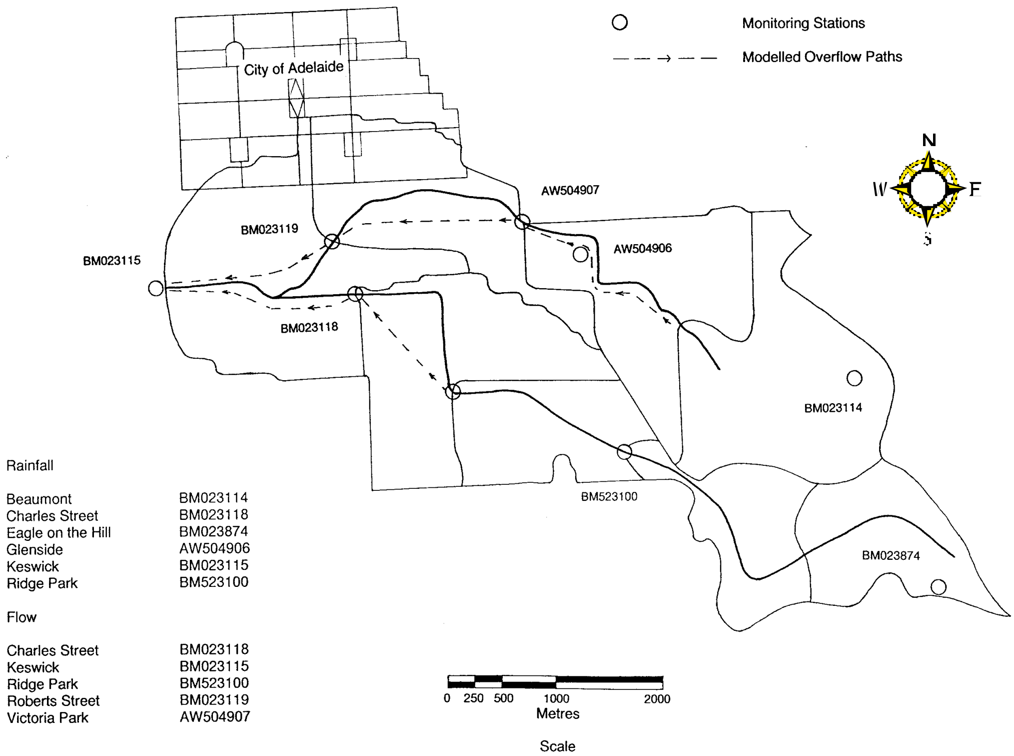


Figure 11-2 Keswick Creek Catchment with the RRR Model Sub-areas

11.2.1 The Advantages of the RRR Model

Most models cannot deal with both pipe flow and the surface flow that occurs when the capacity of the pipe system is exceeded. This can be handled by ILSAX, but the model requires extensive data input to model large urban catchments. For Keswick Creek the RRR model was modified by the addition of a separate layer of storages through which these flows can be routed. At each channel storage location within each sub-catchment total flows are checked with respect to the flow capacity of the pipe system and flows in excess of the capacity are diverted to the parallel series of storages which represent the flow path through the street system.

The features of the RRR model as modified make it ideal for application on the Keswick Creek catchment because:

- The model can deal with a hydrological system that behaves differently for small and large flood events, as does the urban drainage system; and
- The model can be calibrated to any number of individual locations, without affecting the results at other parts of the model that would not be expected to change as a result of the calibration. This is part of model self-consistency, which does not exist in models such as RORB and RAFTS. In addition there is no single catchment wide storage parameter in RRR as there is in RORB. Different storage parameters and losses can be applied to different parts of the model which have different land use characteristics. For instance hydrological differences between rural and urban areas can be readily accommodated.

In addition because the RRR model treats the in-channel and process storages separately (or in-channel and pipe system storage routing) the RRR model will intrinsically give a better indication of flood peak travel times than will RORB or RAFTS.

11.2.2 Approach

The approach adopted was to set up a RRR model based on previously derived values for the percentage of directly connected impervious area, storage parameters and channel characteristic velocities.

The model was then run for two storm events in January and February 1997 for which good stream flow and pluviometer information was available. To better fit the measured hydrograph a reduction was made to the channel characteristic velocity in the urban area, from 3m/sec to 2 m/sec, but in general the model performed well on the initial runs.

The model was then run for storm events in August and December 1993. For these storms pluviometer information was available from only two stations, and the result was not expected to be as good as the two storms used to derive the model. However it was considered the model performed acceptably well, given these limitations.

The model was then adjusted to form a design model by the addition of further overflow paths, the inclusion of a newly constructed detention basin in Glenside and the better definition of some overflow paths, particularly in the showgrounds area.

A major storm event that occurred during the course of the study on 30th/31st October 1997 gave the opportunity to verify the model against a storm event that had a recurrence interval of between 20 and 50 years ARI for 24 hour duration.

The design model was also verified with regard to the historical evidence of flooding of the showgrounds. As a result of this verification, changes were made to the unconnected area losses within the urban area.

11.2.3 Features of the Catchment Incorporated in the Model

11.2.3.1 General

One feature of the model that varied from the normal RRR model is that 5 storages only were used for each sub-area channel and process modelling. This was necessary due to the limitation on the maximum number of nodes available in the RAFTS model software that was used (200 in the case of the license used), and as was shown in Chapter 9 similar results could be expected.

For the urban catchments the only effect was that the channel storage parameter k was double the value used previously for the RRR model.

In the rural catchments upstream of Ridge Park k_p had to be varied from values previously derived in rural catchments. The value of k_p derived from a normal 10 channel storage RRR model was multiplied by a factor of $2^{0.2}$ because of the effect of the non-linearity of the storages. The channel storage parameter was double that of a 10 channel storage model.

The model was set up such that rainfall data from the 6 pluviometers available for calibration could be applied to relevant areas. In addition the model required sub-area boundaries at points of interest such as gauging stations. In all a total of 8 sub-areas covered the catchment area, this being the minimum necessary to allow the application of the pluviometers and define points of interest. The layout of the sub-catchments is shown in Figure 11-2.

Flows in excess of the channel or pipe capacity were routed through a series of storages representing surface storage to the next downstream modelled location.

The RRR model is far simpler than other models due to the small number of sub-areas needed.

11.2.3.2 Glenside Basin

The City of Burnside has constructed a storage basin on land at the intersection of Fullarton Road and Greenhill Road, with construction commencing in March 1997. The basin provides flood storage as well as some minor improvement in water quality.

Storage - elevation data for the basin was obtained from BC Tonkin & Associates and incorporated into the model.

Peak inflows and outflows have been compared for the design events summarised in BC Tonkin's calculations. The results are summarised in Table 11-1, for the 1hr duration storm, which was assessed to be critical by BC Tonkin & Associates.

Table 11-1 Glenside Storage Basin Flow Confirmation (1 hour duration design storm)

Event ARI	BC Tonkin inflow (m^3/s)	Model Inflow (m^3/s)	BC Tonkin outflow (m^3/s)	Model Outflow (m^3/s)
5yr	17.8	15.9		
50yr	22.2	23.7	14.3	14.3
100yr	28.5	28.2	21.5	25.5

This model is consistent with BC Tonkin's analysis (BC Tonkin, 1996). The differences are probably due to differences in the model structure (with the Tonkin model being much more detailed) and different unconnected area losses, the unconnected area being the sum of the supplementary paved and pervious areas. This difference in losses leads to a different total runoff volume and thus a higher peak outflow at the 100 year ARI level.

11.2.3.3 South Parklands

The South Parklands channel was assumed to have a bank full capacity of 6m^3/s, from the findings of the WBCM report (WBCM, 1984). The characteristic velocity of flows within the bank was assumed to be 1m/sec, and once the flow exceeded bank full the velocity on the floodplain was assumed to be 0.5m/sec. The 1m/sec was based on calibrated values from the four rural catchments in the Mount Lofty Ranges, on the assumption that the channel through the parklands will behave in a similar fashion to a rural creek channel.

The in-channel characteristic velocity was confirmed by examining the recorded and predicted hydrographs at the gauging station immediately downstream of the Parklands.

11.2.3.4 Glen Osmond Creek Upstream of Ridge Park

Examination of the records from the Ridge Park gauging station indicated that Glen Osmond Creek upstream of Ridge Park behaves very differently to a normal rural catchment. For the events examined it was clear that there existed a substantial baseflow component, and only a small component of what would be considered surface runoff. In addition the surface runoff occurred very quickly, and indeed more quickly than would be expected for a rural catchment. The behaviour can be explained by the physical nature of the catchment. The creek system is piped beneath Mount Barker Road for much of its length. Inflow through the pipe joints has been observed during inspections of the pipe, and may explain the observed baseflow.

The indication from initial calibration was that runoff was occurring from only the paved road area for the events examined. In addition to this there was a contribution from the groundwater inflow to the pipe. This hydrologic behaviour may be expected to occur for events up to the stage where direct surface runoff occurs.

The sub-areas upstream of Ridge Park were modelled in RRR by the inclusion of a paved area directly connected to the pipe system and representing the road, with a process lag (equivalent to time of entry) of 5 minutes. The paved area was estimated to be the road area. The balance of the sub-area was treated as rural RRR model, but using 5 channel storages instead of 10.

11.2.3.5 Showgrounds

Examination of the WBCM report indicated that the entrance to the showground tunnel has a capacity of 25m^3/s. Flows in excess of this will cross Goodwood Road and enter the Showgrounds, where they will flow west towards the railway. A site inspection revealed that there is a substantial barrier to flows along the western boundary of the Showgrounds due to buildings and corrugated steel fences. If the fence is breached it is likely that a substantial proportion of the flow will escape to the north towards the Keswick railway station, before entering Mile End from behind the Advertiser complex.

The behaviour was modelled by the use of an overflow at Goodwood Road. However, it is difficult to predict how much will return to the Keswick Creek channel downstream of the railway. For this reason the model did not return the overflow back to the creek upstream of the gauging station.

11.2.3.6 Windsor Street

The WBCM report identified a limitation in the capacity of the Glen Osmond Creek channel at Windsor Street, Fullarton. The channel capacity is limited to 10m^3/s with flows in excess of this following a separate overflow path to Charles Street, as is indicated by the WBCM floodplain mapping and the local topography.

11.2.3.7 Adelaide Crafers Highway Detention Dam

A new detention dam was constructed in conjunction with the Adelaide Crafers Highway project on the upper reaches of Glen Osmond Creek. The storage-elevation-discharge relationship for the dam was obtained from BC Tonkin & Associates was been included in the final model for the prediction of flood flows.

11.2.4 Parameters

11.2.4.1 Storage Parameters

Urban Areas

The surface flow time for flows in excess of the pipe or channel was based on gutter flow times derived from the procedure used in the ILSAX model. It is assumed that flows in excess of the capacity of the pipe system behave linearly, as do all channel flows in the RRR model.

The equation used for the calculation of flow times, and thus storage parameters is based on the ILSAX model as derived in Chapter 8 as follows:

$$k = 7.26*10^{-6}\frac{L_g}{\sqrt{S_g}}\ hours \qquad \text{Equation 11.1}$$

Where L_g is the total gutter flow length (m)

S_g is the mean slope (m/m)

and k is the lag parameter in the storage equation S = 3600kQ

The constant in the above equation is double that quoted previously (3.63), due to the use of half the number of storages (5 instead of 10) in this model. The same formula was used to derive the storage lag for flows in excess of the channel capacity.

Most of the pipe systems within the urban catchment have a 5 year ARI capacity. The lower limit of surface flows was initially set by the use of a simple relationship of 5 year ARI flow versus area, derived from the Glenelg and the Paddocks catchments. It was refined following initial design runs of the RRR model such that it approximated the 5 year ARI flow from the sub-area.

The pipe flow lag parameter was derived based on the value derived in Chapter 8. The value was doubled to account for the use of 5 instead of ten channel storages. The equation used is:

$$k = [(0.666 * 10^{-3})\frac{L_p}{\sqrt{s_p}\, r_m^{0.667}} + (7.26 * 10^{-3})\frac{L_g}{\sqrt{s_g}}]\ x\, 10^{-3}\ hours$$ Equation 11.2

and for the process storages;

$$k_{pi} = 0.0083 hours$$ Equation 11.3

and

$$k_{pu} = 0.0183 hours$$ Equation 11.4

Where	L_p	Is the longest pipe length in the sub-area (m)
	s_p	is the mean pipe slope (m/m)
	r_m	is the mean pipe hydraulic radius (m)
	k_{pi}	is the process lag parameter in the storage equation S = 3600k_pQ for the directly connected impervious area
and	k_{pu}	is the process lag parameter in the storage equation S = 3600k_pQ for the unconnected area

The above derivation assumes that k_{pu} is 0.01 hours greater than k_{pi}, as determined in Chapter 4. In the case of urban sub-areas where there was substantial channel flow time as well as pipe flow time to the outlet this time was included at the characteristic channel flow velocity, which was determined by initial calibration of the RRR model to be 2m/sec.

The mean pipe hydraulic radii had to be estimated for use in the equations. Values of 0.2m in the Beaumont catchment to 0.4m in the Keswick catchment were used and this gave good results on the calibrations. It is assumed that as the catchment average slope is reduced, the pipe mean hydraulic radius will increase.

The percentage directly connected area within urban sub-areas was estimated based on the findings from the Glenelg and Paddocks catchments. The percentage directly connected as a percentage of the total sub-area varied from 12.5% for the Roberts Street catchment, containing parkland area to 26% for the Glenside catchment.

Rural Areas

The selection of storage parameters the model upstream of Ridge Park was based on calibration of other catchments in the Adelaide Hills. Calibration had been carried out on four catchments, with results as shown in Table 11-2:

Table 11-2 Calibrated Storage Parameters for Adelaide Hills Catchments

Location	Station	c_p1	c_p2
Torrens at Mount Pleasant	AW504512	0.61	0.20
Inverbrackie Creek	AW503508	0.57	0.23
Echunga Creek	AW503506	0.86	0.20
Scott Creek	AW503502	0.80	0.22

It can be seen that the storage parameters are stable, and do not change substantially from catchment to catchment. After examining the above the Scott Creek parameters were chosen, on the basis that the response of the catchment is most likely to be similar to Scott Creek, which is the closest calibrated catchment and also the most similar physically, with respect to climate and topography.

11.2.4.2 Losses

Urban Areas

Any hydrological model is sensitive to design losses. Unfortunately there is little information available on pervious area losses within urban areas. An investigation of ILSAX losses carried out for the Paddocks catchment (Department of Transport, 1996) indicated an initial loss of at least 45mm for the pervious area, but since there was no pervious area runoff a continuing loss was not able to be determined.

The ILSAX model adds the rainfall from the supplementary paved area to the pervious area before subtracting the losses. It is possible to have pervious area runoff in ILSAX with rainfalls less than the value of the initial loss.

The RRR model does not distinguish between supplementary paved area and pervious area, but takes the loss from the total of the supplementary paved and pervious area, termed the unconnected area. Thus an initial loss of 45mm in the ILSAX model will be equivalent to a lesser loss in RRR, by the ratio of the supplementary paved area to the total of the unconnected area.

For the creation of the model initial loss of 40mm and a continuing loss of 3mm/hr was used. The initial loss of 40mm is equivalent to a loss of approximately 55mm in ILSAX. This was considered to be reasonable, given that design storms are likely to occur in summer, when pervious areas are likely to be dry and little runoff can be expected.

Rural Areas

The losses for the rural sub-areas were determined from calibrated losses on other catchments, and by reference to other estimates of design flows.

Calibrated losses for the RRR model are shown in Table 11-3:

Table 11-3 Calibrated Losses for Adelaide Hills Catchments

Location	Station	*PL1*	*IL2 (mm)*	*PL2*
Torrens at Mount Pleasant	AW504512	0.75	11.5	0.28
Inverbrackie Creek	AW503508	0.74	16.6	0.42
Echunga Creek	AW503506	0.89	8.7	0.73
Scott Creek	AW503502	0.78	21.6	0.76

There were not enough instances of process 3 contribution on the calibrated catchments to enable good definition of *IL3* and *PL3*. Values of 50mm (*IL3*) and 0.76 (*PL3*) were selected on the basis of the available information.

At present no relationship has been derived for design initial loss for process 2. A loss of 25mm was initially selected for examination, on the basis that the above losses were derived from storm bursts with antecedent rainfall.

The proportional losses of 0.78 and 0.76 were initially selected for process 1 and 2, with an initial loss of 50mm and a proportional loss of 0.78 for process 3, based on limited information. These losses were subject to testing on the Glen Osmond Creek catchment at Ridge Park, with storm Average Recurrence Intervals of 5 and 100 years, and durations of 0.5 to 36 hours.

These results were compared with the results of regional regressions carried out on Adelaide Hills catchments, as shown in Table 11-4.

These include the Mount Barker Road Regression (BC Tonkin, 1991), Akter and Daniell (1993) and Eusuff (1995). The results show a higher flow predicted by RRR at the 5 year ARI than any of the regressions, but the 100 year ARI is comparable. This may be reasonable, given that the increase in flow due to the presence of Mount Barker Road should be most noticeable at the 5 year ARI level, with the effect reducing with increasing ARI. The peak flow for the 5 year ARI storm occurred for the 72 hour duration event, when the rural part of the catchment is contributing the most flow. It should be noted also that the catchment lies within the part of the Mount Lofty Ranges with the highest average intensities for long durations.

Table 11-4 Comparison of Predicted Flows at Ridge Park

Event	BC Tonkin (m^3/s)	Akter & Daniell (m^3/s)	Eusuff (m^3/s)	RRR (m^3/s) *PL1* = 0.78 *PL2* = 0.76 *PL3* = 0.78
5 yr ARI	2.8	4.5	3.4	5.3
100 yr ARI	14.1	13.8	9.9	12.3

The hydrology carried out for the Mount Barker Road design had an estimate for the 100 year ARI flow of 22.3m^3/s. This flow was derived by RORB, with the storage parameter adjusted such that lined channels were assumed in the catchment. However, this is not considered appropriate for Mount Barker Road where the still substantial rural catchment has to discharge with a normal lag to the piped system beneath the road.

Table 11-5 summarises the losses used in the calibration of the model.

Table 11-5 Adopted Losses for Calibration

Process	Initial Loss	Proportional or Continuing Loss
Rural Process 1	0mm	0.78
Process 2	25mm	0.76
Process 3	50mm	0.78
Urban Impervious	1mm	0mm/hr
Urban unconnected	40mm	3mm/hr

11.2.5 Model Calibration

11.2.5.1 General

The model as described in the previous section was calibrated on events for which good rainfall and flow data was available. The Bureau of Meteorology supplied rainfall and stream flow information for a total of five storms, in January and February 1997 and August, September and December 1993. The 1997 storms had data from a total of 5 gauging stations and 6 pluviometers, whereas the 1993 storms had data from only two gauging stations and two pluviometers.

The stations listed in Table 11-6 and Table 11-7 were used:

Table 11-6 Keswick Creek Catchment Rainfall Stations

	Station Number	Period of Record
Beaumont	BM023114	1997 only
Charles Street	BM023118	1997 only
Eagle on the Hill	BM023874	1993 and 1997
Glenside	AW504906	1997 only
Keswick	BM023115	1993 and 1997
Ridge Park	BM523100	1993 and 1997

Table 11-7 Keswick Creek Catchment Gauging Stations

	Station Number	Period of Record
Charles Street	BM023118	1997 only
Keswick	BM023115	1993 and 1997
Ridge Park	BM523100	1993 and 1997
Roberts Street	BM023119	1997 only
Victoria Park	AW504907	1993 and 1997

One gauging station, in Victoria Park near Fullarton Road was not rated and was therefore not used in the calibration.

Initial runs with the 1997 events indicated that the rating of Roberts Street was in error, with the flow gauged at Roberts Street being nearly three times that predicted by the model, even though the model predictions at the Keswick Creek gauging station, downstream of Roberts Street were reasonable.

Flows derived from a revised rating were received from the Bureau of Meteorology, which gave good fits for the two 1997 storms, and reasonable fits for the other storms, given the lower standard of the rainfall information.

In addition during the calibration a direct input was provided at Ridge Park to allow for the groundwater inflow. This was approximately 0.3 m^3/s for the events modelled.

Appendix 10 gives the plots of the measured and predicted hydrographs.

As a result of the calibration runs it was considered that the initial loss to be applied to the catchment upstream of Ridge Park should be higher than the losses determined from the fitting of the RRR model on the rural catchments in the Mount Lofty Ranges.

All the fitted storms on the Mount Lofty Ranges catchments occurred in the period July to October, apart from the December 1992 storm, which occurred during an unusually wet year. The design rainfall intensities are derived from all storm events, but the highest intensities occur during the summer period. The WBCM study recognised this effect by examining both the summer and winter periods, with different rainfall intensities and losses to account for the difference in rainfall and catchment behaviour.

It is considered that the fitted losses should be increased to account for the difference in catchment behaviour between the fitted events and the design events, with design rainfall intensities. The design rainfall intensities relate to rainfall bursts within a larger storm event, and so in general design rainfall losses can be expected to be larger than losses fitted to individual events. Hill et al (1998) discusses this problem.

Upstream of Ridge Park the January and February 1997 storm events had rainfalls of less than 23mm, and runoff occurred only from the impervious areas.

The sensitivity of the peak to Ridge Park was tested by varying each loss individually, whilst keeping all other losses constant in accordance with the following regime.

Table 11-8 Sensitivity Trial Values

Loss	Calibrated	Medium Increase	Large Increase
IL1	0	10mm	20mm
IL2	25mm	35mm	45mm
IL3	50mm	70mm	90mm
PL1	0.78	0.82	0.86
PL2	0.76	0.80	0.84
PL3	0.78	0.82	0.86

The resultant flows, in m^3/s are shown in Table 11-9, for the 100 year Average Recurrence Interval event. The initial estimate is $12.3m^3/s$.

Table 11-9 Predicted Flows with Sensitivity Adjustments

Parameter Changes	Medium Increase (m^3/s)	Large Increase (m^3/s)
IL1	12.1	11.9
IL2	11.6	11.0
IL3	12.3	10.8
PL1	12.0	11.7
PL2	11.6	10.8
PL3	11.4	10.8

It can be seen that the peak flow is reasonably insensitive to the adopted losses, but critical storm duration is longer with increased loss, and is up to 72 hours in some cases.

On the basis of the above, two possible scenarios were examined, for both 5 and 100 year Average Recurrence Interval events. The proportional losses remained the same, but the initial losses were increased. This was considered to be the most likely effect of dry catchment conditions at the commencement of the storm.

The losses considered were

IL1 = 10mm and *IL2* = 35mm, and
IL1 = 20mm and *IL2* = 45mm.

The results were as follows:

For the first case, Q5 = 4.2 m^3/s and Q100 = 10.6 m^3/s.
For the second case, Q5 = 3.5 m^3/s and Q100 = 10.2 m^3/s.

Comparing these flows with the regional regression flows and the nature of the catchment (with the Mount Barker road being a significant feature) it was decided to adopt the first case above, i.e.

IL1 = 10mm and *IL2* = 35mm.

Because the main contribution of the rural catchment is for long duration storms, it was considered that it may not be critical for the prediction of peak flows in Keswick Creek, which will be dominated by urban area flow.

Following this verification, it was decided to split the model design intensities to account for much higher long duration intensities in the catchment to Ridge Park. The above parameters were retained.

11.2.5.2 Losses Adopted After Calibration

The losses adopted following calibration on the January and February 1997 storms are shown in Table 11-10:

Table 11-10 Losses Adopted After Calibration

Process	Initial Loss	Proportional or Continuing Loss
Rural Process 1	10mm	0.78
Process 2	35mm	0.76
Process 3	50mm	0.78
Urban Impervious	1mm	0mm/hr
Urban unconnected	40mm	3mm/hr

11.2.6 Model Verification

The above calibrated model was then subject to validation, both with a 24 hour storm of ARI between 20 and 50 years that occurred during the period of the investigation, and with the evidence of historical flows through the showgrounds.

A storm occurred over the catchment on the 30th and 31st of October that could be used for the validation of the model.

Rainfall from the six pluviometers within the catchment ranged from 83.0mm to 127.0mm for a period of around 24 hours. The temporal pattern of the rainfall was fairly uniform throughout the storm, so that the ARI of short durations was less than 5 years, but the overall storm was of the order of 20-50 years.

Data for the six pluviometers and three gauging stations (Keswick, Charles St and Roberts St) were obtained from the Bureau of Meteorology.

It became clear that although the time to peak and the shape of the predicted hydrographs was good predicted flows were in excess of those measured at all gauging stations. Losses in October 1997 were not in accordance with those assumed after the calibration.

A systematic approach was used then to determine the actual parameters for the event, with a view to varying the design losses if necessary.

The initial approach was to remove all runoff from the unconnected portion of the urban area. Predicted flows were still in excess of gauged flows at the Charles Street gauging station. However, when the process 2 and 3 contributions were removed from the area above Ridge Park a good fit was achieved at Charles Street, but the predicted flow at the Keswick Creek station was less than that observed.

Observation during the storm on the morning of the 31st October indicated that the South Parklands were saturated and it was possible that runoff was occurring. It was decided therefore to provide a separate loss model for the unconnected portion of the two sub-areas having substantial parkland area.

It was determined that a continuing loss model produced peaks that were too high, but a proportional loss applied to the areas resulted in a good fit with the observed flow at the Keswick Creek station. The losses used were:

Initial Loss	30mm
Proportional Loss	0.75

This however led to over prediction of flows at the Roberts Street station, where the measured peak flow was 2.77m^3/s compared with the predicted flow of 6.6m^3/s.

As there was doubt with the accuracy of the measured flow an estimate of flow in the South Parklands at the entrance to the pipes beneath Greenhill Road was made based on the observed headwater depth at the pipe inlet. This resulted in a flow estimate of 6.8m^3/s, close to the predicted flow at Roberts Street.

Following examination of the total rainfalls, it was concluded that the Charles Street pluviometer may have registered less rainfall than actually occurred (86.8mm compared with the closest stations being 107.6mm and 127.0mm). In discussion with the staff of the Hydrology Section, Bureau of Meteorology it was noted that there is a large tree on the north east side of the rain gauge that could have influenced the record at this site.

The rainfall at Charles Street was increased by 30% to a total of 112.8mm, and the model again run. There was still the indication of unconnected area runoff from the parklands, but with losses as follows:

Initial Loss	35mm
Proportional Loss	0.80

The fit at the Keswick Creek station was marginally improved. Appendix 10 gives the final fitted hydrograph.

As a result of the verification it was decided to review the unconnected area loss rates, with reference to the historical evidence at the showgrounds.

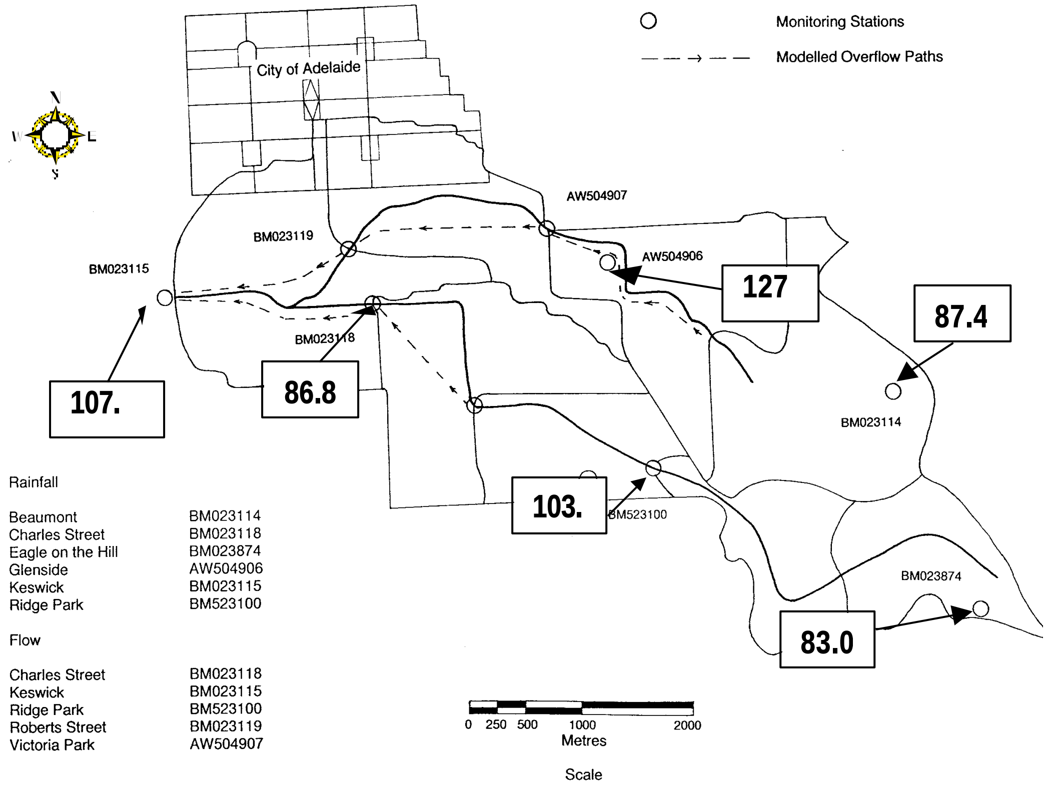


Figure 11-3 Rainfall (mm) Recorded for Storm of 31/10/97

The capacity of the tunnel beneath the showgrounds is 25m^3/s, according to the WBCM report. A copy of the Advertiser dated Friday 14 February 1913 contained the following report:

"A little lower down a broad sheet of water was to be seen rushing over Goodwood Road at a terrific pace, and the creek as it passed through the Royal Agricultural Society's new show ground had all the dimensions and appearance of a river."

However, a review of the Agricultural Society's records revealed no evidence of problems within the showgrounds since the tunnel was constructed in 1915. The show however did not move to its present site until 1928 (pers. comm. RAHS Archives staff).

In the 82 years since the tunnel was built beneath the showgrounds there is a greater than 90% chance of the 50 year Average Recurrence Interval event having occurred, and so it would be expected that there would be a record of overflows through the showground.

Mr Chris Tually of the Unley Council has 20 years experience of the flooding history in the Unley Council area and recalls water only once flooding Goodwood Road and backing up to the east. Water did not enter the Showgrounds. This was probably the March 1983 event.

One reason for the lack of evidence of water crossing Goodwood Road may also be that the catchment has been changing over the years, with more impervious area being created by closer development, and the increase in directly connected area as more of this development is connected to the street system.

The above evidence indicates that the 50 year ARI event produces only small flows across Goodwood Road. The model should produce similar results. An initial loss of 45mm and a proportional loss of 0.8 were chosen for testing on the unconnected area, based on the October 1997 storm and the evidence from the Glenelg and Paddocks catchments

A review of the two storm events modelled by RRR and having unconnected area runoff at the Glenelg catchment had proportional losses of 0.95 and 0.75.

This proportional loss rate on the unconnected area is also supported by Burfill and Boyd (1992) who found that for a selection of 13 catchments in 5 countries that the mean runoff coefficient for the unconnected areas was 0.24, leading to a proportional loss of 0.76.

The model was run to determine flows at Goodwood Road, for a range of recurrence intervals to determine the unconnected area runoff and to compare with the WBCM flows. The Glenside basin was not included in the model. It was found that in all cases the 90 minute storm produced the highest flows in all cases. Table 11-11 summarises the flows. The 18 hour storm is also included in the table to indicate the result of a longer duration storm.

Table 11-11 Comparison of Flows at Goodwood Road

ARI years	WBCM Flow (m^3/s)	90 minute			18 hour		
		Rainfall (mm)	Flow (m^3/s)	Grassed Runoff (mm)	Rainfall (mm)	Flow (m^3/s)	Grassed Runoff (mm)
5	20.5	25.3	20.4	0.0	58.0	12.7	2.6
20	27.4	35.0	27.3	0.0	78.7	17.1	6.7
50	32.1	45.3	30.5	0.0	96.3	23.9	10.3
100	37.1	53.2	39.2	1.6	111.0	30.6	13.2

This indicates that overflows will commence over Goodwood Road at the 20 year Average Recurrence Interval.

The predicted peak flows are close to those of WBCM, and little unconnected area runoff occurs during critical storm events. Thus the predicted peak flows are insensitive to unconnected area runoff.

Factors that may account for the lack of observed flow through the showground could include:

- The catchment has been subject to change over the period due to urbanisation with the attendant provision of stormwater drainage infrastructure. The RRR model accounts only for current catchment characteristics;
- The record of flooding through the showgrounds would be present only if direct damage occurred. It may be that flows have occurred that have not been recorded;
- The model assumes that overflows within the catchment will reach the Showgrounds. It may be that there are flows leaving the catchment in major events; or
- The Australian Rainfall and Runoff rainfall intensities and temporal patterns do not reflect actual events, in which case the design storm is not producing a flow that would be produced by an actual storm of the same recurrence interval.

The sensitivity of the model to the unconnected area loss rate was assessed by comparing flows with the 45mm Initial Loss / 0.8 Proportional Loss and a model having no unconnected area runoff for the 3 hour storm, and recurrence intervals of 50 and 100 years

The predicted peak flows in m^3/s are given in Table 11-12:

Table 11-12 Keswick Creek Predicted Peak Flow Sensitivity to Loss

	Ridge Park (m^3/s)	Charles Street (m^3/s)	Victoria Park (m^3/s)	Roberts Street (m^3/s)	Goodwood Road (m^3/s)
50 Year ARI					
45/0.8 No Contribution	3.9 3.9	14.9 14.9	14.0 14.0	14.0 12.6	27.5 27.5
100 year ARI					
45/0.8 No Contribution	5.3 5.3	17.5 17.5	14.3 14.3	15.3 13.7	32.8 29.5

The model is relatively insensitive to the unconnected area loss rate, and thus predicted flows at Goodwood Road, and the Showground are reasonable. Design runs were carried out with the updated loss model for the urban areas.

The sensitivity of the model to the storage delay time in the overflow paths was assessed by the increase in storage delay time for each path by 50%. Table 11-13 gives the predicted flows.

Table 11-13 Sensitivity of Model to Overflow Storage Delay Time

ARI (years)	Baseflow (m^3/s)	+50% Overflow Delay Time (m^3/s)
5	20.4	20.3
20	27.3	26.9
50	30.5	29.5
100	39.2	34.2

The effect of a greater storage delay time for the overflow paths increases with increasing Average Recurrence Interval, but there is not a substantial effect on the Average Recurrence Interval at which overflow commences at Goodwood Road. The initial storage delay times were retained.

Following the verification on the October 1997 storm the losses have been adjusted as shown in Table 11-14 to use in the design model

Table 11-14 Adopted Losses for Design Runs

Process	Initial Loss	Proportional or Continuing Loss Rate
Rural Process 1	10mm	0.78 (proportional)
Process 2	35mm	0.76 (proportional)
Process 3	50mm	0.78 (proportional)
Urban Impervious	1mm	0mm/hr
Unconnected (including Parkland)	45mm	0.80 (proportional)

11.2.7 Model Results

The final design model has been run for a range of storm durations and recurrence intervals to determine both peak flow and time to peak at a number of locations.

In the design model the detention basin to be built in conjunction with the Adelaide Crafers Highway works has been included. Since peak flows in the urban area occur with short duration rainfall events the Adelaide Crafers Highway detention basin is likely to have only a small effect on peak flows. For example at Charles Street the predicted flow for a 2 hour, 50 year ARI storm is 15.4m^3/s both with and without the basin.

The model has produced flows for durations of 30 minutes to 24 hours, for recurrence intervals of 50, 100 and 200 years. Maximum flows at five locations have been shown in Figure 11-4 to Figure 11-6. It should be noted that these are maximum potential flows, and may not represent the actual channel flows, due to limited channel capacity. Flows in excess of the channel capacity will be carried through local streets, or in low areas adjacent to the channel.

At Goodwood Road flows in excess of 25m^3/s (the capacity of the showgrounds tunnel) will enter the Showgrounds.

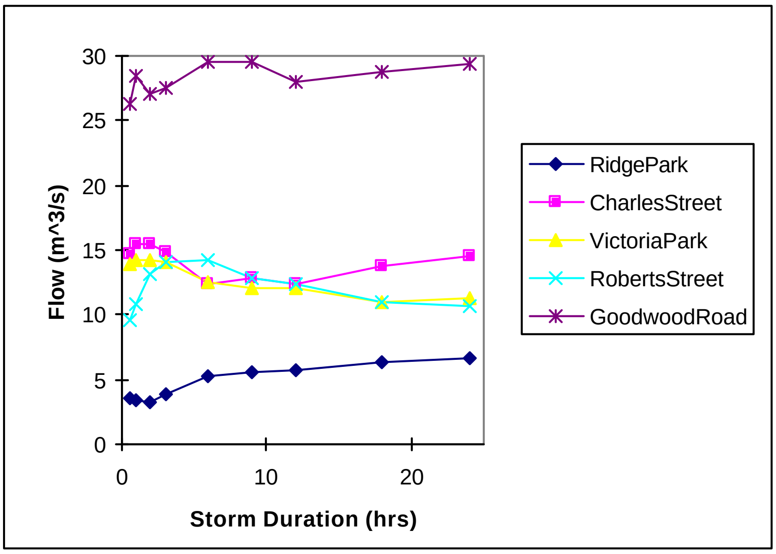


Figure 11-4 Keswick Creek Maximum Potential Flow - 50 year ARI

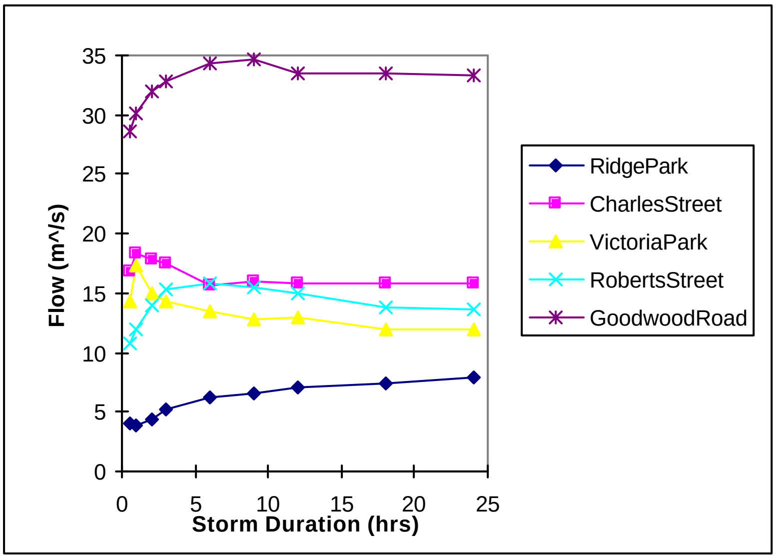


Figure 11-5 Keswick Creek Maximum Potential Flow - 100 year ARI

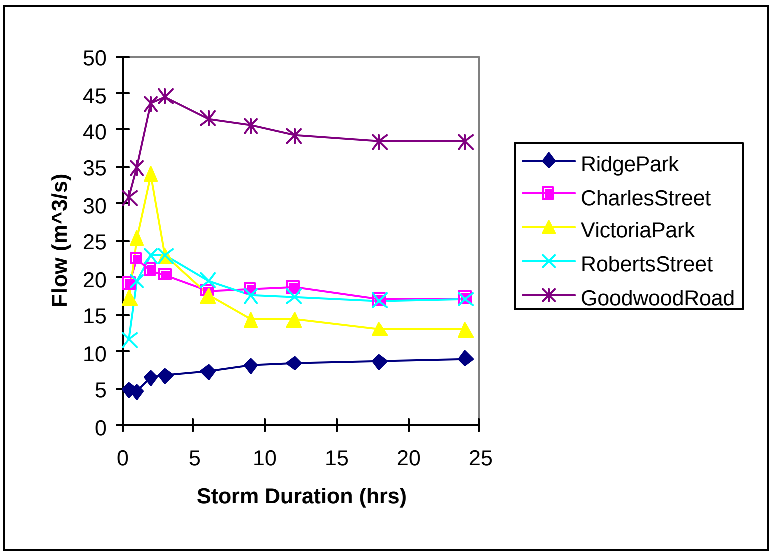


Figure 11-6 Keswick Creek Maximum Potential Flow - 200 year ARI

11.3 Brownhill Creek

11.3.1 Introduction

Flood mapping of Brownhill Creek carried out in 1984 (WBCM, 1984) indicated that significant areas of the western suburbs are at risk of flood damage.

As with the Keswick Creek catchment the work was carried out by WBCM Consultants, using limited information. The catchment is shown on Figure 11-7. No flow data was available for the catchment, and the nearest pluviometers were located at Kent Town, Stirling and at the Waite Institute at Urrbrae. The study report drew attention to the lack of event data for the catchment and proposed that further work be done to confirm the extent and frequency of flooding.

This review has been carried out to provide a current assessment of flood risk as input to the Brownhill Creek Water Management Plan, produced by ID&A on behalf of the Patawalonga Catchment Water Management Board (ID&A, 1998).

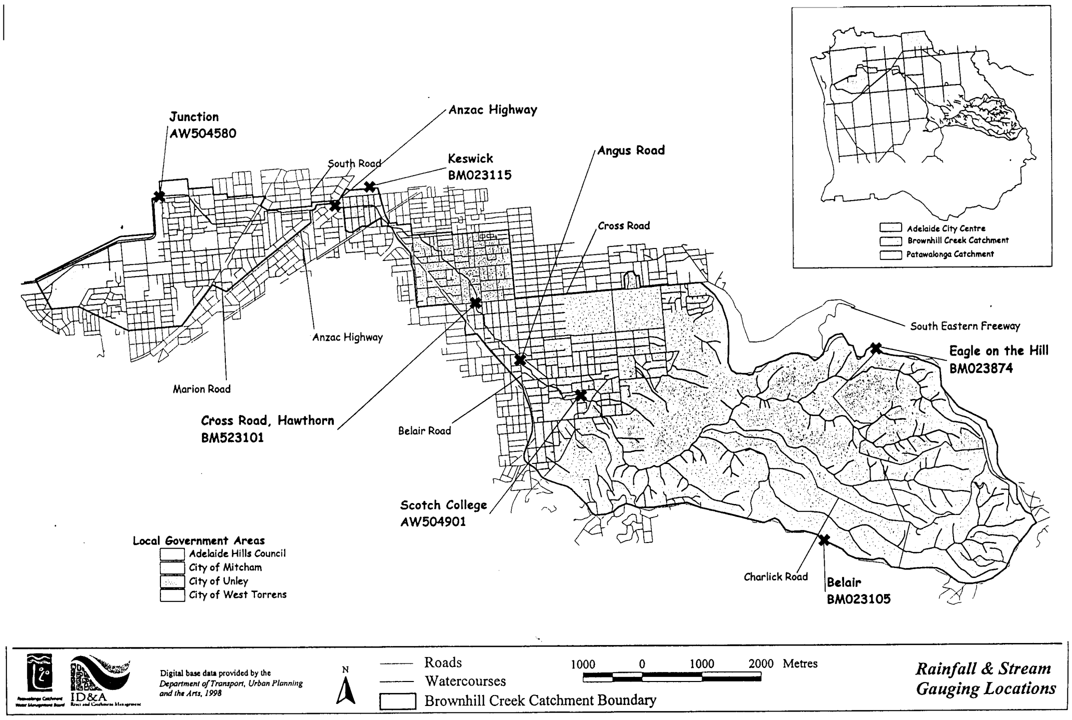


Figure 11-7 Brownhill Creek Catchment (After ID&A, 1998)

11.3.2 Approach

The approach adopted was to set up a RRR model based on previously derived parameter values for the directly connected impervious area, the storage parameters and the channel characteristic velocities.

A separate smaller model was set up for the catchment to the Scotch College gauging station, and calibration carried out on 7 storm events producing the largest flows between 1991 and 1997.

The full model was then run for three storm events in 1997, with the results being compared at three gauging stations.

The calibrated parameters were used with design rainfall events, and the results compared with flows derived by flood frequency analysis at Scotch College.

The model parameters were adjusted such that the model produced results consistent with the historical evidence, whilst at the same time having parameters that could be considered to be reasonable, given the calibrated parameters from both the Brownhill Creek catchment and other catchments that have been subject to calibration.

11.3.3 Features of the Catchment Incorporated in the Model

11.3.3.1 General

The RRR model as set up for the Brownhill Creek has similarities to the Keswick Creek model in the structure of the model, which allows for surcharge flows within the urban area.

The main difference between the two catchments, which are adjacent, is that the Brownhill Creek catchment is approximately half rural (16.4 km^2 out of 32km^2). This necessitated an approach of calibration of the rural catchment first, followed by the calibration of the urban catchment.

In all a total of 9 sub-areas covered the catchment area.

11.3.3.2 Urrbrae Wetland

The City of Mitcham has constructed a wetland on land on Cross Road, with construction commencing in March 1997. The basin has been incorporated into the design model.

Storage - elevation data for the wetland was obtained from the calculations undertaken by Kinhill Engineers and incorporated into the model.

Peak inflows and outflows have been compared for the design events.

11.3.3.3 Brownhill Creek Upstream of Scotch College

Brownhill Creek upstream of Scotch College is essentially rural, and the model was initially set up as such.

Initial calibration runs using the 1997 storms indicated that there was a significant contribution from the small urban area near Scotch College, so a separate contribution with urban parameters was added to the model. The balance of the catchment was treated as a normal RRR model, but using 5 channel storages instead of the normal 10.

11.3.4 Parameters

As the Brownhill Creek catchment is adjacent to the Keswick Creek catchment storage and loss parameters were initially selected based on the Keswick Creek modelling. The losses selected as initial values to be used in the calibration of the model are given in Table 11-15.

Table 11-15 Losses for Calibration

Process	Initial Loss	Proportional or Continuing Loss Rate
Rural Process 1	0mm	0.78 (proportional)
Process 2	25mm	0.76 (proportional)
Process 3	100mm	0.78 (proportional)
Urban Impervious	1mm	0mm/hr
Unconnected	45mm	0.8 (proportional)

11.3.5 Model Calibration and Verification

11.3.5.1 Rural Catchment

The RRR model for the catchment to Scotch College was calibrated on selected events. The Bureau of Meteorology supplied rainfall and stream flow information for a total of seven storms, from September 1991 to October 1997.

The following stations were used:

Table 11-16 Scotch College Rainfall Stations

	Station Number	Period of Record
Eagle on the Hill	BM023874	1997 only
Belair	BM023846	all events
Scotch College	BM023105	all events

Table 11-17 Scotch College Gauging Station

	Station Number
Scotch College	AW504901

The results from the calibration events are attached as Appendix 10. The Event of July 1993 could not be fitted, probably due to the insufficient areal definition of catchment rainfall.

It was found that all events could be fitted well with one set of storage and channel lag parameters, with only losses changing from event to event. This stability in parameters is very welcome as it indicates that the model is functioning well on the catchment. It is expected that the calibrated loss will change from event to event, due to changes in antecedent catchment conditions and the problem of adequate areal definition of rainfall across the catchment.

The fitted parameters are given in Table 11-18:

Table 11-18 Results of Calibration at Scotch College

Start Date	c_p1	c_p2	v_c	IL1 (mm)	PL1	IL2 (mm)	PL2
14/09/91	1.72	0.46	1.24	0	0.65	20	0.76
28/08/92	“	“	“	“	0.60	15	0.85
03/10/92	“	“	“	“	0.88	25	0.65
02/08/96	“	“	“	“	0.90	15	0.70
20/08/96	“	“	“	“	0.90	10	0.70
29/10/97	“	“	“	“	0.97	20	0.95
Mean	1.72	0.46	1.24	0	0.82	17.5	0.77

There was no indication of process 3 (fast flow) occurring for any of the events modelled.

Baseflow was occurring at the commencement of all the recorded storm events, so the initial loss on the first process was zero. In some cases it was also necessary to add a continuous flow (less than $0.4 m^3/s$) to the modelled flow to allow for the antecedent baseflow. The mean parameters are close to those of the other Adelaide Hills catchments, apart from the process storage parameters (c_p1 and c_p2), which are double those previously found. However the ratio between these two parameters is consistent with the ratio of the two parameters for the other hills catchments. Preliminary calibration runs of the RRR model on the First Creek catchment also show this larger process lag. The difference may be due to differences in catchment soils or geology.

11.3.5.2 Total Model

The RRR model including the urban catchment to the Keswick Creek junction was calibrated on three storm events in 1997. For two of the events, in January and February there was no contribution from the rural catchment upstream of Scotch College. The October event produced a flow in the rural catchment, and the rural best fit parameters from above were used for the rural part of the catchment.

The following stations were used in the calibration:

Table 11-19 Brownhill Creek Rainfall Stations

	Station Number
Eagle on the Hill	BM023874
Belair	BM023846
Scotch College	BM023105
Hawthorn	BM523101
Keswick	BM023115

Table 11-20 Brownhill Creek Gauging Stations

	Station Number
Scotch College	AW504901
Hawthorn	BM523101
Upstream Keswick Ck. Junction	AW504580

The gauging station just upstream of the Keswick Creek Junction was not functioning for much of the January 1997 storm event. This gauging station lies on a straight, uniform section of channel. The gauging station at Hawthorn is on an irregular natural channel, and thus the rating of the station is less reliable than the station near the junction.

A reasonable fit was achieved at the Scotch College station for the January 1997 storm event, but the flow predicted at Hawthorn was greater than that recorded (6.2 vs 4.0 m^3/s). The shape of the hydrograph at Hawthorn was however satisfactory.

The February storm modelled with RRR again produced hydrographs with reasonable shape and time to peak. However, the flow was underestimated at Hawthorn and overestimated at Scotch College and the junction.

The October 1997 storm event produced the best fit, with the hydrographs at Scotch College and the Junction being good representations. However the predicted peak flow at Hawthorn (8.4m^3/s) was larger than the recorded peak flow (3.1m^3/s). In this event it was necessary to include unconnected area runoff within the urban area, with an initial loss of 40mm, and a proportional loss of 0.83. This was close to the values initially chosen. Appendix 10 has a copy of the measured and predicted hydrographs.

11.3.6 Flood Frequency Analysis at Scotch College

Flood frequency analysis has been carried out on the 8 full years of flow data available at Scotch College, with the addition of one historical event in 1981 that was described in the WBCM report. Analysis was carried out in accordance with the procedures outlined in Australian Rainfall and Runoff. The results are shown on Figure 11-8.

The number of years of record is short, so the results of the analysis must be seen as only one part of the evidence to arrive at estimates for design flood flows.

The 1981 flow estimated at 16m^3/s is by far the largest flow in the period of record. The maximum annual flows are as follows:

Table 11-21 Ranked Flows at Scotch College for Flood Frequency Analysis

Year	Flow (m^3/s)	Rank
1981	16	1
1992	5.0	2
1991	4.8	3
1996	4.1	4
1993	3.7	5
1995	3.4	6
1990	2.0	7
1997	1.2	8
1994	0.9	9

The resultant frequency distribution was nearly log normal (skew = 0.047) in contrast to most South Australian catchments, which show negative skews.

The predicted flood frequency flows are given in Table 11-22.

Table 11-22 Flood Frequency at Scotch College

AEP	ARI (years)	Flow (m^3/s)	5% Confidence Limit (m^3/s)	95% Confidence Limit (m^3/s)
0.01	100	21.7	115.6	6.5
0.02	50	16.6	68.7	6.4
0.05	20	11.2	34.3	5.7
0.1	10	8.0	20.2	4.9
0.2	5	5.5	11.8	3.7
0.5	2	2.8	5.3	1.9

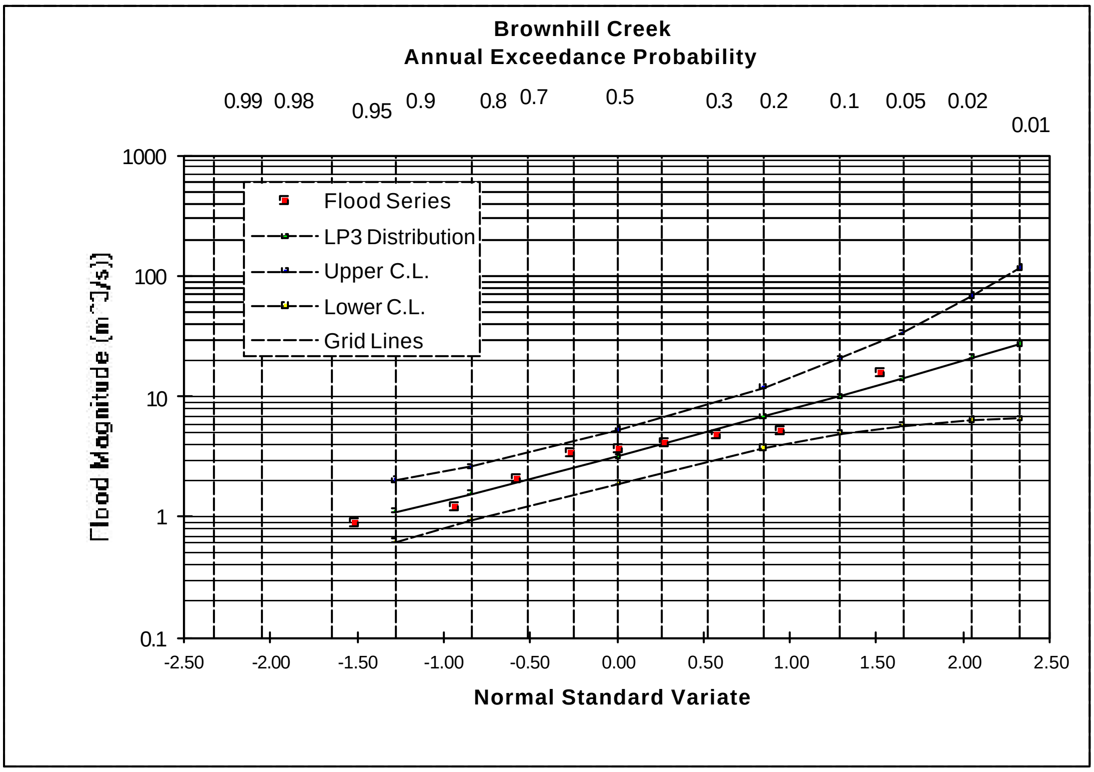


Figure 11-8 Brownhill Creek at Scotch College Flood Frequency

11.3.7 Other Historical Evidence

Reference can be made to the storm event that occurred on 2 July 1981 that was documented in the WBCM report.

The event produced a peak flow estimated at 16m^3/s at Scotch College, from a rainfall at Stirling of 74.0mm. The catchment was very wet at the commencement of the storm, having recorded 150mm of rain in the previous 10 days.

Hourly rainfalls at Stirling are given in the WBCM report are shown in Table 11-23:

Table 11-23 Stirling Rainfalls for 2 July 1981

Time	Rainfall (mm)
05:00 - 06:00	3.5
06:00 - 07:00	19.0
07:00 - 08:00	24.0
08:00 - 09:00	11.5
09:00 - 10:00	3.0
10:00 - 11:00	2.0
11:00 - 12:00	2.5
12:00 - 13:00	4.0
13:00 - 14:00	4.5

From these hourly rainfalls the recurrence interval of the storm rainfall are determined as shown in Table 11-24:

Table 11-24 Recurrence Interval of 2 July 1981 Rainfall

Duration	Rainfall Intensity (mm/hr)	Recurrence Interval (years)
1 hour	24	5
2 hours	21.5	20
3 hours	18.1	30
9 hours	8.2	15

Given the catchment condition at the start of the rainfall it would be expected that the peak flow from the storm would be in excess of 20 year Average Recurrence Interval.

Reference can also be made to regional flood frequency analysis, where flood flows are determined based on relationships between the catchment of interest and other catchments that are considered to be hydrologically similar.

The two most recent regional flood frequency analysis are that of Akter and Daniell (1993) and Eusuff (1995), where a relationship was found between flow and catchment area for a range of ARI's.

The predicted flows for the Brownhill Creek catchment to Scotch College using these relationships are given in Table 11-25:

Table 11-25 Flows at Scotch College predicted by Regional Flood Frequency Analysis

ARI (years)	Flow (m^3/s) Akter & Daniell	Flow (m^3/s) Eusuff
5	11.6	11.7
10	14.8	16.1
20	not available	20.2
50	25.7	25.0
100	31.4	29.8

The flows predicted by both Akter & Daniell (1993) and Eusuff (1995) are greater in magnitude than those by direct flood frequency, and the distribution is negatively skewed in contrast to the on-site frequency analysis.

11.3.8 Selection of Design Loss Parameters

Selection of design loss parameters should follow those found by calibration, adjusted to account for catchment condition at the commencement of the design rainfall events, and to account for catchment historical behaviour.

The catchment to Scotch College was used for the selection of rural loss parameters. The model was reduced from three to one sub-area above Scotch College as for the derivation of design flows it was not necessary to model non-uniform rainfall. At the same time the number of channel storages in the one sub-area was increased from 5 to 10, as would normally be used for the RRR model.

The sensitivity of the model to the change was determined by using the selected storage parameters, with an assumed rainfall pattern (100 year, 36hr duration Australian Rainfall and Runoff design storm) and preliminary losses. There was very little effect, as shown by Figure 11-9.

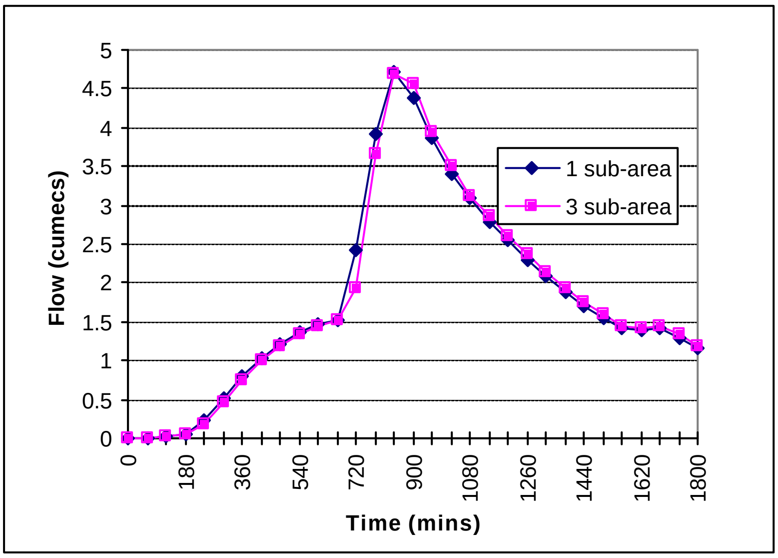


Figure 11-9 Scotch College RRR Model Sensitivity Check

The catchment to the Scotch College gauge is substantially rural, but has approximately 40 ha of urban development, mainly near the gauging station. For the unconnected area within the urban development an initial loss of 40mm was used, with a proportional loss of 0.8, based on the losses adopted in the Keswick Creek review. The rural process loss parameters were then adjusted to match the frequency analysis and other historical evidence. The 5 and 100 year ARI flows were chosen to undertake this analysis.

All the storms that were calibrated occurred in the period July to October. Design rainfall intensities are derived from all storm events, but the highest intensities occur during the summer period. The WBCM study recognised this effect by examining both the summer and winter periods, with different rainfall intensities and losses to account for the difference in rainfall and catchment behaviour.

Therefore the fitted losses should be increased to account for the difference in catchment behaviour between the calibration events and the design events, with design rainfall intensities. This is particularly the case with initial losses, which are known to have a strong relationship to catchment Antecedent Moisture Condition (AMC).

For this investigation the losses were adjusted such that they accord generally with the fitted losses and to give results in accordance with historical flows (based on flood frequency analysis at Scotch College) and the regional regression when used with design rainfalls. A number of possible loss scenarios were investigated to choose the most appropriate model.

Since the process 3 runoff was not observed for any of the calibration events the initial values selected were based on observed values for other catchments and storm events that have been calibrated with the RRR model. However as a first estimate 100mm was used as no process 3 runoff was observed in October 1997, with a rainfall in excess of 100mm.

Table 11-26 Trial Loss Parameter Values for the Rural Catchment

Scenario	IL1 (mm)	IL2 (mm)	IL3 (mm)	PL1	PL2	PL3	Q5 (m^3/s)	Q100 (m^3/s)
1.	10	35	100	0.82	0.77	0.78	4.7	21.1
2.	10	35	50	0.82	0.77	0.78	7.7	26.0
3.	0	35	50	0.82	0.77	0.78	7.9	26.2
4.	0	35	50	0.78	0.76	0.78	8.3	27.0
5.	10	35	50	0.78	0.76	0.78	8.1	26.7
6.	10	35	100	0.82	0.77	0.78	7.6	29.7

These scenarios can be described as follows:

1. Initial losses for process 1 and 2 as per Keswick Ck review, process 3 initial loss first estimate, with calibrated proportional losses for process 1 and 2, and an estimate for process 3. Based on increasing the initial losses only to account for the likely antecedent conditions for a design rainfall event.

2. As for scenario 1, but with the process 3 initial loss lowered to 50mm, as was used in the Keswick Creek review.

3. Scenario 2 but with baseflow occurring at the commencement of the storm (IL1 = 0)

4. As for scenario 3, but for the use of proportional losses as per the Keswick Creek review.

5. All losses as per the Keswick Creek review

6. As for scenario 1, but with the process storage lags halved to normal values for hills catchments. This allows a comparison with Eusuff.

The model has produced flows for durations of 30 minutes to 72 hours, for recurrence intervals of 50, 100 and 200 years. It was found that in all cases the peak flow occurred as a result of the 72 hour duration storm, with the second highest peak occurring with the 36 hour storm.

As the catchment is not very large it would not be expected that the critical storm duration is 72 hours. An investigation was therefore carried out into the possible reasons (design rainfalls, temporal patterns and the model itself) that could be causing the effect.

The temporal patterns were first checked to see if full filtering was necessary to ensure that long duration design storms of Australian Rainfall and Runoff did not have rainfall bursts of greater intensity than those of shorter duration storms. It was determined that no filtering was necessary.

To determine whether it was the RRR model that was causing the effect a single node RAFTS model was set up for the catchment to Scotch College and peak flows determined for the range of storm durations.

The single node RAFTS model had the following properties:

B	0.782
n	- 0.2
IL	35mm
CL	7mm/hr

The value of B is equivalent to the RORB k_c value of 3.91 that is derived from a regional regression for the Adelaide Hills derived by Maguire et al (1986)

Table 11-27 summarises the results obtained.

Table 11-27 Brownhill Creek at Scotch College - Design Flows

Model	24 hour (m^3/s)	36 hour (m^3/s)	48 hour (m^3/s)	72 hour (m^3/s)
RRR	23.8	25.9	24.5	28.4
RAFTS	46.8	50.1	46.7	50.4

It can be seen that the same pattern of peak flows exists for the RAFTS model.

It can be seen that it is not a problem that can be attributed to either the temporal patterns or model. It was decided that for the prediction of flows in Brownhill Creek that the 72 hour duration design storm would not be used. The critical storm duration then becomes 36 hours, with a lower flow at 48 hours.

Given the results of the trial parameter values, and comparing them with historical evidence and the regional regressions, it was determined that scenario 2 should be adopted. This gives flows higher than but still consistent with the flood frequency analysis, and lower than the regional regressions. This is expected, due to the greater process storage lags than other Adelaide Hills' catchments.

Losses for the urban areas are as for the Keswick Creek review, which were supported by the historical evidence of the October 1997 storm event.

As another test of the selected parameters with regard to historical evidence the selected parameters were used in the prediction of the 20 year ARI, 36 hour storm event. Table 11-28 lists the predicted flows in m^3/s.

Table 11-28 Predicted Flows for 20 Yr ARI, 36 Hour Storm

	Scotch College (m^3/s)	Angas Road (m^3/s)	Cross Road (m^3/s)	Anzac Highway (m^3/s)	Junction (m^3/s)
RRR Flow	15.0	16.9	20.3	21.4	21.1
1981 flow (WBCM)	16.0	15.4	19.1	18.2	17.4

The predicted flows are of the order of the capacity of the channel, and slightly in excess of the 1981 flows, which had a maximum rainfall recurrence interval of 30 years for the 3 hour duration. As this duration is less than the critical duration for the catchment it would be expected that the flows are less than 30 year ARI, and are most probably 15 - 20 years ARI, confirming that the parameter values are reasonable.

11.3.9 Adopted Losses for Design Runs

The adopted loss parameters are given in Table 11-29:

Table 11-29 Adopted Losses for Design Runs

Process	Initial Loss	Proportional or Continuing Loss
Rural Process 1	10mm	0.82 (proportional)
Process 2	35mm	0.76 (proportional)
Process 3	50mm	0.78 (proportional)
Urban Impervious	1mm	0 mm/hr
Unconnected	45mm	0.80 (proportional)

11.3.10 Model Results

The final design model has been run for a range of storm durations and recurrence intervals to determine both peak flow and time to peak at a number of locations.

Maximum flows at five locations have been shown in Figure 11-10 to Figure 11-12. It should be noted that these are maximum potential flows, and may not represent the actual channel flows, due to limited channel capacity. Flows in excess of the channel capacity will be carried through local streets, or in low areas adjacent to the channel.

Table 11-30 summarises the predicted peak flows at selected locations on Brownhill Creek. These flows are potential flows, and are not necessarily contained within the channel.

Table 11-30 Predicted Peak Flows at Selected Locations

ARI	Scotch College (m^3/s)	Angas Road (m^3/s)	Cross Road (m^3/s)	Anzac Highway (m^3/s)	Junction (m^3/s)
20	15.0	16.9 (20.5)	20.3 (22.0)	21.4 (22.4)	21.1 (21.7)
50	20.9	23.5 (29.1)	27.6 (31.3)	27.8 (31.9)	27.3 (30.9)
100	25.9	29.0 (36.9)	33.8 (40.3)	34.0 (40.6)	33.4 (39.6)
200	31.4	35.3 (46.2)	41.1 (50.4)	41.3 (51.2)	40.5 (50.2)

Note: WBCM (1984) flows in brackets

The increase in the magnitude of the difference between the WBCM flows and the predicted peak flows may be attributed to the difference in the models used, with the RRR model being able to better model flows in excess of the pipe and channel capacities.

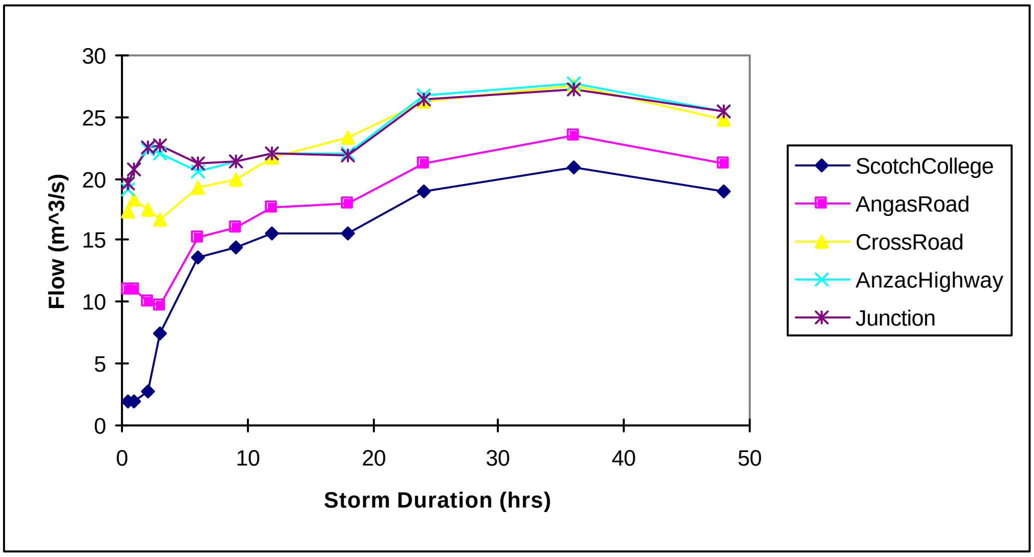


Figure 11-10 Brownhill Creek Maximum Potential Flow - 50 Year ARI

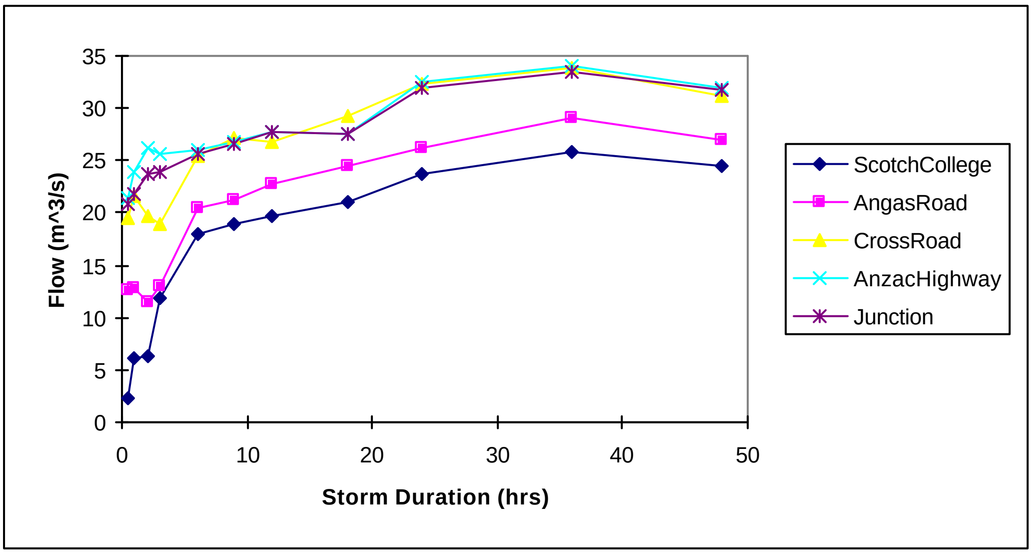


Figure 11-11 Brownhill Creek Maximum Potential Flow - 100 Year ARI

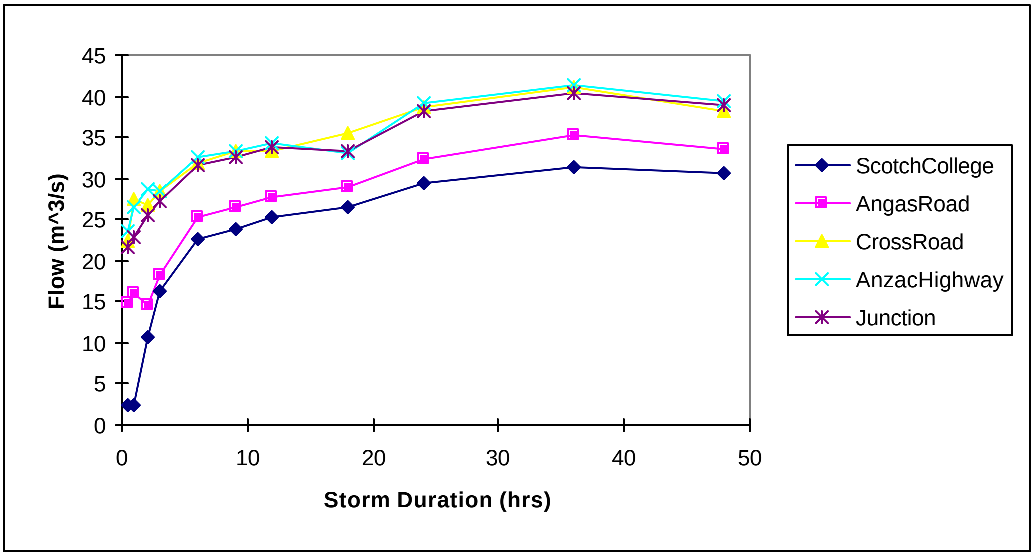


Figure 11-12 Brownhill Creek Maximum Potential Flow - 200 Year ARI

11.4 Probable Maximum Flood (PMF)

Following the completion of the analysis of Keswick and Brownhill Creek floodplain mapping was carried out. It was decided that a simplified PMF would be mapped. To undertake a rigorous analysis would require the calculation of PMP at every site of interest, apply the rainfall isohyets applying to the contributing catchment and produce a PMF hydrograph. There are 17 such sites.

It was considered that the floodplain for the PMF would be relatively insensitive to the flow, and this fact together with the possibility of inflows from other catchments and the uncertainties in the prediction of the PMF led to the adoption of the simplified approach.

It was decided to map a single event covering the entire catchment, and to assume a uniform rainfall distribution. BC Tonkin in the analysis of the River Sturt (BC Tonkin, 1996b) found that for catchments less than 100km^2 there is no need to calculate spatial variations in PMP for input into a rainfall – runoff model to derive PMF, since resulting increases in the PMF are minimal.

PMP estimates for short duration storms (less than 3 hours) were derived using the procedures of the Bureau of Meteorology publication Bulletin 53 – "The Estimation of Probable Maximum Precipitation in Australia: Generalised Short-Duration Method". The procedure was amended in accordance with the amendment published in December 1996.

The catchment is considered to be rough, as all the catchment lies within 20km of terrain that can be considered to be rough. No elevation adjustment is required. A moisture adjustment factor of 0.65 was adopted. Because of the size of the catchment durations of 1, 1.5, 2, 2.5 and 3 hours were assessed. The mean catchment rainfall depths were calculated as follows in Table 11-31:

Table 11-31 Brownhill Creek Short Duration PMP Estimates

Duration (hours)	Mean Depth (mm)
1.0	268
1.5	338
2.0	387
2.5	432
3.0	466

The temporal patterns for short duration storms (<3 hours) were taken from Bulletin 51.

For the conversion of PMP to a PMF the rainfall must be applied to the hydrological model, with an appropriate adjustment to losses to account for the low probability of the event.

The following losses have been applied to the design rainfalls up to 500 year ARI:

Table 11-32 Design Losses for Frequent Events

Process	Initial Loss (mm)	Proportional Loss
Rural process 1	10	0.82
Rural process 2	35	0.76
Rural process 3	50	0.78
Urban impervious	1	0
Urban unconnected	45	0.8

The procedure used to derive losses for the PMF was from Book 6 of Australian Rainfall & Runoff. Book 6 recommends the use of a continuing loss rather than proportional loss, on the basis of the interpretation that the proportional loss as the unsaturated part of the catchment. As storm magnitude increases the unsaturated proportion decreases, and thus proportional loss reduces. It is difficult to extrapolate the rate. It is thus recommended that a small continuing loss (say 1mm/hr) be used instead for extreme events. A nominal baseflow is then added to obtain an estimate of the total flow.

The difficulty with the RRR model is that three processes are being modelled with a proportional loss model. It is difficult to see that processes 1 and 2 (related to baseflow and slow flow) will give an increasing contribution at extreme events. To follow the principles of Book 6 of Australian Rainfall and Runoff it would be more appropriate to assign a relatively large loss to processes 1 and 2, and a small loss to process 3. It is proposed initially to allow a proportional loss of 0.95 for processes 1 and 2, and 0.10 for process 3.

For the unconnected area it can be assumed that all rainfall appears as runoff, with the proportional loss being zero. The initial loss for all processes can be considered to be zero.

It was thus proposed to use the following losses for PMF:

Table 11-33 PMF Losses fror Brownhill Creek

Process	Initial Loss (mm)	Proportional Loss
Rural process 1	0	0.95
Rural process 2	0	0.95
Rural process 3	0	0.10
Urban impervious	0	0
Urban unconnected	0	0

The results of the prediction of PMF at the Scotch College, and for the whole catchment are given in Table 11-34 and Figure 11-13. It should be noted that the channel capacities through the metropolitan area are very small compared with the PMF, and thus most of the flow will be on a floodplain. The analysis assumes that all flow remains within the Brownhill Creek catchment, which is very unlikely to occur. There is also substantial storage on the floodplain, more so than if the flood was confined to a channel. The flow can thus be considered to be preliminary, and subject to confirmation following hydraulic analysis.

Table 11-34 Brownhill Creek PMF

Duration (hours)	Peak Flow (Scotch) (m^3/s)	Peak Flow (Outlet) (m^3/s)
1	643	1023
1.5	729	1258
2	754	1429
2.5	751	1566
3	728	1648
6	472	1458

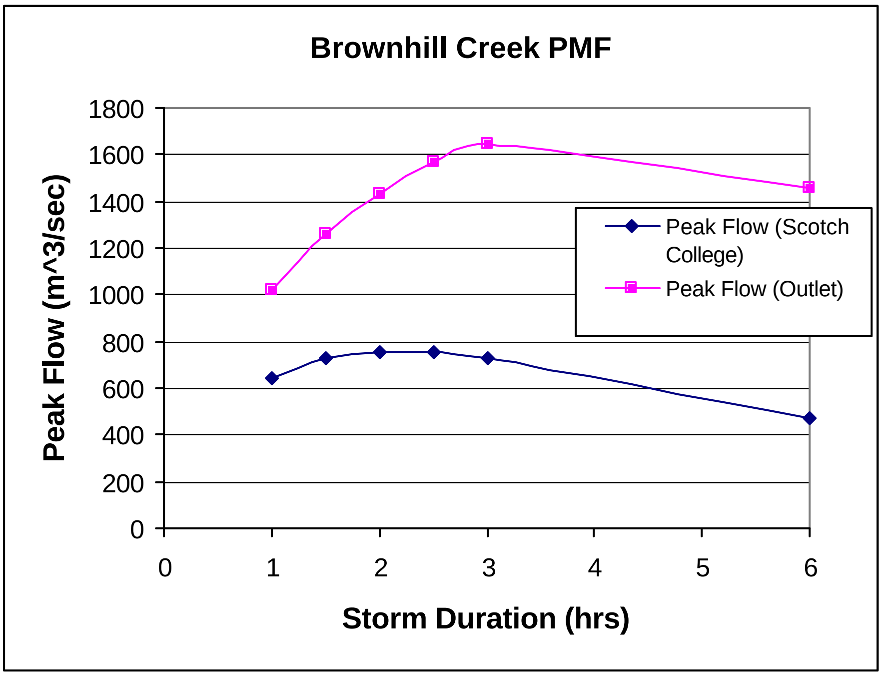


Figure 11-13 Brownhill Creek PMF

These predicted flows can be compared with Nathan et al (1994), who devised a quick method for estimating PMF in south east Australia. The method assumes a simple relationship between catchment area and PMF, and the estimated PMF is 756m^3/sec at Scotch College and 1724m^3/sec at the outlet.

The predicted flow from the RRR model is comparable at Scotch College (which has a substantially rural catchment), but is lower at the outlet. This is expected, given the large amount of storage on the floodplain.

11.5 The Olary Floods

Early February 1997 saw the occurrence of heavy rainfalls over a wide area of South Australia's north. One of the worst hit areas was near Olary, in eastern South Australia, where over a three day period, rainfall totals up to 320 mm were recorded. Within this period, localised, short duration intense rain occurred.

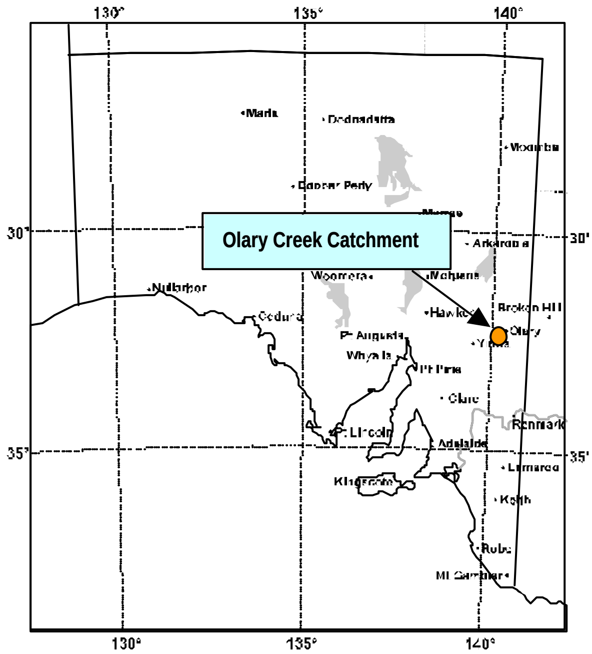


Figure 11-14 Location of the Olary Creek Catchment

The rain produced floods that washed away large sections of the main Sydney to Perth railway and inundated long sections of the Barrier Highway. Repair costs were of the order of $6 million for the railway and $1.5m for the road. Damage to rural infrastructure in the region was substantial. Flows within the catchment would have been sufficient to wash away most stream gauging stations.

Figure 11-15 Olary Creek at Wawirra, on the Broken Hill Road, February 1997

The airmass over much of South Australia was of tropical origin, contained a high amount of moisture and was unstable. Thunderstorms were the main rain producer, consequently the event was characterised by localised, very intense rain episodes. This contrasts with the March 1989 floods, where it rained at a fairly steady rate over large areas for durations up to 24 hours, as a monsoon low tracked across the state.

Four hour rainfall totals of 192 mm and 211 mm were recorded at two locations in the Olary Creek catchment on the morning of 7 February.

The largest recorded six hour rainfall was approximately 240 mm between 6:00am and 12noon on 7 February at Wiawera Station. This corresponds to an average intensity of 40 mm/h, which is more than twice the rainfall intensity estimated to have an Average Recurrence Interval (ARI) of 500 using the procedures from Australian Rainfall and Runoff. Australian Rainfall and Runoff procedures indicate that the observed intensity at both Wiawera and Eringa Park probably had an ARI in excess of 10,000 years. However, the accuracy of the recurrence interval of these extreme events is very doubtful.

Information regarding water levels over time at Wiawera Station was used to estimate an event hydrograph, shown in Figure 11-16. At Wiawera Homestead on Olary Creek the flow was estimated to be 5500 m³/sec. The hydrograph indicated a mean runoff depth of 125 mm, which can be considered reasonable, given the mean catchment rainfall.

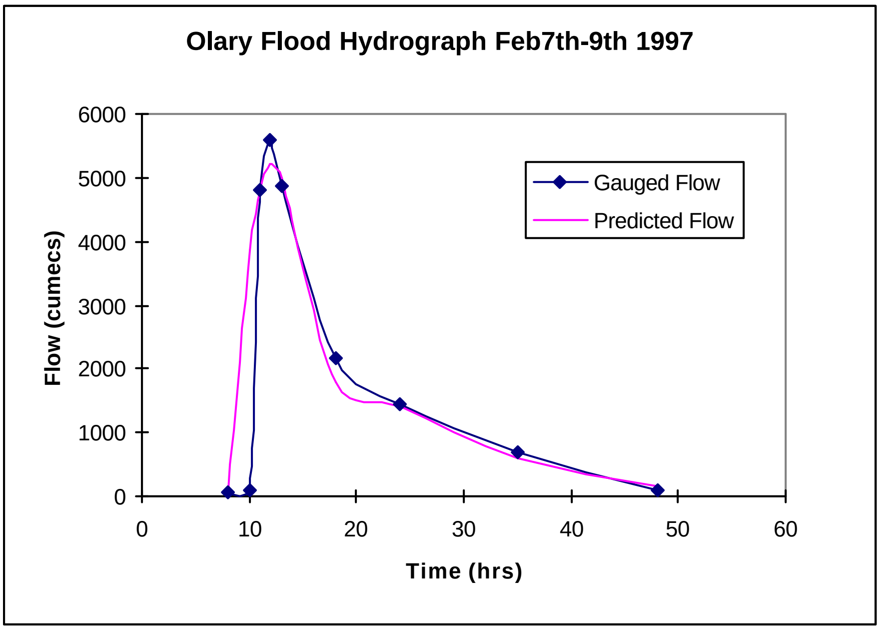


Figure 11-16 Olary Creek Hydrograph and RRR Prediction

The estimated flow of 5600 m^3/sec can be compared with the PMF (Probable Maximum Flood). A quick method of deriving PMF as outlined by Nathan et al (1994) gives a flow of 9700 m^3/sec.

Given the lack of pluviometer data several assumptions must be made regarding the rainfall applied to the model. The assumptions were as follows:

- 6 hour duration storm;
- 180 mm mean catchment rainfall; and
- temporal pattern as for a 6 hour storm, zone 6 of Australian Rainfall & Runoff

The calibrated model parameters are as follows:

For the channel:

k = 0.5, giving

v_c = 3.9 m/sec

For the runoff processes (two of which were identified):

Process	Initial Loss (mm)	Proportional Loss	k_p	c_p
1	30	0.43	6	1.4
2	80	0.43	0	na.

The estimated hydrograph and the RRR model fit are shown on Figure 11-16.

Comparison of these results with the calibration of the RRR model on other catchments in South Australia indicates that these calibrated parameters lie within the expected bounds. The characteristic channel velocity v_c is higher than expected, but this may be due to the mismatch in the actual and assumed temporal pattern. The process 1 c_p is similar to that associated with baseflow in other catchments. The proportional loss is lower than for events calibrated on other catchments, but this can be expected given the extreme rainfall.

Of interest is the second process modelled, where the process storage parameter k_p is zero. Zero process storage indicates overland flow is occurring, the catchment storage then being effectively that of the channel system.

Video taken from the verandah of the Olary Hotel on the morning of 7 February reinforces the fact that overland flow is occurring, as it can be seen that the whole of the ground surface is covered by flowing water. This assumption was also validated by inspection of some of the hillside slopes.

It is thought that overland flow does not occur frequently in catchments, but may occur more in arid areas. If this is the case then "normal" catchment behaviour should not be used to extrapolate to extreme events.

11.6 Summary

The RRR model was successfully applied to two mixed urban and rural catchments in the Adelaide suburban area, and a PMF derived for the total catchment. It has also been applied in the investigation of an extreme rainfall and flood event. The model has demonstrated its advantages over previous models in its ability to simulate separate flood processes in rural catchments, and to model flows in excess of the capacity of the underground pipe systems and open channels within the urban area.

For the Keswick and Brownhill Creek catchments the model predicted similar flows to the earlier WBCM (1984) study for lower flows, but predicted lower flows as the recurrence interval increased. This was most likely due to the ability of the RRR model to apply a different lag to flows in excess of the pipe and channel system capacity.

The PMF derived for the total Brownhill Creek catchment is consistent with PMF derived by other models for Australian catchments.

The application of the RRR model to the Olary Creek catchment indicated the presence of overland flow, and showed that catchment behaviour changed between minor and extreme events.

The model should therefore be developed to reflect the modeller's conceptual understanding of the processes involved, and should not be more complex than can be validated by the available data. The development of a model, or the adaptation of an existing one, should proceed from the simple to the more complex, where additional model complexity should only be retained if it can be shown that it yields a significant and consistent improvement.

Rodger Grayson (1993)

12. Summary and Conclusions

12.1 Summary

The investigation of the structure of runoff routing models and their calibration on Australian catchments has revealed the need for a runoff routing model that allows for the different runoff processes that occur on a catchment.

A model has been developed that performs better at predicting flows from catchments than existing models. More importantly it leads to a better understanding of catchment runoff behaviour. It can be used without catchment sub-division where spatial variability does not need to be included, or with catchment sub-division to include spatial variability. With the use of a catchment characteristic lag parameter for each runoff process the model will be internally consistent, unlike previous runoff routing models.

One of the basic objectives of the study was that any new model if developed needed to be an appropriate model.

12.2 RRR as an Appropriate Model

In the introduction to this thesis the point was made that any hydrological model is of value only if captures the essence of the runoff process, with the simplest structure, and with the least number of variables to be determined.

To answer the question as to whether the RRR model is an appropriate model the following questions must be answered:

- Does the model fulfil the functions for which it is intended?
- Is there a simpler structure that could be used for the model that would still fulfil the intended function?
- Does the model have the least number of parameters necessary?

The answers to these questions are included in this thesis and will now be summarised.

12.3 Functionality

The RRR model is intended to perform the functions of the current runoff routing models. These models were developed to enable the prediction of flood hydrographs in rivers and urban drainage systems. They are event models, and it is assumed that when used with appropriate initial conditions they can be used to transform a design rainfall of a given ARI to a design flow of the same ARI.

When categorised under the criteria of Grayson and Chiew (1994) the RRR model would be described as a simple conceptual model, as less than 8 parameters are calibrated. It can be used as a lumped model, without spatial variability, where single loss and storage parameter values are adopted to represent the entire catchment. For a multiple sub-catchment model, allowing for spatial variability, some parameters (mainly loss parameters) may be applied to different parts of the catchment. These catchments are often mixed urban and rural catchments, where there are significant differences in hydrological behaviour across the catchment.

Conceptual models differ from physically based distributed process models that attempt to predict flow throughout the catchment (Grayson and Chiew, 1994). Parameters in these models have direct physical meaning (eg. hydraulic conductivity, porosity, leaf area index). In theory, parameters for physically based models can be directly measured in the field. Storage parameters for the RRR model cannot be directly measured in this way.

The RRR model fulfills the intended function as a predictor of hydrographs from a catchment as it includes baseflow as an integral part of the model. Of the three existing runoff routing models (RORB, RAFTS and WBNM) only RAFTS with the inclusion of the ARBM loss model can be used to predict baseflow. Australian Rainfall and Runoff (1987) includes recommendations on baseflow separation but has no recommendation on the inclusion of a baseflow component in predicted hydrographs. Even in more recent investigations of flood estimation methodology, Siriwardina et al (1997) separated baseflow, and added a baseflow component from an observed hydrograph to the predicted surface runoff hydrograph. In that work there was no methodology included for predicting the baseflow component.

The RRR model has been shown to be able to predict total hydrographs in validation runs, with differences in peak flow and volume that can be attributed to problems of the definition of catchment rainfall both in the calibration and the verification phase. The RRR model performed better than existing runoff routing models.

RRR has also been shown to perform better than KINDOG, a more complex model incorporating simple linear and non-linear storages to model hillside response and kinematic wave equations to model channel response.

12.3.1 Is There a Simpler Structure?

To consider whether there is a simpler structure available one must consider the requirements for the model. According to Klemeš (1986) the model must reflect, even if only in a simplified form, the essential features of the physical prototype. It must work for the right reasons.

To consider a model with a simpler structure one must look at runoff routing models such as RORB, WBNM and RAFTS. These models make the assumption that surface flow only is being modelled and that all storage can be considered to be in the channel system within the catchment. Laurenson (1964) clearly states these assumptions. However the models will only be valid if direct surface runoff occurs.

Once the model has been expanded to include several processes the assumption that all storage (and thus storage lag) is present in the channels is no longer valid. The model must be split such that channel and process storage can be considered separately. No evidence that a runoff routing model has done this in the past was discovered.

The RRR model makes the assumption that each process may be modelled by a series of storages with rainfall inputs to each of the storages. Each rainfall input and storage has four parameters, two storage parameters and two loss parameters. In this investigation only one storage parameter has been used, as the measure of non-linearity of the process storages has been kept constant. The results of the calibration runs give no indication that this is an unreasonable assumption.

The only means by which a simpler model can be produced is by reducing the number of storages (process and channel). A preliminary investigation into the effect of this was undertaken on one catchment. It showed that as the number of storages reduced the model generally could not be fitted as well to known events. However the predictive ability of the model was less affected.

There is no simpler structure that will fulfil the function of the RRR model, however it may be that less storages for both the channel and process elements are needed.

12.3.2 The Number of Parameters

The number of parameters in the model must be minimised. A parameter is a value that may vary from time to time depending on catchment condition. Parameters are usually varied during calibration of the model. If no variation is found in the value of the parameter then it can be considered a constant.

For the RRR model it is worth considering urban and rural catchments separately.

12.3.2.1 Urban Catchments

The RRR model for urban catchments is simple. The storage parameters will remain constant for all events and related to catchment pipe and gutter lengths and grades. The value of proportional loss for the runoff from the directly connected impervious area is zero. Only two processes are considered, being runoff from the directly connected impervious area and unconnected area.

For these processes there are then only three parameters, which consist of two initial losses and a proportional loss from the unconnected area. The verification of the model on Adelaide catchments has shown that runoff from the unconnected areas can be ignored for most events, so in fact there is only one parameter in this situation. For large events this will not be the case.

12.3.2.2 Rural Catchments

When the RRR model was applied to rural catchments it was shown that up to three processes occur. In the events that were used for calibration there was only one event that had a contribution from direct surface runoff (fast flow).

If two processes are modelled there are a total of 7 parameters, being an initial and proportional loss for each process, two parameters related to process storage (characteristic lag) and one channel storage parameter (characteristic velocity). In all cases initial loss on the first process was zero, as baseflow was occurring, and so 6 parameters were calibrated. If a third process is modelled there are two extra parameters, being the two loss parameters for the third process. The number of parameters cannot be reduced without changing the structure of the model.

When comparing the number of parameters with RORB, WBNM or RAFTS it must be borne in mind that these models deal with only one process. A RRR model with one process modelled would only have one extra parameter, because channel and process storage are considered separately.

12.4 The Factors that Affect Catchment Response

The RRR model has been applied to a total of eleven catchments in the Mount Lofty Ranges to determine model parameters for application to ungauged catchments and to provide some understanding of the factors that determine how catchments respond to rainfall.

In previous work on generalised parameter values for runoff routing models applied to rural catchments no consistent factor has been found that affects catchment response time, apart from the average flow length and average annual rainfall. The average annual rainfall is acting as an indicator that catchment response is changing due to the change in runoff processes that follow climate.

Runoff routing models only model direct surface runoff. This being the case the total storage in the catchment can be represented by the storage available within the channel system of the catchment. Any effect there might be due to the processes occurring on the hillsides of the catchment is bound up with channel storage, and cannot be separately examined.

The RRR model for the first time has enabled the examination of the response time due to the processes occurring on the hillsides, and thus represents a significant advance in runoff routing models.

From this investigation it can be stated that soil depth and the root zone water holding capacity are the two main factors that affect catchment process lag and the initial loss for the slow flow component. The presence of native vegetation increases both catchment response times, and the initial loss. However in the data set used the percentage of native vegetation in the catchment is related to both the percentage of soils with low root zone water holding capacity and the percentage of the catchment with soils described as deep. The percentage of native vegetation may not therefore be an independent variable.

The root zone water holding capacity of a soil is related to soil type, with sandier soils having a lower water holding capacity than clay soils. Therefore as soils within a catchment become sandier both the process lag and the initial loss for the slow flow component increase. The process lag increases as more water is being infiltrated than is directed to the channel by surface or near surface flow paths. It thus has a longer flow path, with a higher response time.

12.5 Limitations of RRR and Further Work Required

12.5.1 Event Versus Continuous Modelling

The main limitation of RRR is that it is an event model. Changing RRR into a continuous model would involve finding a relationship between the loss parameters and a measure of physical catchment condition. The wide variation in calibrated values for loss parameters and the lack of any clear physical relationship between losses and catchment antecedent condition indicate that the prediction of loss parameters to be used in the model may be complex.

12.5.2 Correlation with Catchment Characteristics

The RRR model has been calibrated on catchments in the Mount Lofty Ranges, and correlation carried out to determine the factors that govern catchment response. The model should be fitted to other groups of catchments around Australia to confirm that the factors are consistent across a wide range of climates and soils.

This will give a greater confidence in the application of runoff routing models to ungauged catchments than currently exists.

12.5.3 Catchment Scale

The RRR model needs to be examined for the effects of catchment scale. It might be expected that catchment size does have an effect on catchment response. The RRR model with a single set of global storage parameters and characteristic channel velocities for catchments of all sizes does not take into account a scale effect. In the catchments used for calibration and verification, ranging from 4.3km^2 to 70km^2 no such effects were observed, but this may be due to the catchments selected.

One of the most obvious effects of scale is the development of flood plains. The effect of the change from in bank to overbank flows in rivers has been well documented. Once the river has well defined flood plains the assumption of linearity of channel response may not be realistic. In fact behaviour may become very non-linear, with a value of m greater than 1. It will then be necessary to provide in model channel storages a storage relationship that accounts for this non-linearity.

Secondly for small rural catchments there may not be a clearly defined and separable process and channel storage. The channel storage should tend to zero as catchment size decreases. In urban catchments it can be expected that the model can be applied to the catchment contributing to a single inlet, as there is still a length of gutter flow involved.

12.6 Original Findings and their Implications

The examination of the structure of runoff routing models and the development of the RRR model in this thesis has made a significant contribution to the prediction of flood flows in catchments.

Some of the significant original findings and their implications are:

- The flow predicted by a RAFTS model can be shown to depend on the number of sub-catchments or nodes in the model. This has two implications, the first being that a global storage parameter cannot be used with a model having a different number of nodes. The second is that the RAFTS model is not self-consistent, and there can be no confidence that flows at intermediate points within the model are being correctly predicted.
- The response time of a catchment is dependent on the processes that are occurring within the catchment
- The assumption of runoff routing models such as RORB that all storage within the model can be accounted for in the mainstream system is not generally valid.
- There are generally three distinct runoff processes occurring on rural catchments, termed in this thesis baseflow, slow and fast runoff. The slow runoff is most likely to be capillary fringe flow, and the fast runoff a direct surface runoff by saturation overland or Hortonian overland flow.

- Direct surface (fast) runoff occurs rarely on most rural catchments, with most catchments having a baseflow component and a slow flow component. In extreme events catchment behaviour may change as fast flow, with a quicker response time begins.
- A RORB, RAFTS or WBNM model can be calibrated on events having only baseflow and slow flow. When extrapolating the models by applying extreme rainfall, the fast flow that may occur with extreme rainfall cannot be accounted for. The RRR model can be calibrated on smaller events, and used in the prediction of extreme events as the storage parameter for fast flow can be determined from the events generally used for calibration.
- For baseflow and slow flow the factors that govern response time are the soil depth and the root zone water holding capacity. The initial loss before slow flow commences is also determined by these two factors.

It may be that the reason there is so much variation in regional RORB k_c relationships is that different processes are occurring both within and across the different catchments used in the derivation of regional relationships. The RRR model therefore has greater potential to model the response of ungauged catchments, where regional relationships for storage parameters need to be used.

12.7 Conclusions

The conclusions that can be drawn from this study are as follows:

- There are clear links between the main parameters of the runoff routing models (RAFTS, RORB, ILSAX and WBNM). Consideration of and comparison of the structures of the models derived these links.
- For the RAFTS model the number of nodes upstream of the point of interest influences the result of modelling. This means that the model is not self-consistent, and that regional parameters derived cannot be used in a model with a different number of nodes.
- The WBNM model structure is such that to retain the same storage lag for a modelled catchment the ratio of the lag of ordered basins which receive no inflow across any boundary and interbasin areas which contain a stream draining upstream areas would have to be varied depending on the number of sub-catchments upstream of the point of interest. As the WBNM

model retains a constant value the results of modeling are governed by the number of sub-catchments in the model.

- Generalised storage parameters for a single node RAFTS model can be derived from RORB parameters for rural areas and from the ILSAX model for urban areas.
- The lag of a catchment is related to the runoff process that is occurring in the catchment. The lag displayed by the runoff process is determined mainly by soil depth and root zone water holding capacity.
- A runoff routing model structure incorporating more than one process can be formulated that is in most cases simple to apply. For spatially uniform catchments the model can have a simpler structure and parameter input than the RORB, RAFTS and WBNM models.
- The new runoff routing model structure can perform better than existing runoff routing models on rural catchments, and at least as well in urban catchments. The RRR model will be more appropriate than existing models for application to extreme events. It has been successfully applied to an extreme event on the Olary Creek catchment.
- It may be possible to reduce the number of storage elements in the RRR model, and still have a model that can perform well at the prediction of storm runoff hydrographs. The RRR model structure performed best when fitting to known hydrographs.
- As with any new model RRR must be applied both to a range of catchments and a range of event magnitudes if the effects of catchment and event scale are to be determined. Experience must be gained in the application of the model.

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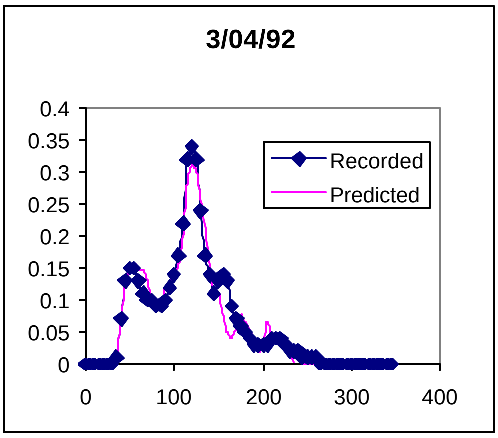
Appendix 1 – Electronic Files Associated with the Thesis

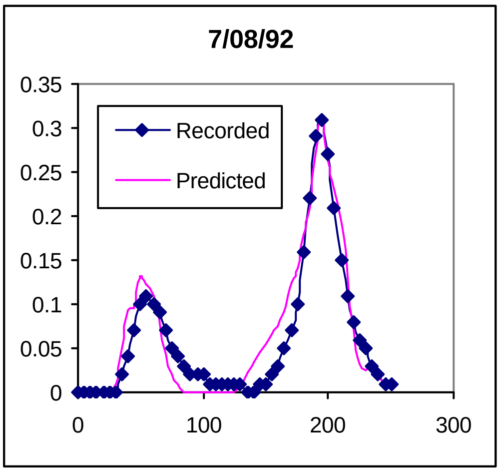
Structure – Thesis Files

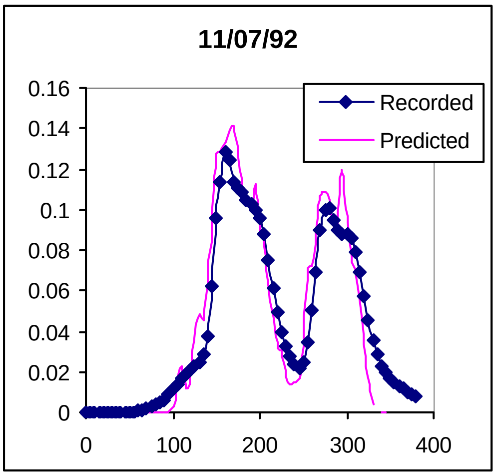
Data	Glenelg. Files in RAFTS historical file format.
	Paddocks. Files in RAFTS historical file format
	Sauerbier. Files in RAFTS historical file format
ILSAX	Glenelg. ILSAX rainfall and pipe files for the Glenelg catchment (Section 6.2)
	Paddocks. ILSAX rainfall and pipe files for the Paddocks catchment. Sub-directory has calibration of the ILSAX model by PEST. (Section 6.3)
KinDog	Files associated with the calibration and verification of KINDOG on the Inverbrackie Creek catchment (Section 9.3.10)
KSSM	Spreadsheet files for the KSSM model, applied to the Inverbrackie Creek catchment. (Section9.3.12)
RAFTS	Aldgate. Investigation of the number of nodes in a RAFTS model (Section 5.4)
	Aroona. RAFTS models of the Aroona Creek catchment with both translation between nodes and channel routing. (Section 5.4)
	Glenelg. RAFTS modelling of the Glenelg catchment. (Section 7.3)
	Happy Valley. RAFTS modelling of the Sauerbier Creek catchment. (Section 7.5)
	Paddocks. RAFTS modelling of the Paddocks catchment. (Section 7.4)
	Windy. RAFTS models of the Windy Creek catchment. (Section 5.4)
RRR	Aldgate. Aldgate Creek RRR Modelling. (Section 10.2.3)
	Brownhill. Brownhill Creek RRR modelling. (Section 11.3)
	Brownhill PMF. Brownhill Creek PMF (Total catchment, including Keswick Creek) (Section 11.3)
	Burra. Burra Creek calibration and verification (Section 9.3.9)
	Celias. Celias Creek calibration and verification (Section 9.3.8)
	Chapter 9. Preliminary fitting of the RRR model to Kanyaka and Aldgate Creek (Section 8.8)
	Cox. Cox Creek RRR Modelling. (Section 10.2.1)
	Echunga. Echunga Creek calibration and verification (Section 9.3.6)
	First. First Creek RRR Modelling. (Section 10.2.7)
	Glenelg. Glenelg catchment RRR modelling (Section 9.2.1)
	Inverbrackie. Inverbrackie Creek calibration and verification (Section 9.3.5)
	Jamison Park. Jamison Park RRR modelling (Section 9.2.3)
	Keswick. Keswick Creek RRR modelling. (Section 11.2)
	Lenswood. Lenswood Creek RRR Modelling. (Section 10.2.2)
	Olary. Olary Creek modelling (Section 11.5)
	Paddocks. Paddocks catchment RRR modelling (Section 9.2.2)
	Scott. Scott Creek calibration and verification (Section 9.3.7)
	Sixth. Sixth Creek RRR Modelling. (Section 10.2.7)
	Torrens. Torrens River calibration and verification (Section 9.3.4)
	Westb. West Branch RRR Modelling. (Section 10.2.4)
	Woodside. Woodside RRR Modelling. (Section 10.2.5)
WBNM	Windy and Aroona Creek WBNM files (Section 5.3.3)

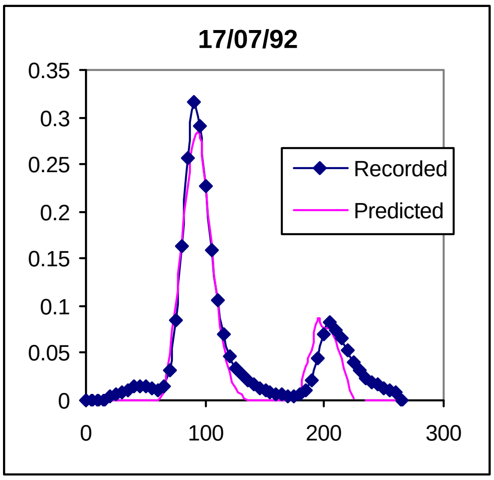
Appendix 2 – Glenelg Catchment ILSAX Calibration Results

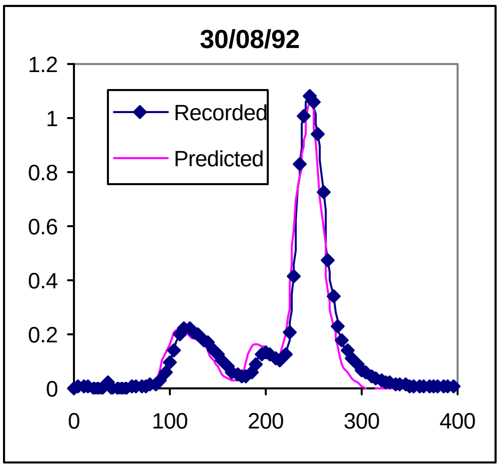
Frederick Street

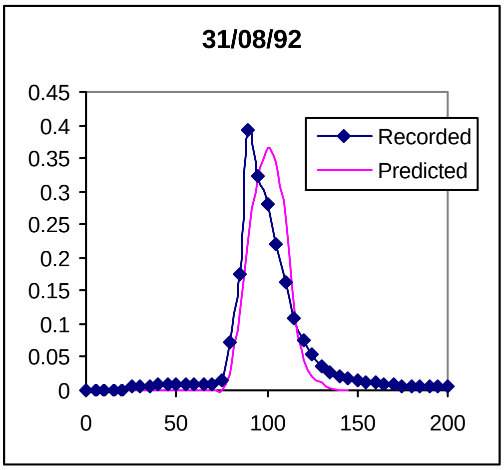


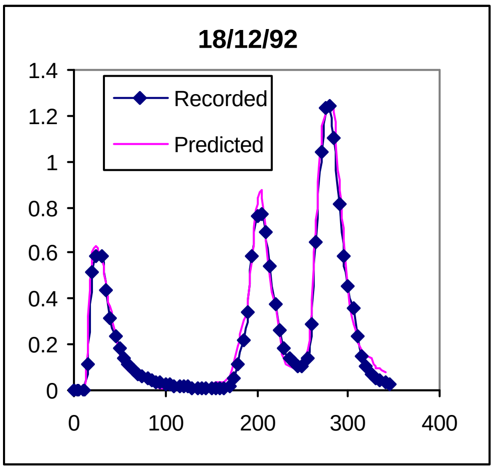


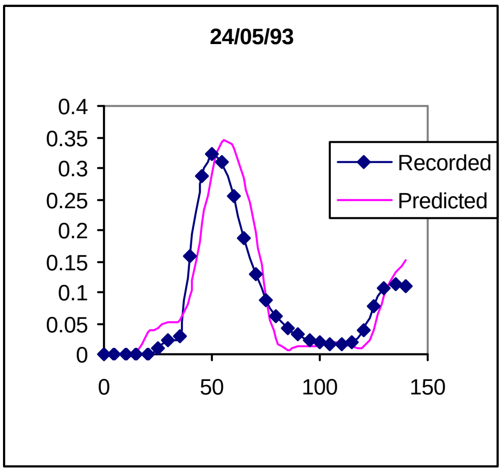


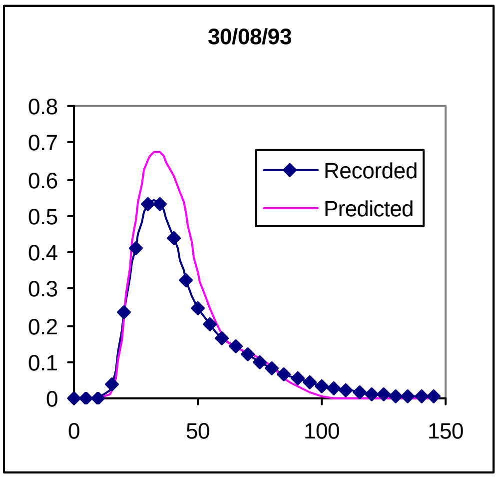


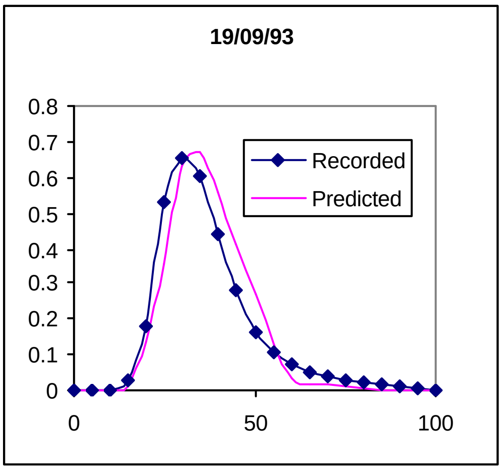


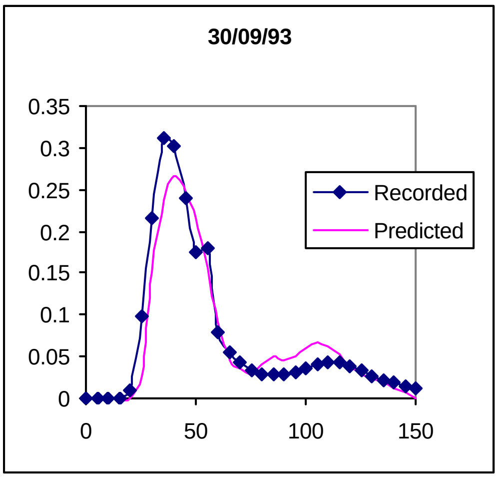


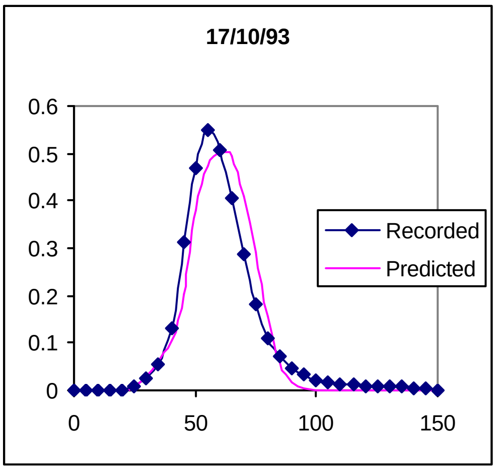

18/12/92
Recorded
Predicted
1.4
1.2
1
0.8
0.6
0.4
0.2
0
0
100
200
300
400


24/05/93
Recorded
Predicted
0.4
0.35
0.3
0.25
0.2
0.15
0.1
0.05
0
0
50
100
150


30/08/93
Recorded
Predicted
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
50
100
150

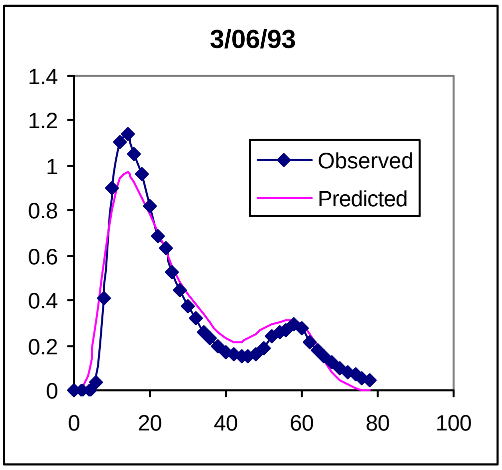

19/09/93
Recorded
Predicted
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
50
100

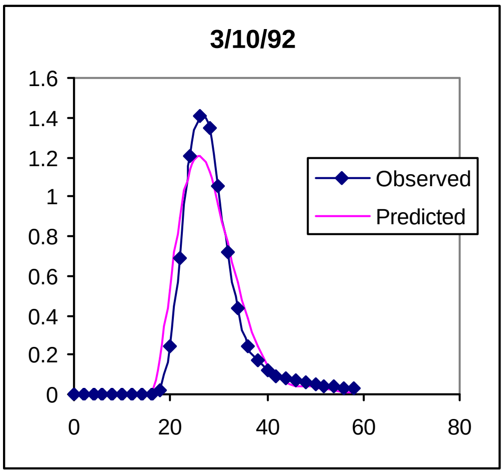

30/09/93
Recorded
Predicted
0.35
0.3
0.25
0.2
0.15
0.1
0.05
0
0
50
100
150

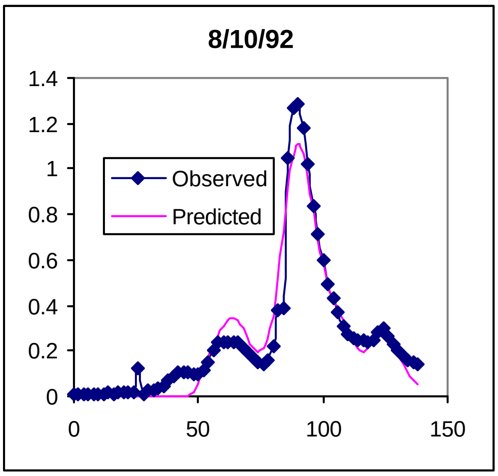

17/10/93
Recorded
Predicted
0.6
0.5
0.4
0.3
0.2
0.1
0
0
50
100
150

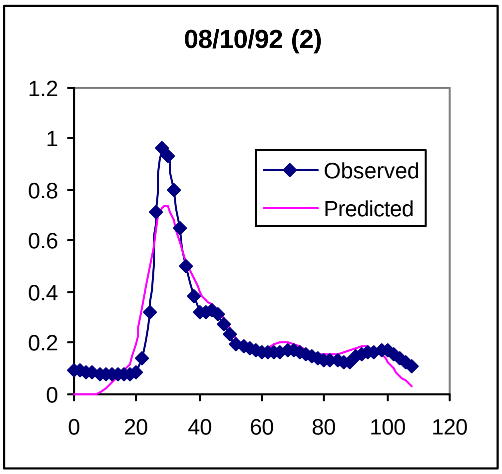
Appendix 3 – Paddocks Catchment ILSAX Calibration Results

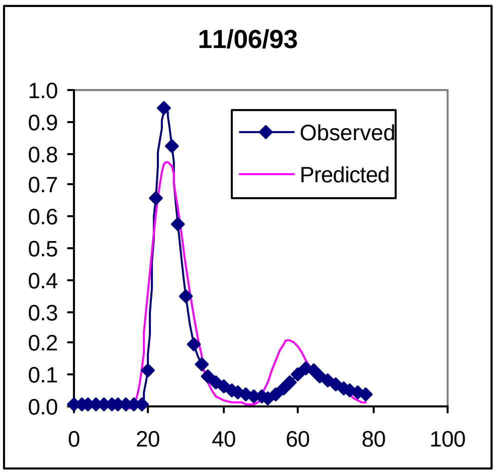
Paddocks Catchment - ILSAX Fits using PEST

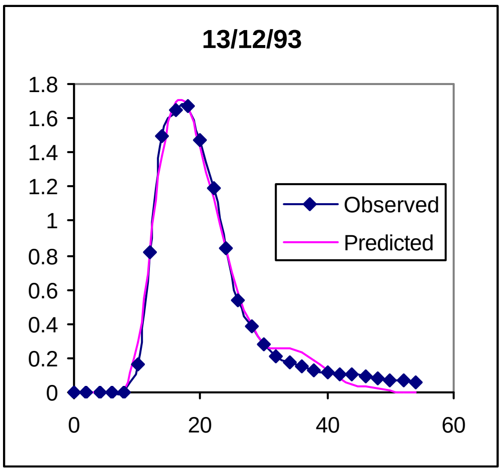


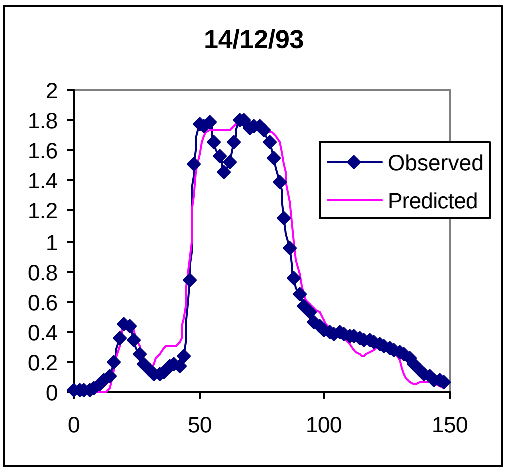


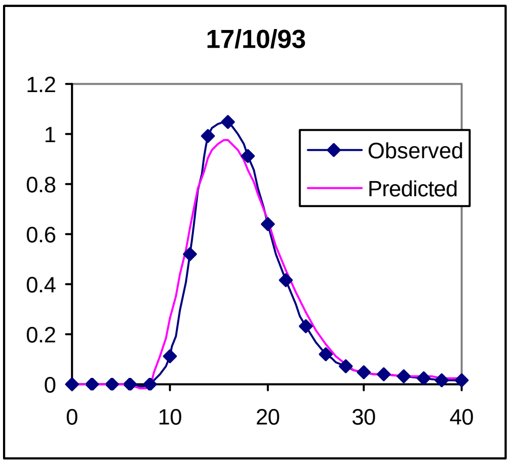


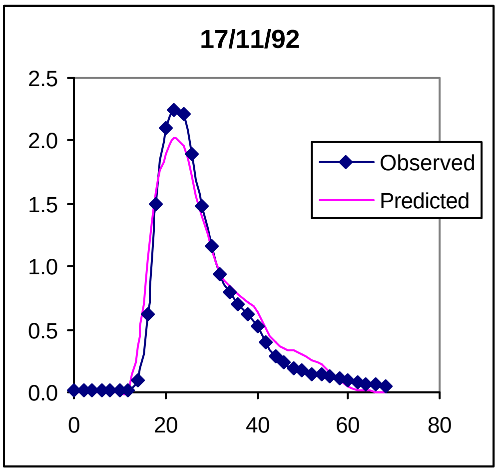


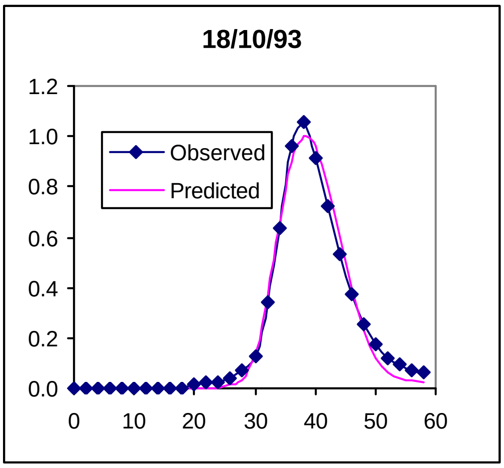


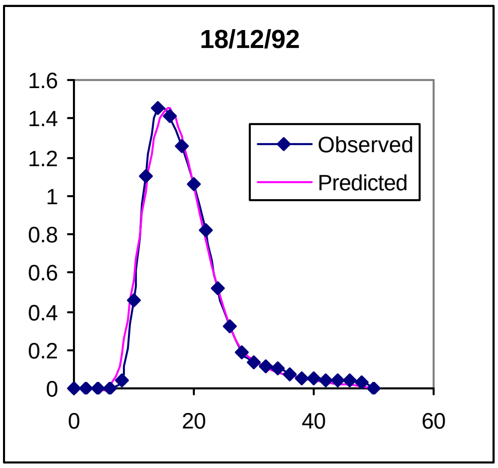


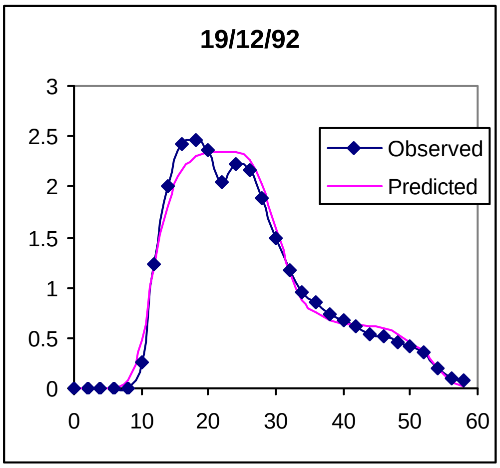

14/12/93
Observed
Predicted
2
1.8
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
0
50
100
150

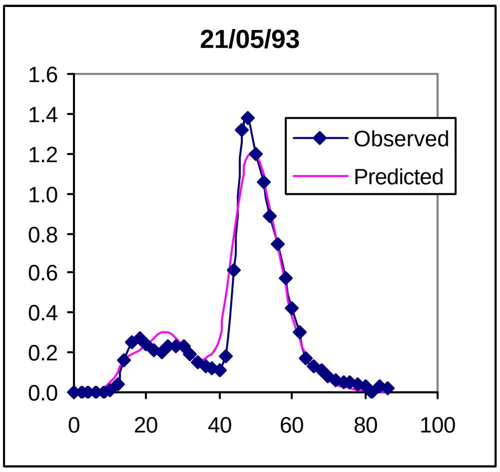

17/10/93
Observed
Predicted
1.2
1
0.8
0.6
0.4
0.2
0
0
10
20
30
40

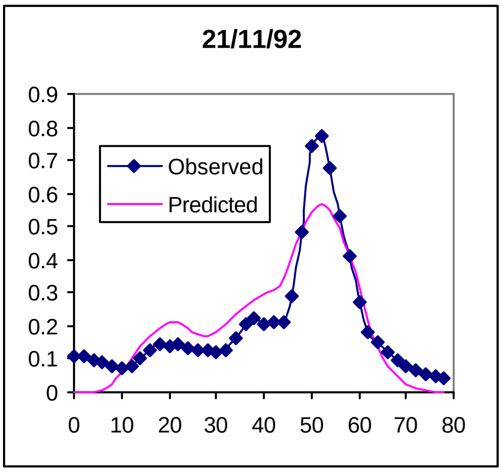

17/11/92
Observed
Predicted
2.5
2.0
1.5
1.0
0.5
0.0
0
20
40
60
80

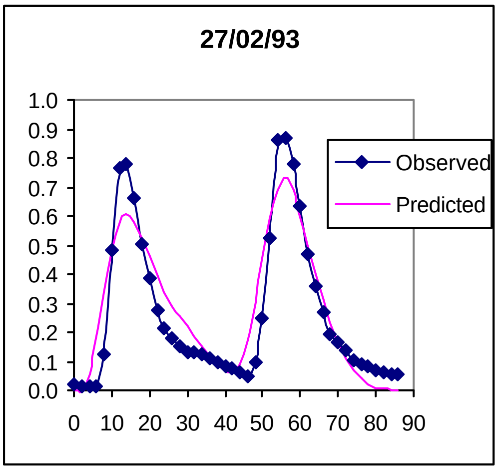

18/10/93
Observed
Predicted
1.2
1.0
0.8
0.6
0.4
0.2
0.0
0
10
20
30
40
50
60

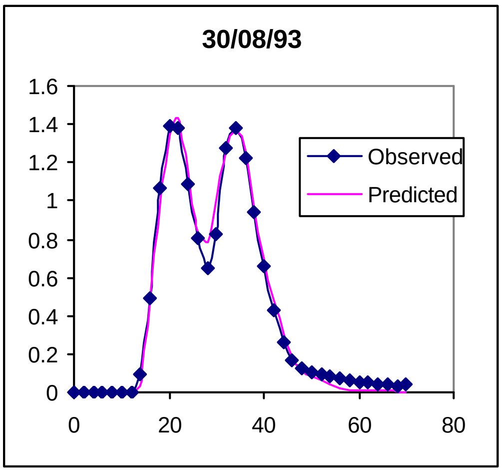

18/12/92
Observed
Predicted
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
0
20
40
60

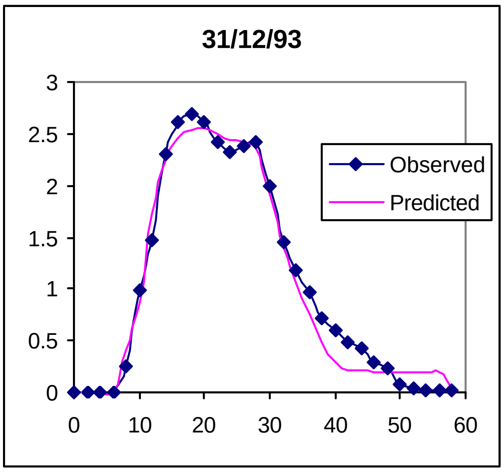

19/12/92
Observed
Predicted
3
2.5
2
1.5
1
0.5
0
0
10
20
30
40
50
60


21/05/93
Observed
Predicted
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
0
20
40
60
80
100


21/11/92
Observed
Predicted
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
10
20
30
40
50
60
70
80

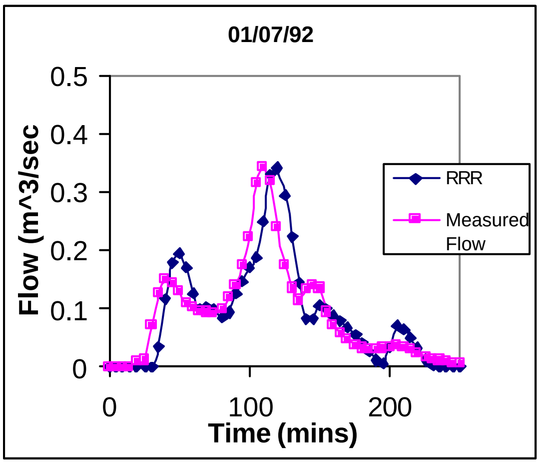

27/02/93
Observed
Predicted
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.0
0
10
20
30
40
50
60
70
80
90


30/08/93
Observed
Predicted
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
0
20
40
60
80

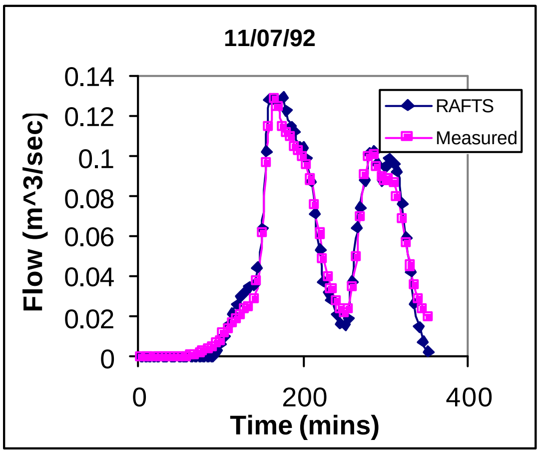

31/12/93
Observed
Predicted
3
2.5
2
1.5
1
0.5
0
0
10
20
30
40
50
60

Appendix 4 – Glenelg Catchment RAFTS Calibration Results

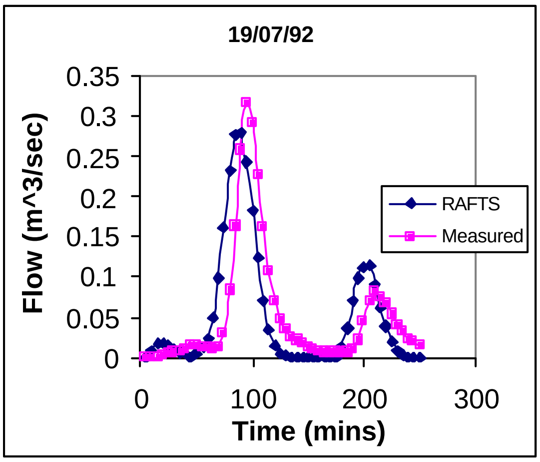
Frederick Street



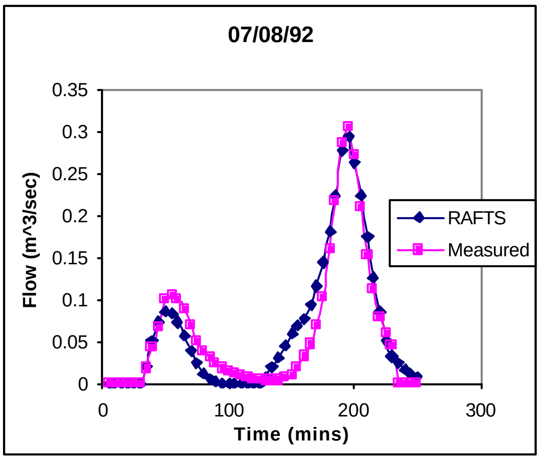
01/07/92



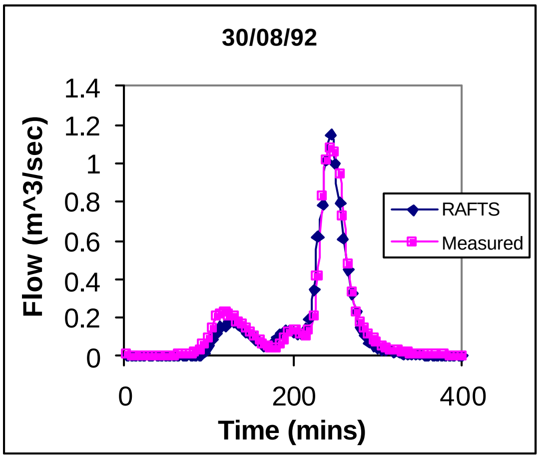
11/07/92



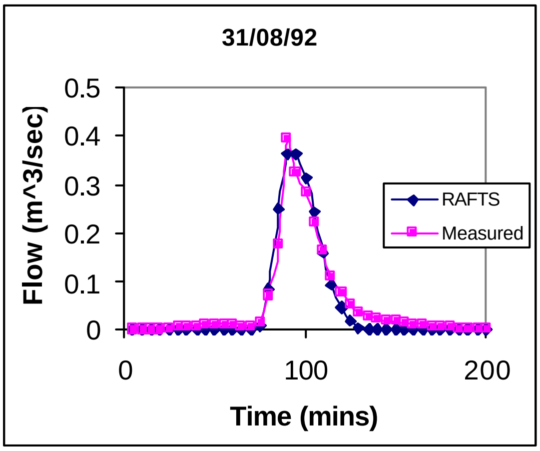
19/07/92



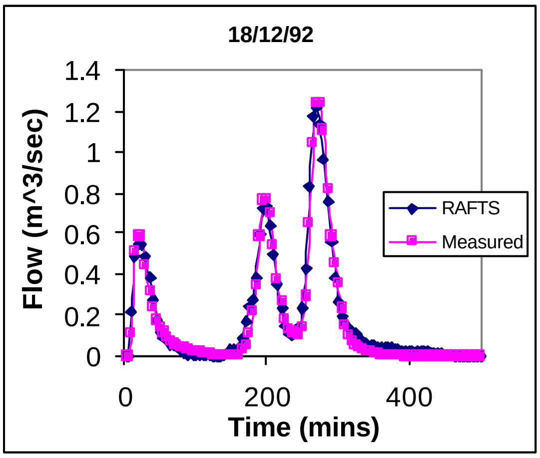
07/08/92



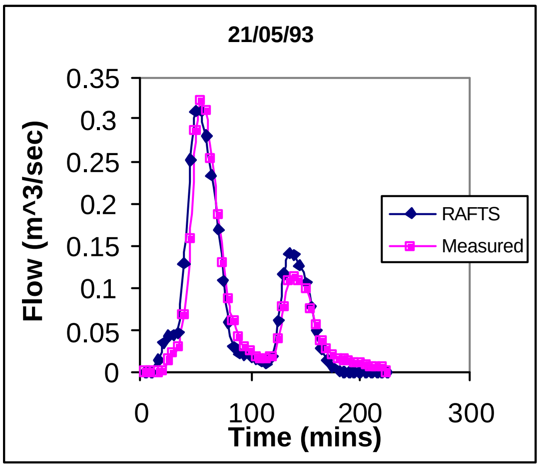
30/08/92



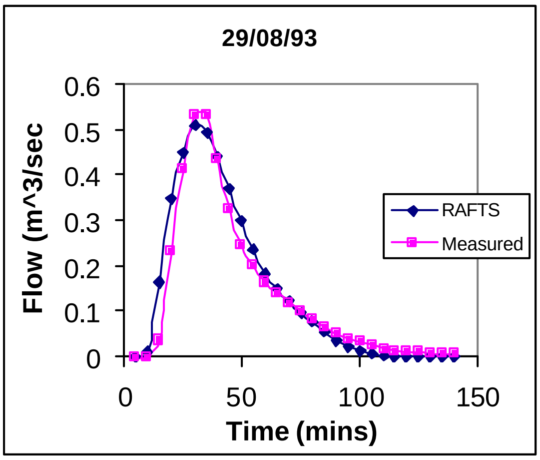
31/08/92



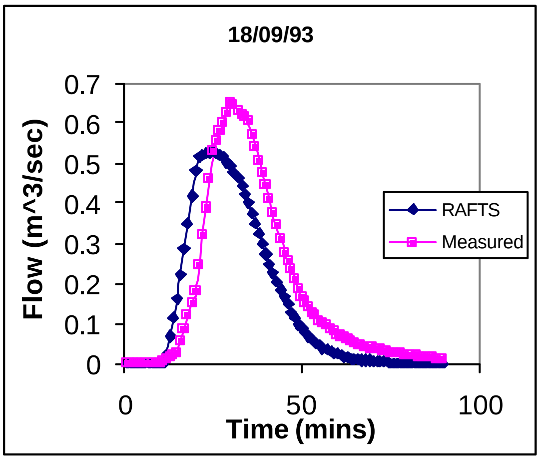
18/12/92



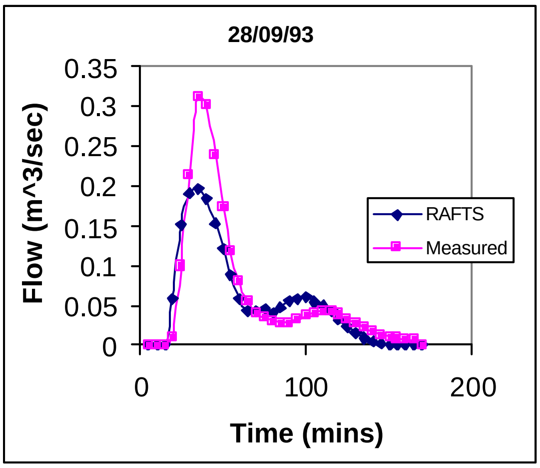
21/05/93



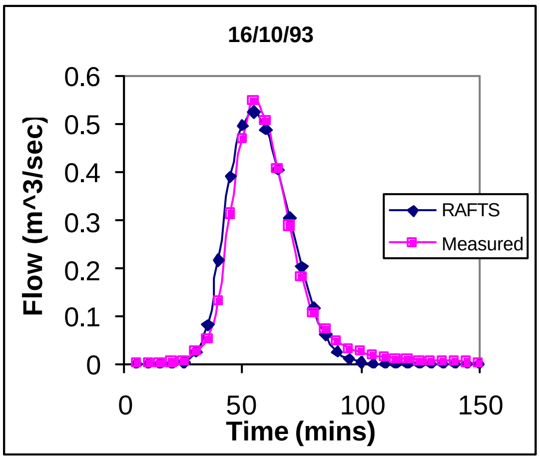
29/08/93



18/09/93

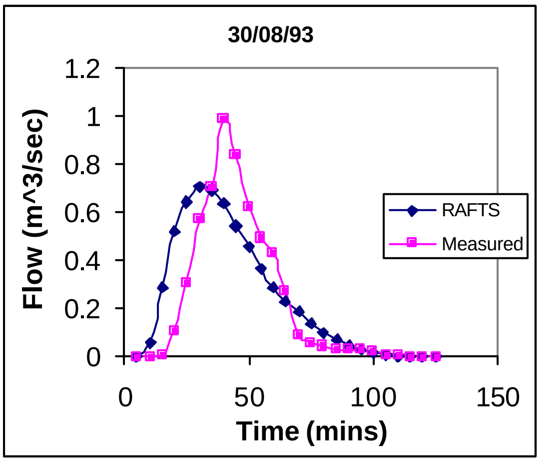


28/09/93

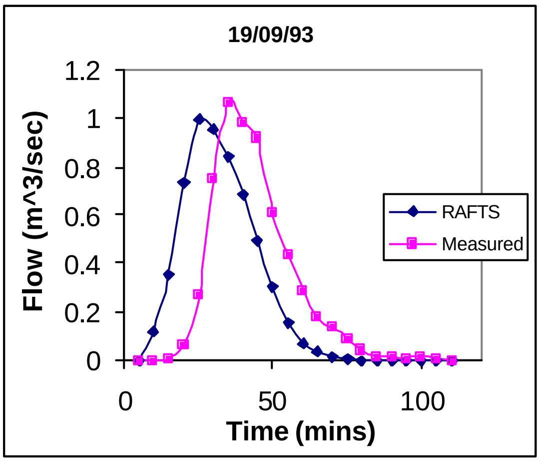


16/10/93

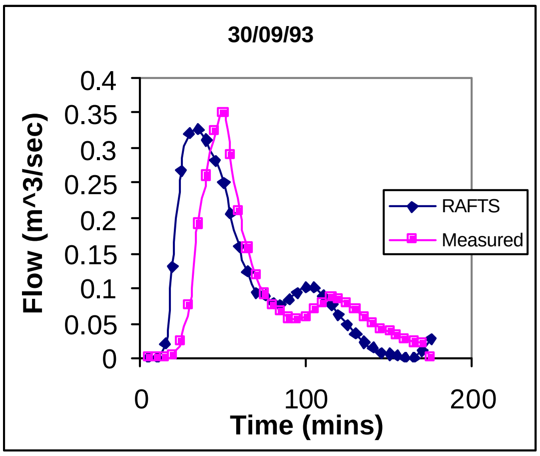
Maxwell Terrace



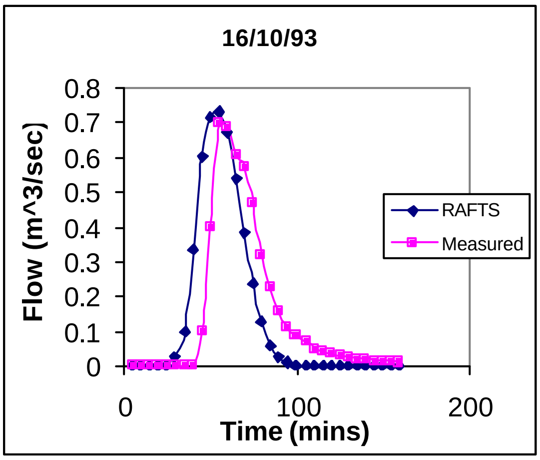
30/08/93



19/09/93

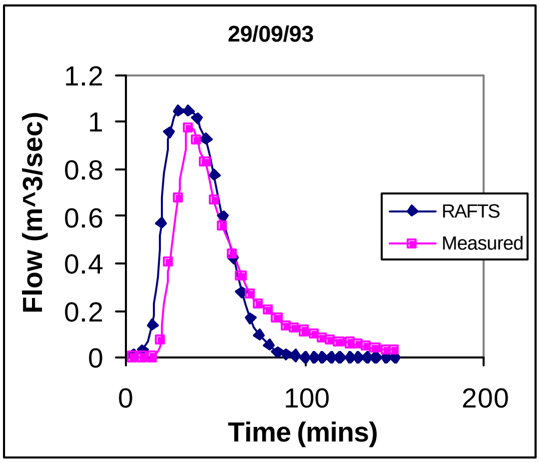


30/09/93

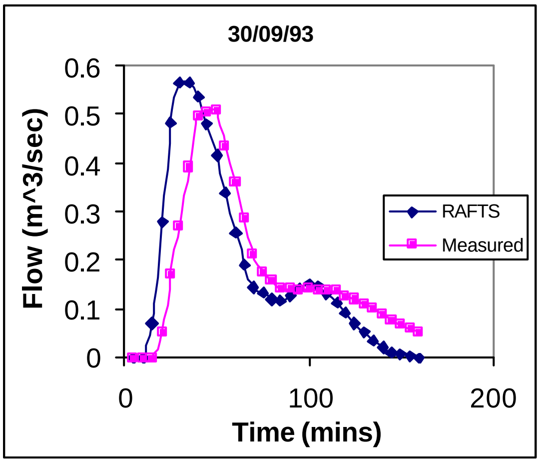


16/10/93

Torrens Square

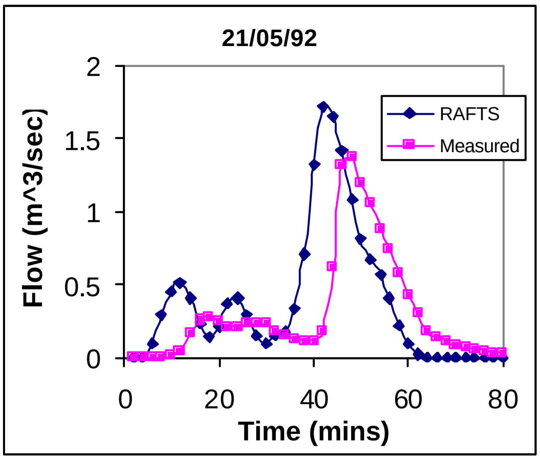


29/09/93

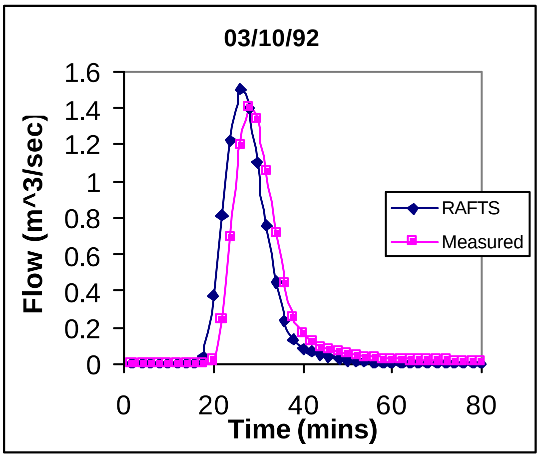


30/09/93

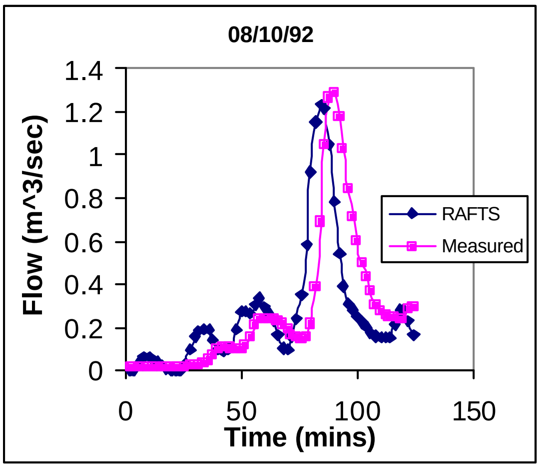
Appendix 5 – Paddocks Catchment RAFTS Calibration Results



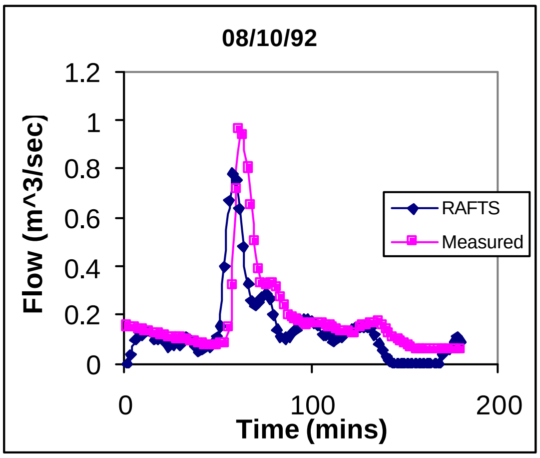
21/05/92



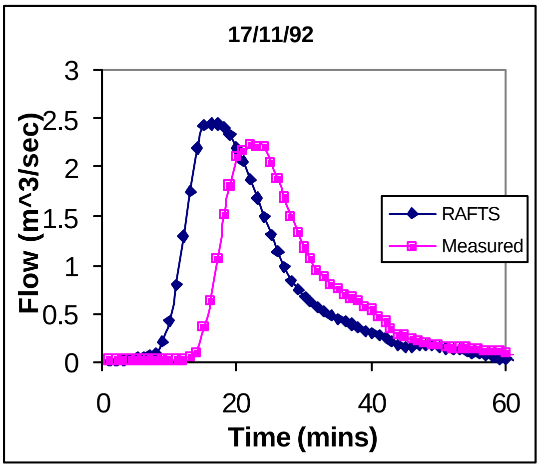
03/10/92



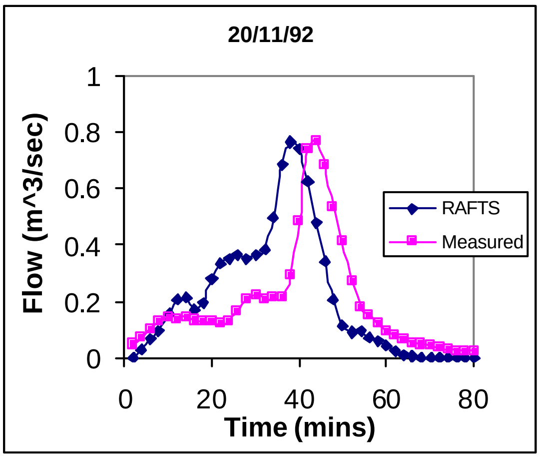
08/10/92



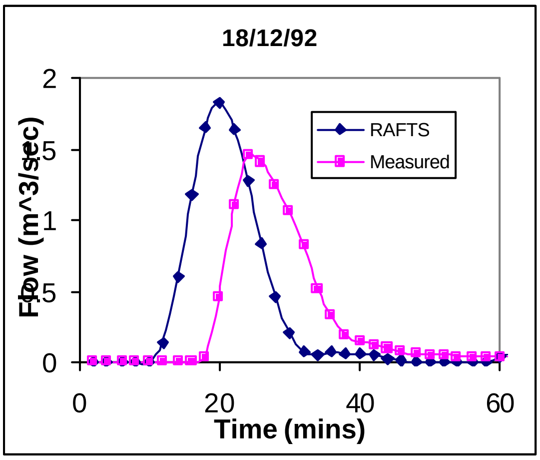
08/10/92 (2)



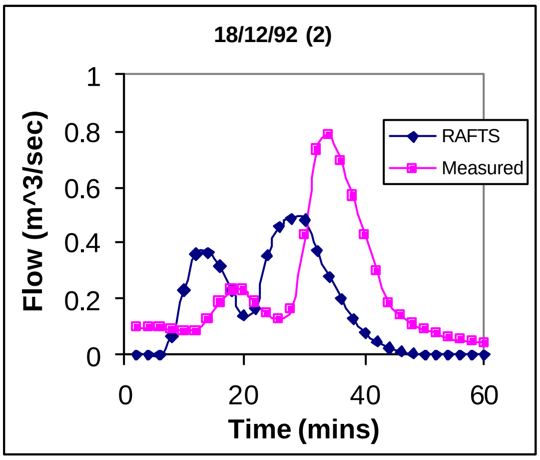
17/11/92



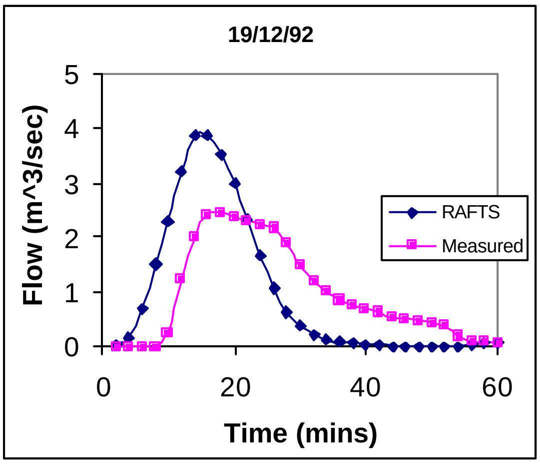
20/11/92



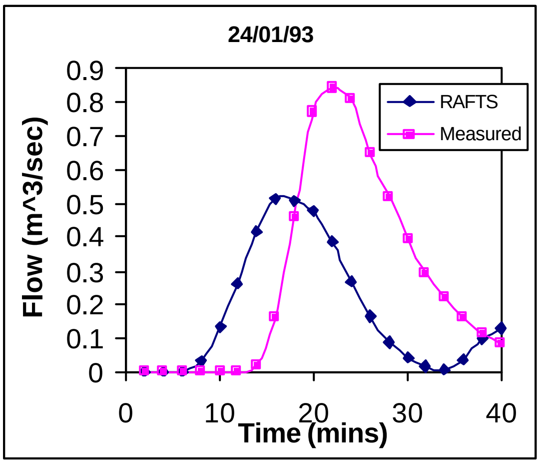
18/12/92



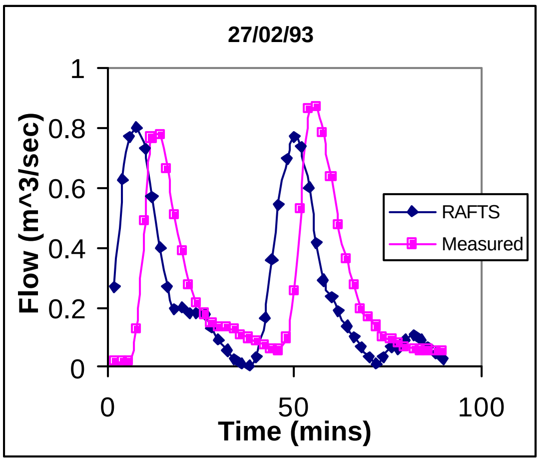
18/12/92 (2)



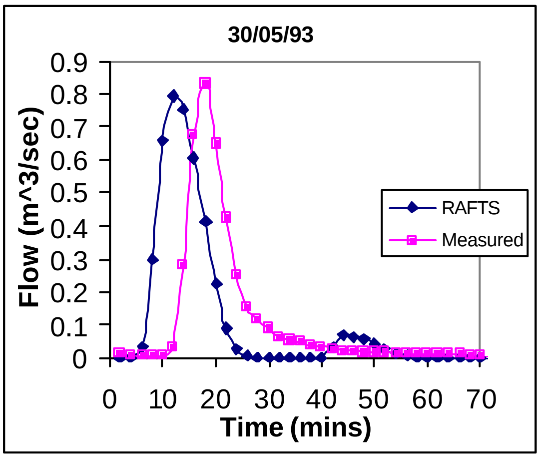
19/12/92



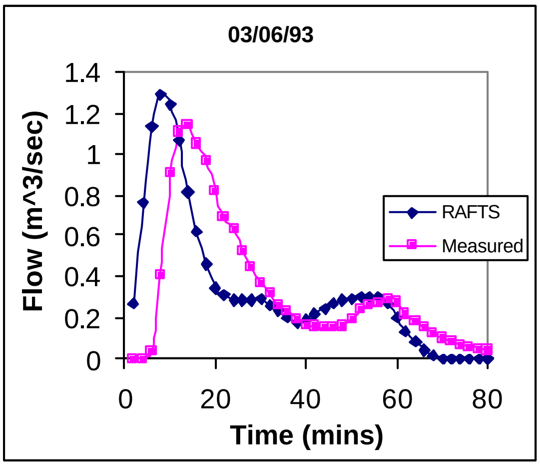
24/01/93



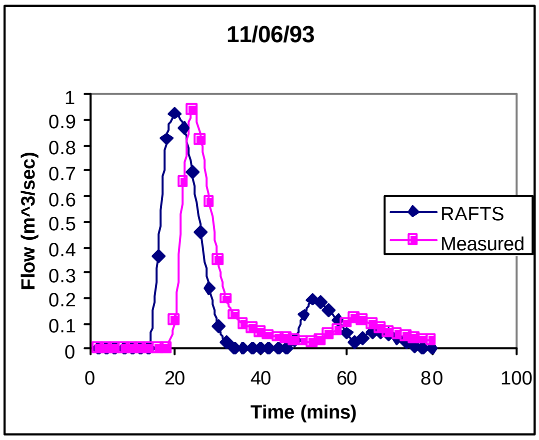
27/02/93



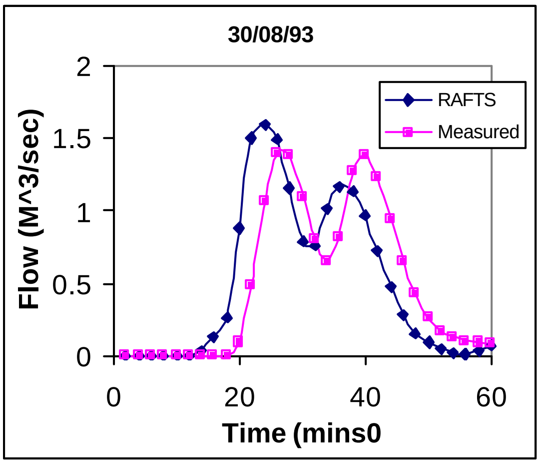
30/05/93



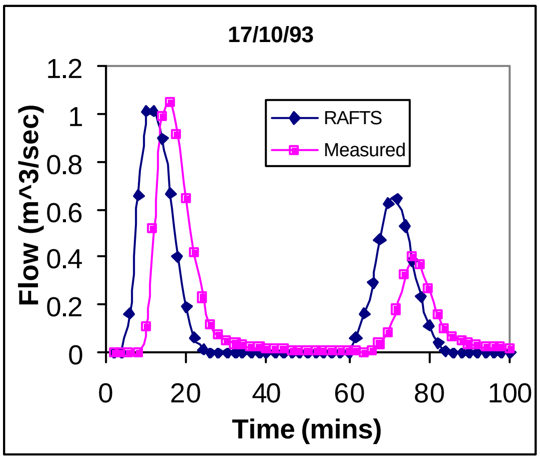
03/06/93



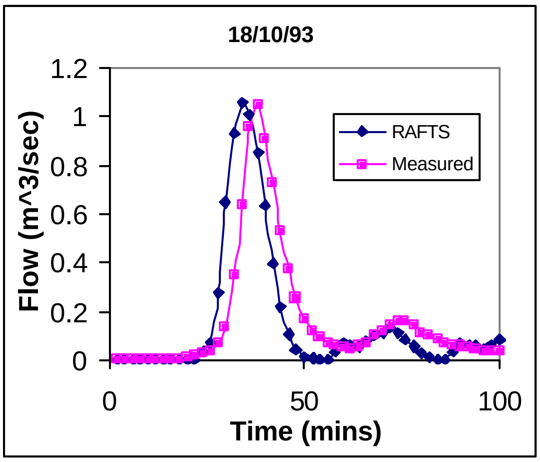
11/06/93



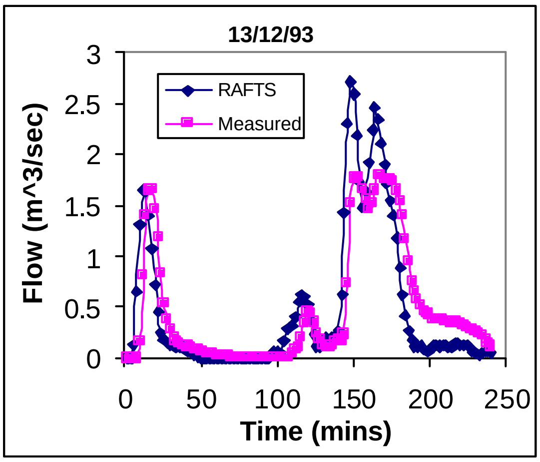
30/08/93



17/10/93



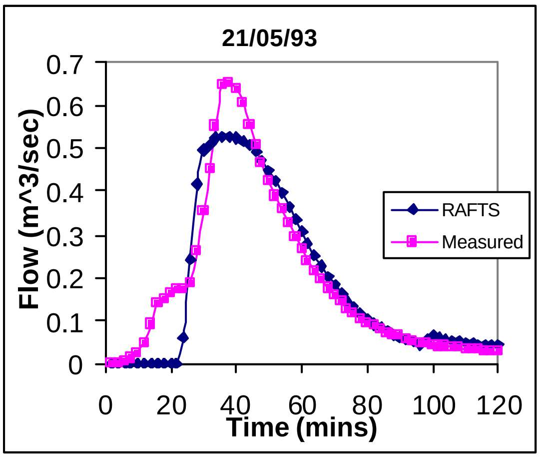
18/10/93



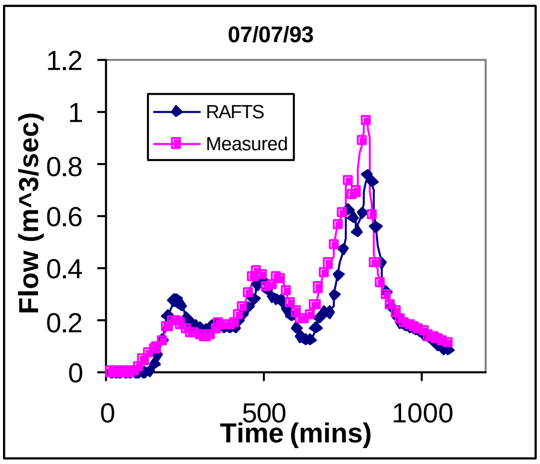
13/12/93

APPENDIX 6 – Happy Valley RAFTS Calibration Results

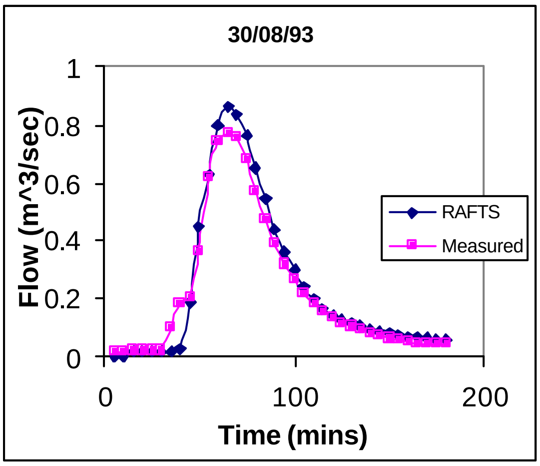
Saubier Creek Catchment



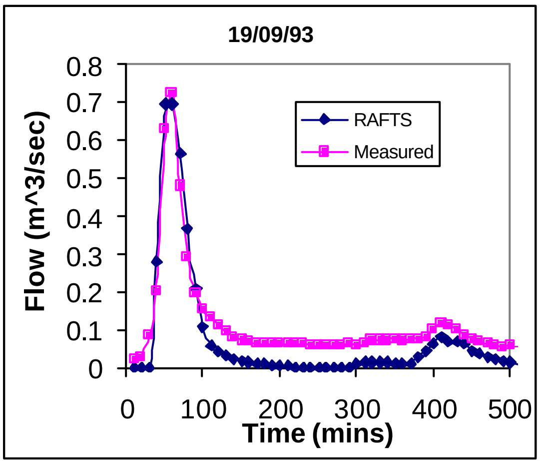
21/05/93



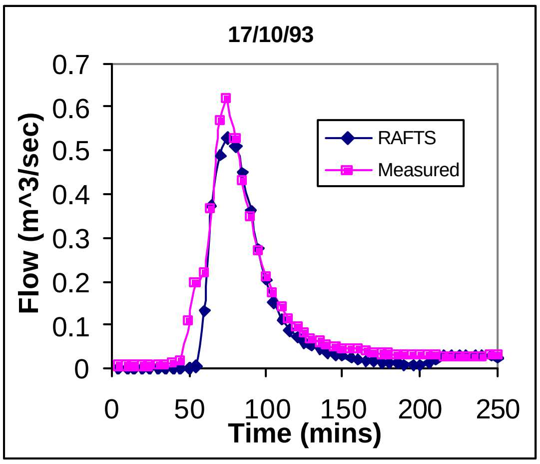
07/07/93



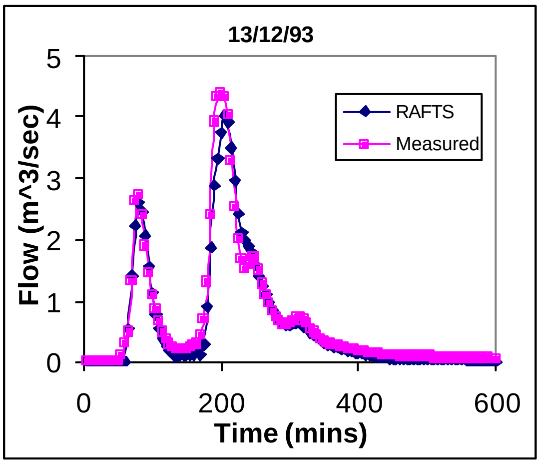
30/08/93



19/09/93



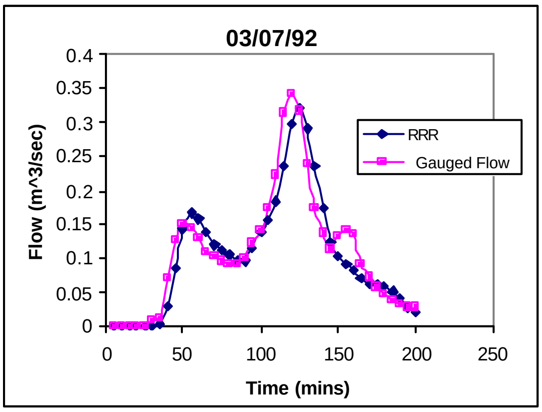
17/10/93



13/12/93

Appendix 7 – Urban Catchments RRR Verification Results

Frederick Street, Glenelg Catchment



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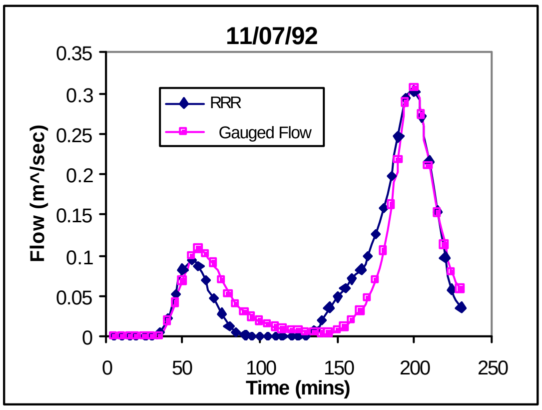

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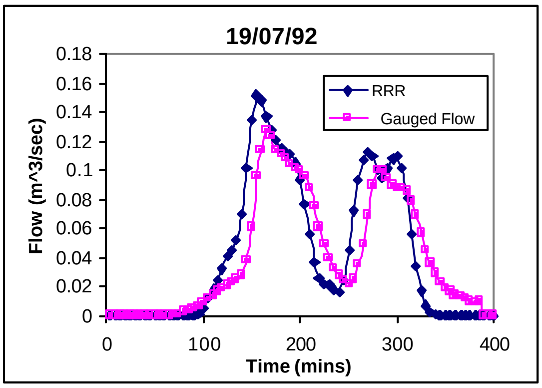

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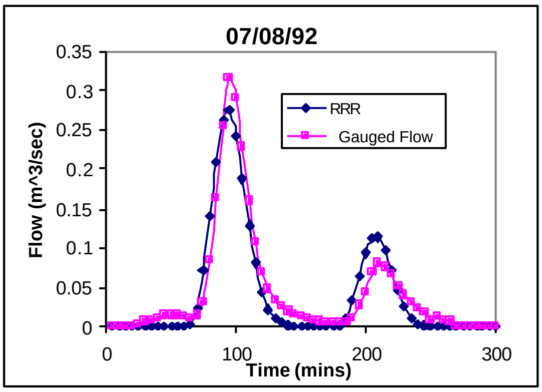

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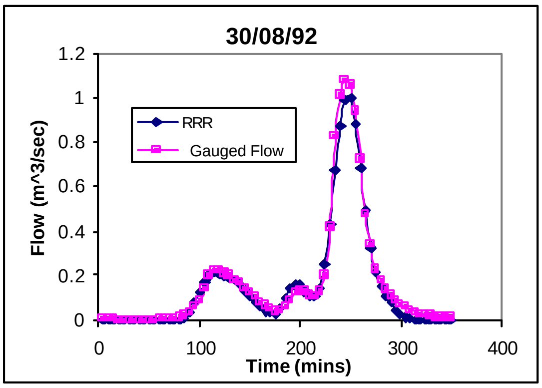

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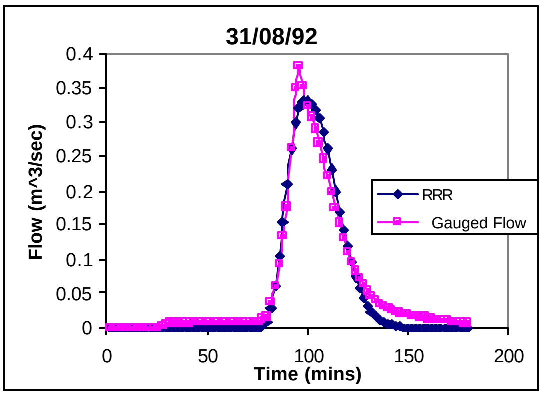

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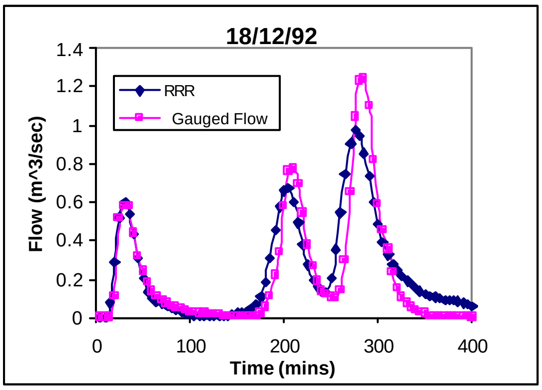

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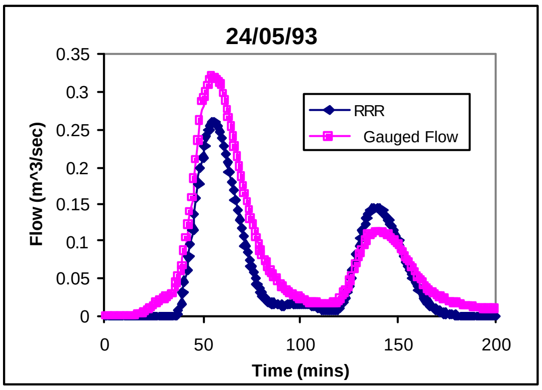

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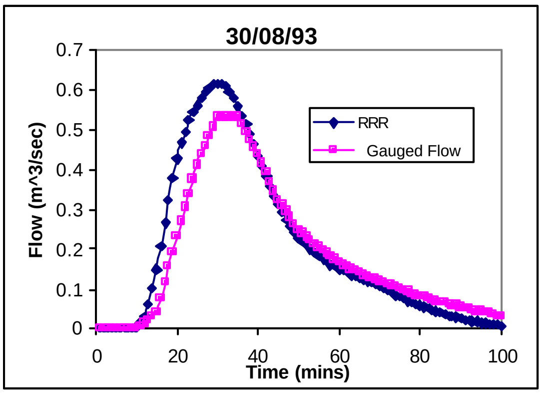

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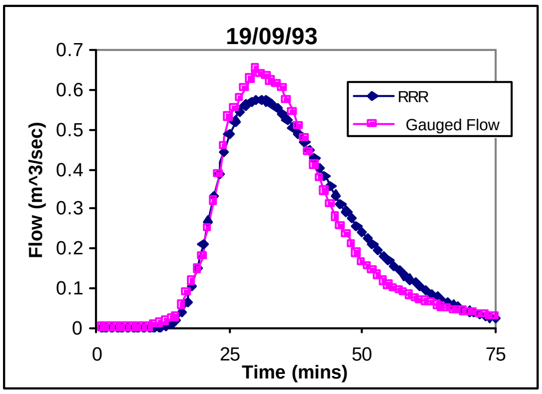

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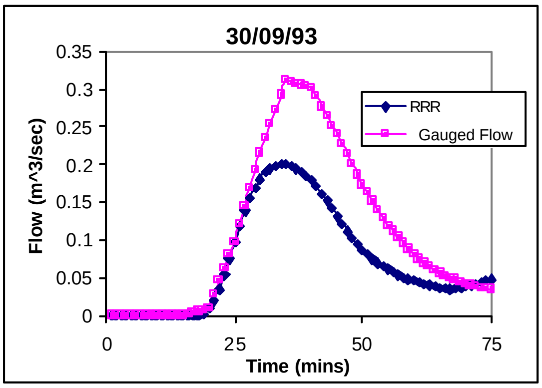

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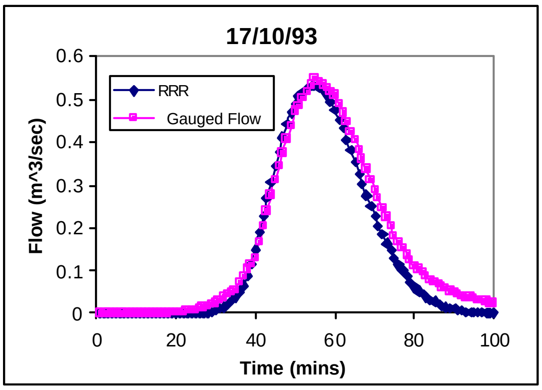

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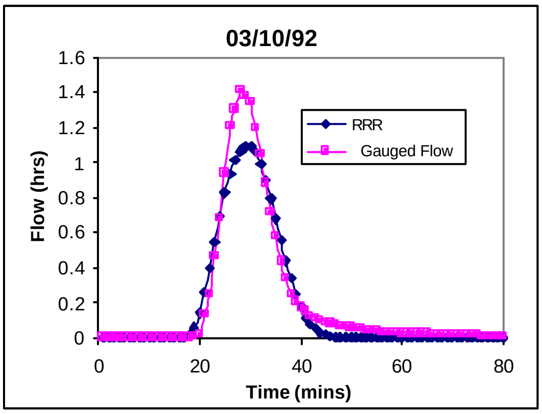
Paddocks Catchment



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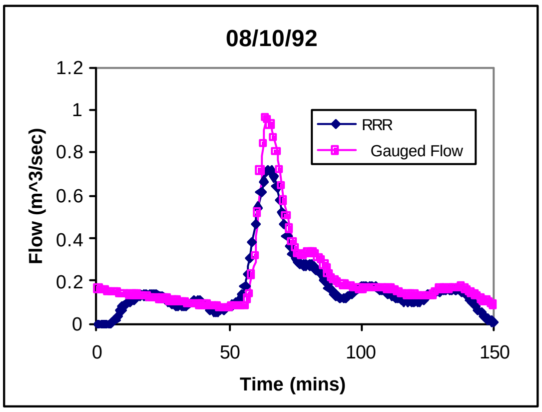

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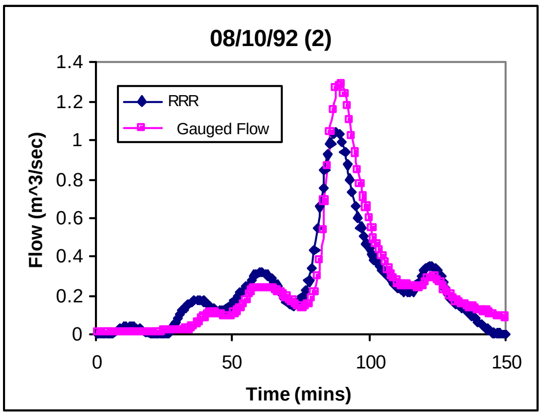

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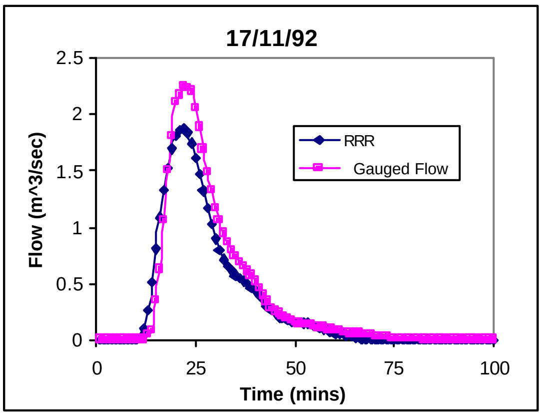

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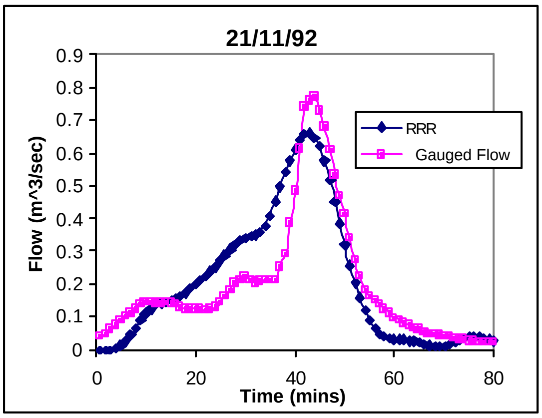

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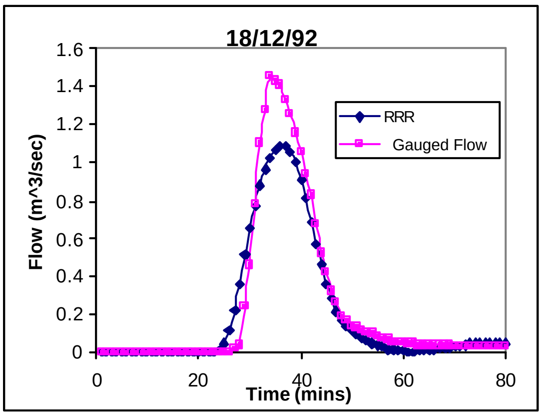

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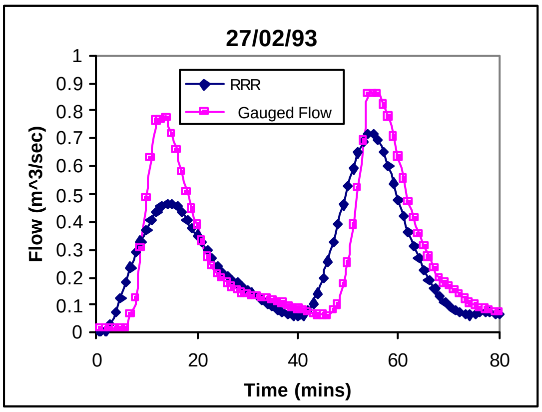

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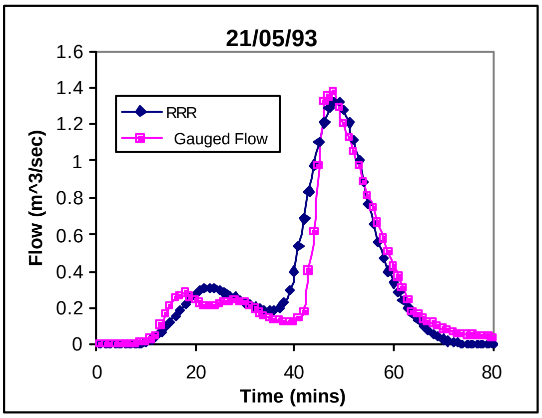

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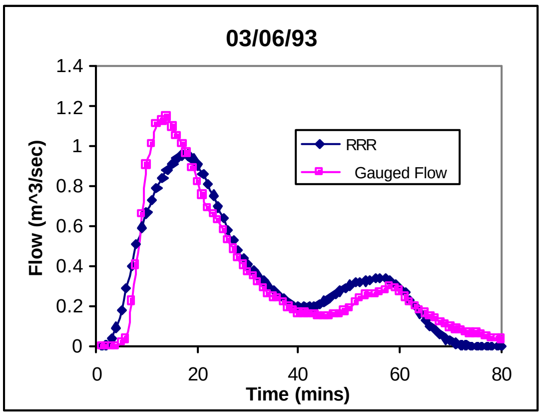

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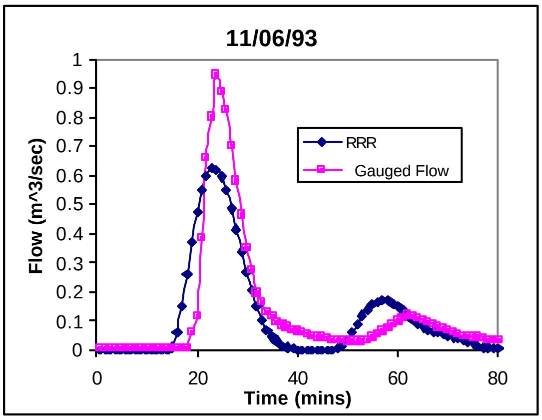

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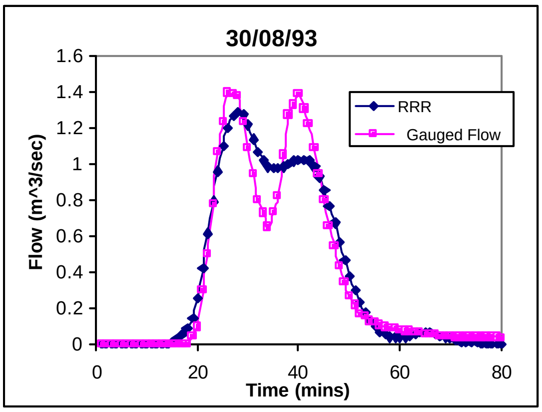

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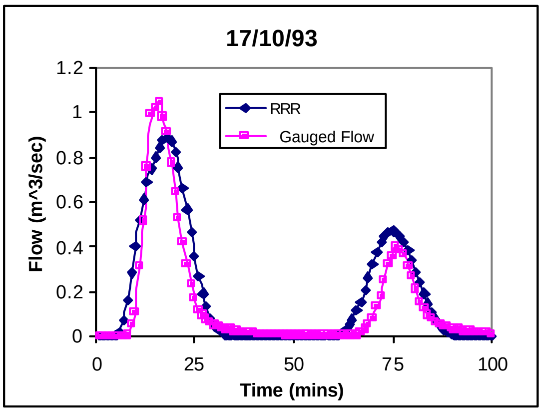

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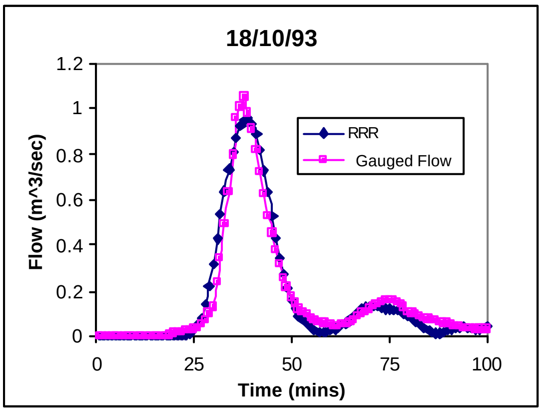

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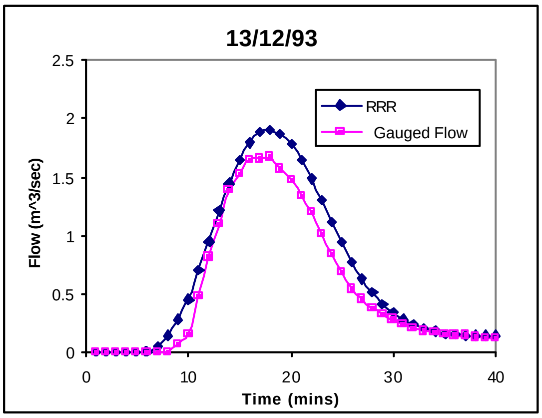
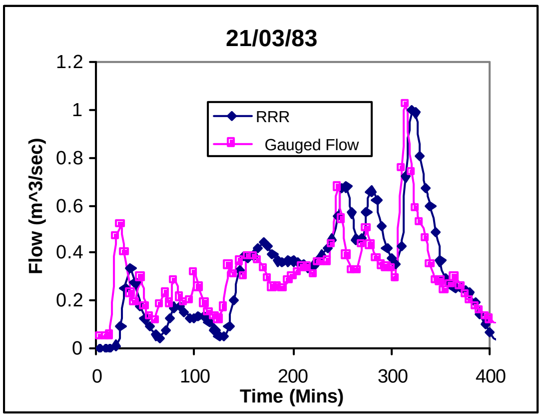


13/12/93

Jamison Park Catchment



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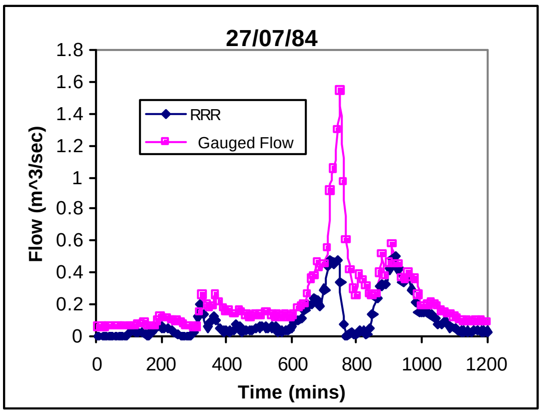

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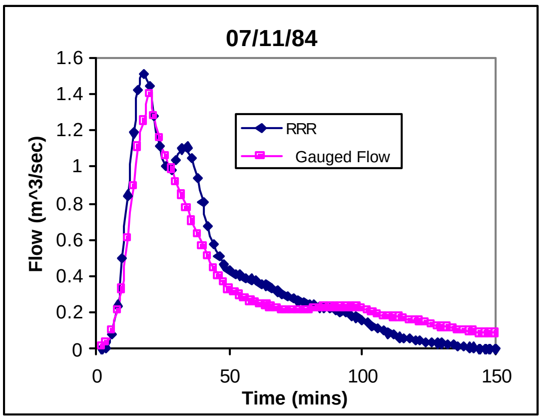

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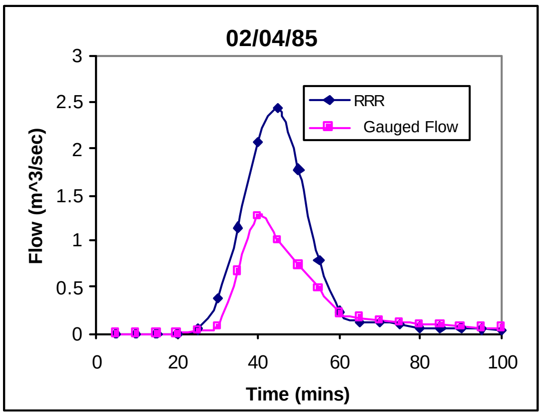

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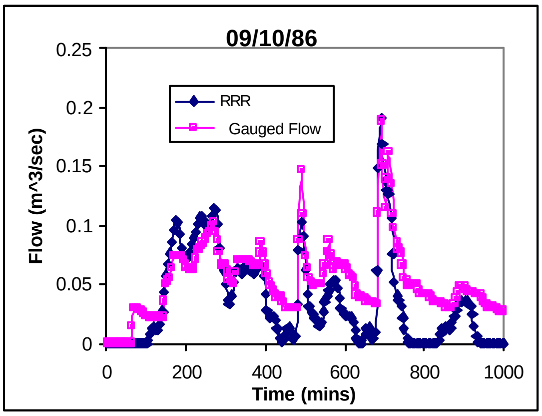

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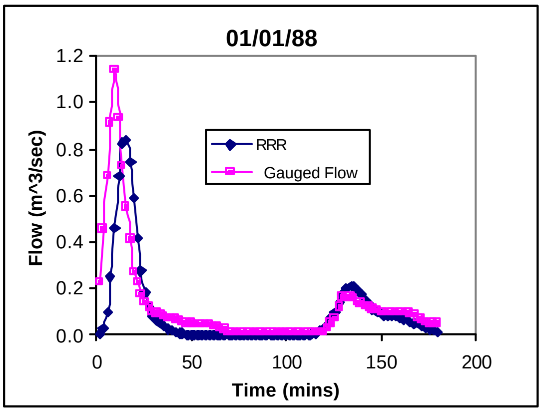

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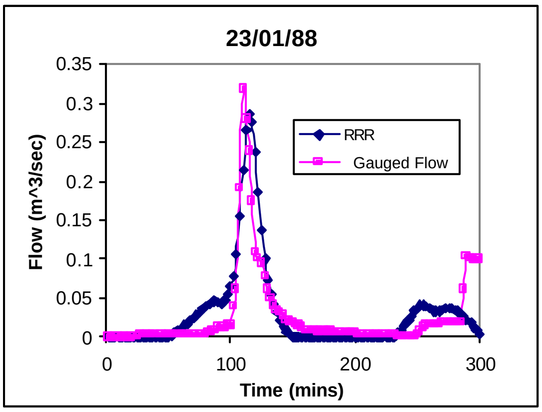

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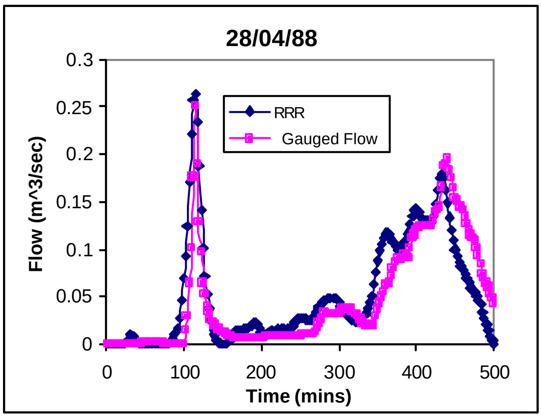

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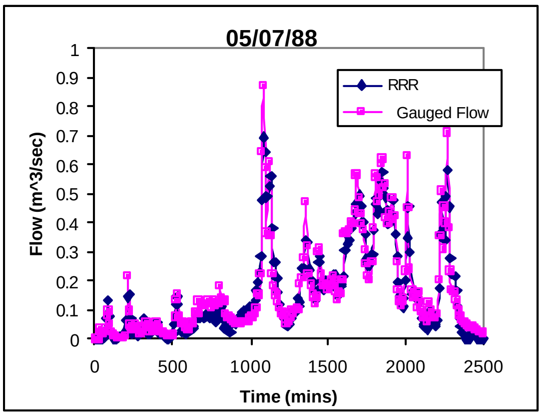

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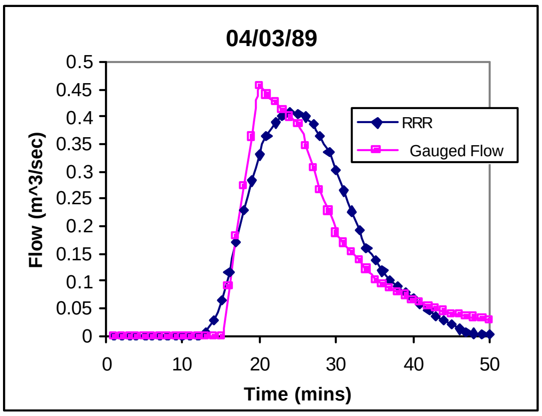

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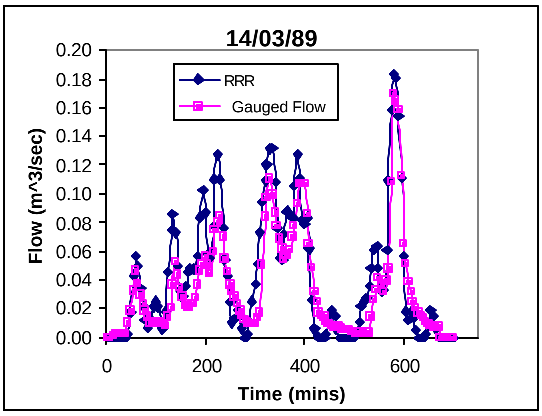

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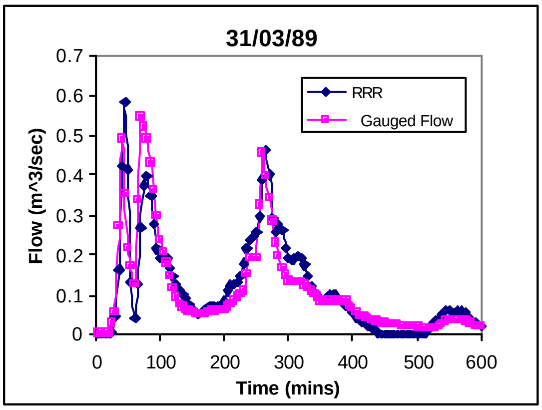


31/03/89

Appendix 8 – Rural Catchments RRR Verification

The PEST Model

The calibration of the RRR model was carried out using the parameter estimation program PEST. PEST can be applied to any model having ASCII text file input and output. The PEST program takes control of the model, by writing to the model data file before each run and then reading results from the model output file for use in the next iteration. To run PEST the following files had to be created for each event:

*.BAT	A batch file to run the model (in this case RAFTS)
*.INP	A file that is used to provide instructions usually entered by the keyboard.
*.INS	A file that instructs PEST where to find results in the output file.
*.PST	The main PEST control file, containing observation values, and information on parameters such as minimum and maximum values.
*.TPL	The PEST template file, used by PEST to create the data file for running the model.

To run the RRR model using the RAFTS program the XP interface was bypassed once the basic model was developed for each event and the *.DAT file was used to create the *.TPL file. The program RAFTSPM.EXE was then called by batch file, and the results exported to a total hydrograph file *.TOT, which could be read by PEST using the instruction file *.INS.

PEST proceeds to vary the parameters selected to minimise the difference between the observed and calculated values, in this case the hydrograph ordinates. It does this by minimising the sum of the squares of the differences between the observed and calculated values, designated phi by PEST. This is an objective function, to be minimised to provide the best fit.

There is the opportunity to provide a weighting to each observation, such that some observations are emphasised. In the case of fitting hydrographs this could be used to emphasise the fitting to the peak flow.

Torrens River

Calibration

Six events from this data set were selected for calibration. The events selected included the event having the largest peak flow (29/08/92) and five other events selected at random.

The events selected are summarised in Table A8-1.

Table A8-1 River Torrens RRR Calibrations Events

Start Date	Start Time	Duration (hrs)	Rainfall (mm)	Runoff (mm)	Observed Peak Flow (m^3/s)
30/07/89	15:00	36	15.8	10.5	12.3
29/08/92	05:00	24	67.0	60.0	66.4
23/09/92	22:00	48	31.4	20.2	14.1
18/07/96	10:00	60	43.0	19.1	11.8
03/08/96	09:00	48	21.4	19.3	12.2
28/09/96	10:30	96	62.2	25.4	20.6

Calibration was carried out using the PEST parameter optimisation model. For each event a RRR model was set up using the RAFTS interface and calibration was carried out manually to provide the initial values for input into the PEST model.

It was found that all hydrographs could be modelled using a maximum of three processes, one of which had a process lag of zero.

The parameters selected for calibration by PEST were as follows:

PL1 Loss for first process (baseflow)
IL2 Initial loss for second process (slow flow)
PL2 Loss for second process
IL3 Initial loss for third process (fast flow)
PL3 Loss for third process
k_p1 Process storage parameter for first process
k_p2 Process storage parameter for second process
k Storage parameter for channel storage

For all events and processes the proportional loss was a proportion of the total rainfall.

PEST was then run for all events selected for calibration. The model was run on a 15 or 30 minute time step, depending on the event duration, with PEST fitting modelled flows to observed flows every 75 minutes or 150 minutes, depending on the time step used for the RRR model. An exception to this was the event of 29/08/92, which had a peak of 66 m^3/s, and the hydrograph would have been poorly represented by 75 minute time steps. For this event an extra fitting of observed and modelled values was made at the time of peak flow.

The start of modelling for each event was chosen to be several hours before the start of rise of the hydrograph.

It was found that the selection of initial parameters was important if the PEST model was to converge quickly. Even so in some cases PEST ran the RRR model in excess of 200 times to provide the calibration.

The result of the PEST calibration is as follows. As a measure of the level of fit of the modelled hydrograph a ratio of mean ordinate error to the peak flow is given.

Table A8-2 River Torrens RRR Calibration Results

Event date	PL1	IL2 (mm)	PL2	IL3 (mm)	PL3	k_p1	k_p2	k (hrs)	MEAN ERROR (m^3/s)
30/07/89	0.85	6.50	0.12	*	*	0.75	0.29	0.221	0.51
29/08/92	*	15.0	0.12	52.4	0.25	*	0.39	0.268	3.20
23/09/92	0.83	6.36	0.37	*	*	2.68	0.48	0.262	0.33
18/07/96	0.81	14.4	0.58	*	*	1.07	0.36	0.087	0.86
03/08/96	0.55	4.48	0.38	*	*	1.54	0.41	0.384	0.97
28/09/96	0.78	14.6	0.25	*	*	0.61	0.49	0.287	0.92

In Table A8-2 the * indicates that there was no contribution was found from this process.

The level of fit on the depth of runoff is given in Table A8-3.

Table A8-3 River Torrens RRR Calibration Runoff

Event date	Measured Runoff (mm)	Modelled Runoff (mm)
30/07/89	10.5	11.1
29/08/92	60.0	58.8
23/09/92	20.2	21.1
03/08/96	19.1	20.1
18/07/96	19.3	20.0
28/09/96	50.8	49.4

A problem that was identified in the fitting was that the rainfall excess during the period in which two processes were occurring was in some cases larger than the event rainfall. With several processes operating the sum of the proportional rainfall excess should be less than 1 to preserve continuity.

The results are given in Table A8-4:

Table A8-4 River Torrens RRR Calibration Volumetric Runoff

Event date	Peak Flow (m^3/s)	runoff depth/ rainfall depth
30/07/89	12.3	1.03
29/08/92	66.4	1.63
23/09/92	14.1	0.80
03/08/96	11.8	1.07
18/07/96	12.2	0.61
28/09/96	20.6	1.01

It was of concern that several events had periods when the runoff was greater than the rainfall input. It could be that the rainfall input has not been well defined by the single pluviometer, the rating of the gauging station is in error, or the model is faulty.

The chance of rating error is supported by the fact that the event with the largest ratio has also the largest peak flow, as is shown by Figure A8. It may also be that in this event the rainfall variation was large.

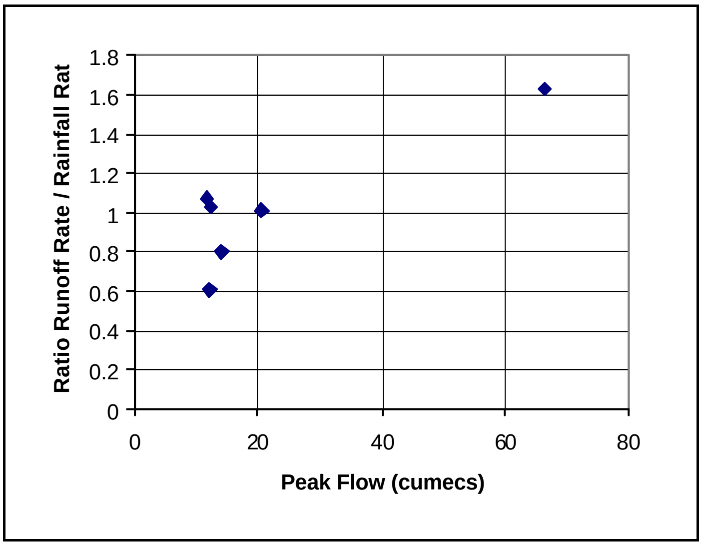


Figure A8-1 Torrens River Volumetric Runoff vs Peak Flow

In order to examine the rainfall input to the model daily rainfall data were obtained from the Bureau of Meteorology for the daily read rainfall station at Mount Pleasant, which lies within the catchment and the rainfall compared for the periods containing the modelled events.

The daily rainfall data suffered from discontinuity in the record, as rainfall was not recorded daily. Totals for several days had to be compared.

Table A8-5 shows rainfall for the event starting 30 July 1989.

Table A8-5 River Torrens RRR Calibration Rainfall Comparisons, July 1989

Date	29/7	30/7	31/7	1/8
Bureau of Meteorology	0.0	0.0	46.2	2.4
DEHAA	13.4	1.8	16.2	0.8

The rainfall totals of 0.0 for 29 July and 30 July should be no record, the sum for the three days then being 46.2mm for Bureau of Meteorology and 31.4 for the DEHAA gauge.

Table A8-6 shows rainfall for the event starting 29 August 1992.

Table A8-6 River Torrens RRR Calibration Rainfall Comparisons, August 1989

Date	29/8	30/8	31/8	1/9
Bureau of Meteorology	nr	nr	86.0	13.6
DEHAA	7.8	54.6	5.2	12.8

The rainfall total for the three days to 31 August being then 86.0mm for Bureau of Meteorology and 67.6mm for the DEHAA gauge.

Table A8-7 shows rainfall for the event starting 23 September 1992, with both records being close.

Table A8-7 River Torrens RRR Calibration Rainfall Comparisons, September 1992

Date	24/9	30/9
Bureau of Meteorology	5.4	25.6
DEHAA	5.2	26.2

Table A8-8 shows rainfall for the event starting 18 July 1996.

Table A8-9 River Torrens RRR Calibration Rainfall Comparisons, July 1996

Date	17/7	18/7	19/7	20/7	21/7	22/7
Bureau of Meteorology	6.2	0.0	16.2	nr	nr	38.8
DEHAA	3.6	0.0	14.0	27.2	1.6	1.0

The three day total to 22 July is 38.8 mm for the Bureau of Meteorology gauge and 29.6mm for the DEHAA gauge.

Table A8-10 shows rainfall for the event starting 3 August 1996:

Table A8-10 River Torrens RRR Calibration Rainfall Comparisons, August 1996

Date	3/8	4/8	5/8
Bureau of Meteorology	nr	nr	27.4
DEHAA	0.4	21.4	0.0

The three day total to 8 August is 27.4 mm for the Bureau of Meteorology gauge and 21.8mm for the DEHAA gauge.

For the event starting 28 September 1996 there was no record from the Bureau of Meteorology gauge for the 8 days until 30 September. The total from the Bureau of Meteorology gauge for this period is 66.3mm compared with 54.0mm for the DEHAA gauge.

The rainfall values are summarised in Table A8-11.

Table A8-11 River Torrens RRR Calibration Rainfall Comparison Summary

Date	BOM (mm)	DEHAA (mm)	% by which Bureau of Meteorology is higher than DEHAA
31/07/89	46.2	41.4	11.6
31/08/92	86.0	67.6	27.2
1/09/92	13.6	12.8	6.3
24/09/92	5.4	5.2	3.8
25/09/92	25.6	26.2	-2.3
17/07/96	6.2	3.6	72.2
19/07/96	16.2	14.0	15.7
22/07/96	38.8	29.8	30.2
05/08/96	27.4	21.8	25.7
30/09/96	66.3	54.0	22.8

It can be seen that the Bureau of Meteorology gauge reading exceeds that of the DEHAA gauge, in most cases by 10% to 20%, which is enough to make the calibrated losses realistic. The two gauges are within 2.5 km, in an area that would not have a steep rainfall gradient. There is thus the possibility that the pluviometer is not recording true rainfall.

It is most likely that there is a combination of rating errors for high flows and error in the recorded rainfall for all events. The verification runs will be carried out on the basis of the calibrated parameters, but the parameters related to proportional or continuing loss are probably in error.

Since the event of 29/08/92 was not fitting well it was decided to attempt another calibration using an absolute continuing loss rate for the third process, with the following result.

Table A8-12 River Torrens RRR Calibration - With Continuing Loss

Event date	PL1	IL2 (mm)	PL2	IL3 (mm)	CL3	k_p1	k_p2	k	MEAN ERROR (m^3/s)
29/08/92	*	15.2	0.21	53.4	0.21	*	0.36	0.282	3.0

The fit was only marginally improved.

The next step was to use the continuing loss rate for the second process as well, and the result is given in Table A8-13.

Table A8-13 River Torrens RRR Calibration - With Continuing Loss

Event date	PL1	IL2 (mm)	CL2	IL3 (mm)	CL3	k_p1	k_p2	k (hrs)	MEAN ERROR (m^3/s)
30/07/89	0.74	6.7	0.52	*	*	0.857	0.257	0.243	0.33
29/08/92	0.81	15.0	1.51	52.6	1.84	1.69	0.282	0.278	2.63
23/09/92	0.55	12.9	2.55	*	*	0.949	0.275	0.289	0.59
18/07/96	0.97	15.0	1.15	*	*	10.0	0.581	0.071	1.16
03/08/96	0.51	4.5	1.56	*	*	1.41	0.399	0.417	0.90
28/09/96	0.59	13.5	0.66	*	*	1.53	0.428	0.332	1.24

The use of a continuing loss gave a better fit in three of the six events, but a proportional loss was used for verification.

Verification

The first step in the verification is to determine parameters to be used with the event rainfall.

The API and initial baseflow associated with each event was determined, and listed in Table A8-14.

Table A8-14 River Torrens RRR Calibration Events API & Initial Baseflow

Event Date	Start Time	API (mm)	Initial Base Flow (m^3/s)
30/07/89	15:00	32.4	0.238
29/08/92	05:00	23.7	0.065
23/09/92	22:00	40.0	0.221
18/07/96	10:00	26.6	0.057
03/08/96	09:00	40.1	0.440
28/09/96	10:30	27.1	0.036

It would be expected that there would be a correlation between API and the initial baseflow, as both are a measure of catchment condition, and in the above case there is, with the baseflow increasing with increasing API.

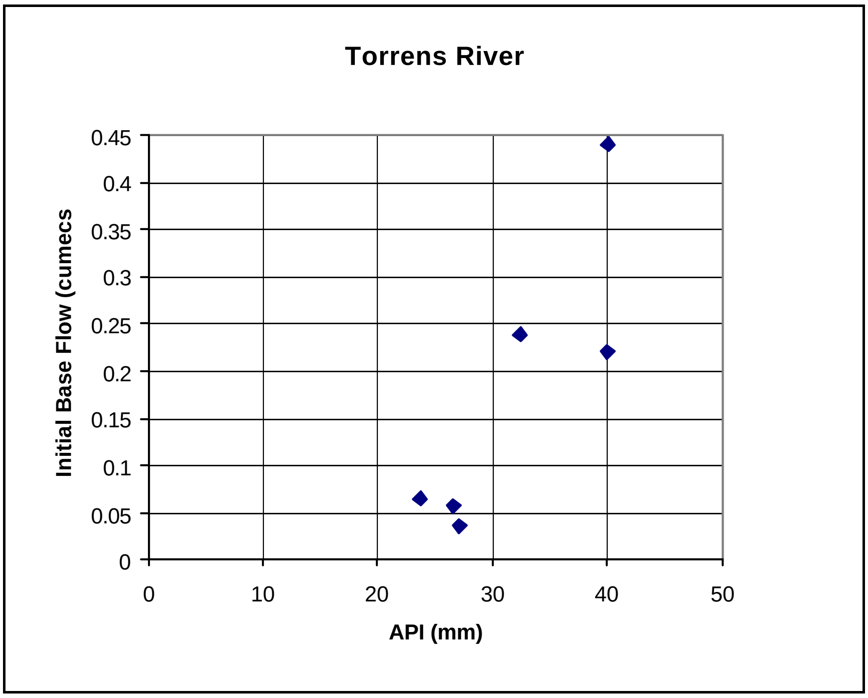


Figure A8-2 Torrens River Initial Baseflow vs API

To determine whether there was any relationship between IL2 and API or initial baseflow scatter diagrams were constructed. Figure A8-3 and Figure A8-4 are the two scatter diagrams.

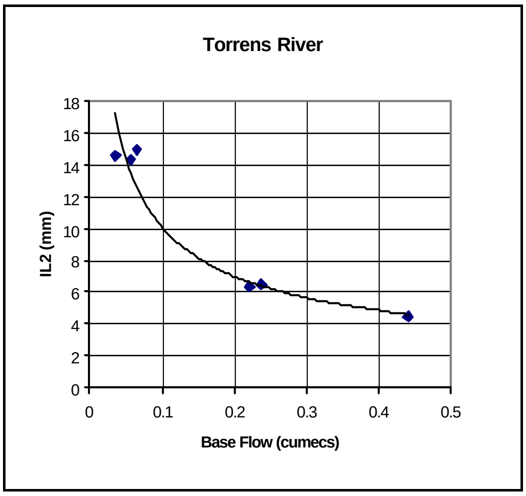


Figure A8-3 Torrens River IL2 vs Initial Baseflow

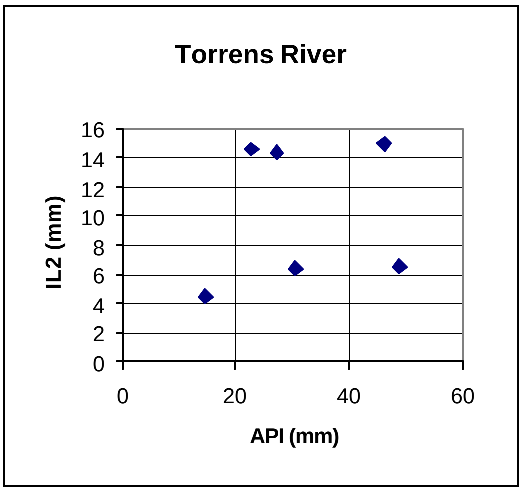


Figure A8-4 Torrens River IL2 vs API

A relationship is evident between the initial baseflow and IL2 as follows:

$$IL2 = 3.0(Initial\,BaseFlow)^{-0.53} \qquad \text{Equation A8-1}$$

$$(R^2 = 0.96, SEE = 1.83mm)$$

No other relationships could be seen between these initial conditions and losses. Verification runs were therefore carried out using this relationship for IL2, and weighted mean values for all other losses and storage parameters.

The parameters used are given in Table A8-15.

Table A8-15 River Torrens Verification Parameters

IL1(mm)	IL2 (mm)	PL1	PL2	k	k_p1	k_p2
0.0	Derived from relationship	0.75	0.28	0.258	1.257	0.406

It was assumed that the process 3 would not occur, as there was insufficient evidence from the calibration runs as to the losses to be applied.

The result of the verification process is given in Table A8-16.

Table A8-16 River Torrens Verification Results

Date	Gauged peak flow (m^3/s)	Predicted peak flow (m^3/s)	Predicted peak / gauged peak	Gauged volume (m^3)	Predicted volume (m^3)	Predicted volume / gauged volume
07/10/92	23.6	15.2	0.69	958 600	855 300	0.89
16/12/92	27.0	26.7	0.99	2 365 000	2 890 000	1.22
20/07/95	34.4	30.4	0.88	1 479 000	1 550 000	1.05
25/08/96	12.0	16.5	1.38	296 600	423 200	1.43
		mean	0.97			1.15

Inverbrackie Creek

Calibration

Data were obtained for 13 highest flow events in the period of record, and calibration carried out with PEST. A weighting factor of 1 was used for all ordinates.

The events that were modelled were chosen at random. Table A8-17 summarises the events selected for calibration.

Table A8-17 Inverbrackie Creek RRR Calibration Events

Start Date	Start Time	Duration (hrs)	Rainfall (mm)	Runoff (mm)	Peak Flow (m^3/s)
14/07/87	12:30	48	46.8	20.5	7.87
05/06/88	06:30	36	46.1	20.0	4.80
19/08/90	22:00	75	25.2	18.5	2.97
04/07/90	02:00	60	28.2	11.1	2.55
29/08/92	06:30	80	102.4	104.0	11.43
28/09/96	09:00	75	72.2	56.0	6.22

As with the River Torrens at Mount Pleasant the fact that the measured runoff is greater than the rainfall for the event of 29 August 1992 is of concern. Either the rainfall recorded at the gauging station is not representative of the catchment rainfall or the rating of the gauging station is in error. There are no other rainfall stations in the catchment so the latter possibility is difficult to confirm.

Table A8-18 summarises the results of the calibration by PEST.

Table A8-18 Inverbrackie Creek RRR Calibration Results

Event date	PL1	IL2 (mm)	PL2	IL3 (mm)	PL3	k_p1	k_p2	k (hrs)	MEAN ERROR (m^3/s)
14/07/87	0.87	22.7	0.37	*	*	0.84	0.31	0.168	0.204
05/06/88	0.73	21.0	0.69	*	*	0.62	0.12	0.299	0.147
19/08/90	0.71	2.5	0.44	*	*	1.88	0.37	0.139	0.094
04/07/90	0.59	*	*	*	*	0.62	*	0.213	0.164
29/08/92	0.82	14.8	0.00	*	*	2.31	0.40	0.231	0.524
28/09/96	0.60	23.0	0.45	*	*	0.89	0.34	0.146	0.266

Again the calibration was carried out using a continuing loss instead of a proportional loss with the results given in Table A8-19.

Table A8-19 Inverbrackie Creek RRR calibration with Continuing Loss

Event date	PL1	IL2 (mm)	CL2	IL3 (mm)	PL3	k_p1	k_p2	k (hrs)	MEAN ERROR (m^3/s)
14/07/87	0.78	18.6	2.43	*	*	0.949	0.010	0.5	0.433
05/06/88	0.75	20.1	4.80	*	*	0.564	0.511	0.131	0.178
19/08/90	0.54	4.42	1.66	*	*	1.83	0.347	0.073	0.163
04/07/90	0.76	17.9	1.28	*	*	0.408	0.401	0.203	0.111
29/08/92	0.70	14.4	0.49	*	*	1.93	0.362	0.255	0.428
28/09/96	0.70	15.0	1.81	*	*	0.876	0.151	0.154	1.061

The use of a proportional loss gave a better fit in most cases, and was used for verification.

Verification

The API and initial baseflow associated with each calibration event was determined, as listed in Table A8-20.

Table A8-20 Inverbrackie Creek RRR Calibration Event API and Initial Baseflow

Event Date	Start Time	API (mm)	Initial Base Flow (m^3/s)
14/07/87	12:30	20.6	0.014
05/06/88	06:30	67.9	0.079
19/08/90	22:00	41.6	0.057
04/07/90	02:00	75.9	0.059
29/08/92	06:30	26.1	0.019
28/09/96	09:00	30.9	0.022

The IL2 was plotted versus both API and initial baseflow to determine if a relationship existed.

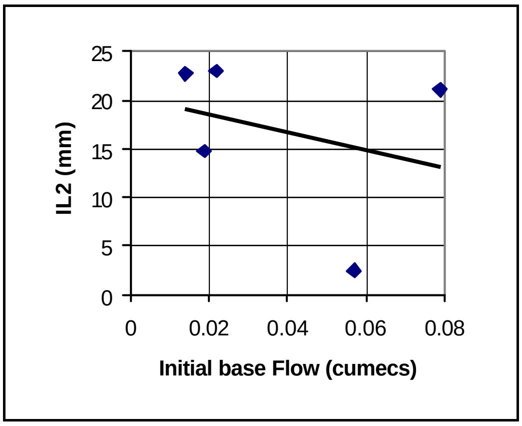


Figure A8-5 Inverbrackie Creek IL2 vs Initial Baseflow

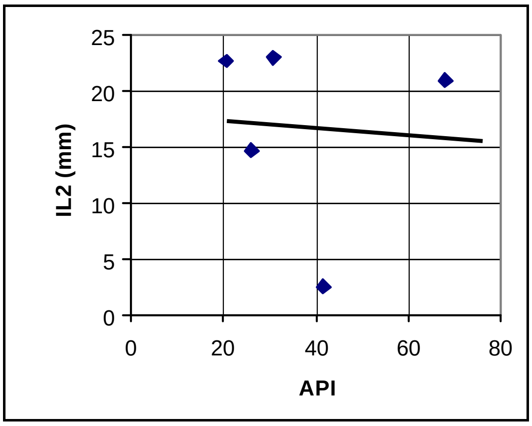


Figure A8-6 Inverbrackie Creek API vs IL2

There was no strong relationship discernible between initial baseflow or API and IL2 (r^2 = 0.09 and r^2 = 0.006), so verification was carried out with the weighted mean values for all parameters, including IL2, as follows:

Table A8-21 Inverbrackie Creek Verification Parameters

IL1(mm)	IL2 (mm)	PL1	PL2	k	k_p1	k_p2
0.0	16.9	0.74	0.42	0.198	1.181	0.299

The results of the verification are given in Table A8-22:

Table A8-22 Inverbrackie Creek Verification Results

Date	Gauged peak flow (m^3/s)	Predicted peak flow (m^3/s)	Predicted peak /gauged peak	Gauged volume (m^3)	Predicted volume (m^3)	Predicted volume/ gauged volume
07-10-92	7.69	6.36	0.83	311 500	298 000	0.96
14-09-92	4.01	2.71	0.67	356 200	319 300	0.90
22-06-87	5.20	5.76	1.11	221 600	308 600	1.40
21-07-95	4.60	3.98	0.87	513 100	416 700	0.81
23-05-88	3.89	9.29	2.39	61 700	231 300	3.75
02-08-96	4.07	4.07	1.00	251 100	206 000	0.82
		mean	1.14			1.44

If the event of the 23/05/88 is ignored as an outlier, as both the peak flow and volume ratios are large then the mean ratio of predicted/gauged peak flows is 0.90, and the ratio of predicted/gauged volumes is 0.98. The gauged rainfall in the event of 23/05/88 may not have been representative of true catchment rainfall.

Echunga Creek

Calibration

Data were obtained for 13 events having the highest flows in the period of record, and calibration carried out with PEST.

The events that were modelled were chosen at random. Table A8-23 summarises the events selected for calibration.

Table A8-23 Echunga Creek RRR Calibration Events

Start Date	Start Time	Duration (hrs)	Rainfall (mm)	Runoff (mm)	Peak Flow (m^3/s)	Initial Baseflow (m^3/s)
10/09/89	06:00	86	56.0	17.0	6.2	0.239
04/07/90	16:00	36	39.4	10.8	16.8	0.210
14/08/90	12:00	48	36.6	11.9	7.2	0.069
29/08/92	02:30	48	79.4	47.2	42.9	0.239
17/12/92	10:00	86	80.6	26.3	10.7	0.033
20/07/95	18:00	72	83.8	91.6	25.2	0.349

The events of 10/09/89 and 17/12/92 were modelled using a 30 minute time step. All others were modelled with a 15 minute time step.

Table A8-24 summarises the results of the calibration by PEST.

Table A8-24 Echunga Creek RRR Calibration Results

Event date	PL1	IL2 (mm)	PL2	IL3 (mm)	PL3	k_p1	k_p2	k (hrs)	MEAN ERROR (m^3/s)
10/09/89	0.84	16.8	0.79	*	*	1.81	0.432	0.267	0.214
04/07/90	0.96	5.2	0.72	*	*	1.80	0.327	0.246	0.361
14/08/90	0.94	5.1	0.65	*	*	2.35	0.530	0.518	0.214
29/08/92	*	17.0	0.18	*	*	*	0.467	0.263	2.07
17/12/92	0.81	1.8	0.79	*	*	3.46	0.324	0.289	0.967
20/07/95	0.80	7.1	0.75	25	0.82	1.10	0.119	0.371	1.07

Again the calibration was carried out using a continuing loss instead of a proportional loss with the result given in Table A8-25.

Table A8-25 Echunga Creek RRR Calibration With Continuing Loss

Event date	PL1	IL2 (mm)	CL2	IL3 (mm)	PL3	k_p1	k_p2	k (hrs)	MEAN ERROR (m^3/s)
10/09/89	0.72	10	5.48	*	*	1.66	0.395	0.360	0.667
04/07/90	0.92	19.6	3.05	*	*	1.22	0.248	0.271	0.308
14/08/90	*	2.9	2.07	*	*	*	0.715	0.404	0.419
29/08/92	*	17.8	0.71	*	*	*	0.466	0.279	2.053
17/12/92	*	4.85	1.45	*	*	*	0.846	0.347	7.826
20/07/95	*	4.14	1.31	*	*	*	0.750	0.095	1.897

In most cases the error in the fitted hydrograph is greater than for the use of the proportional loss, and in some cases the fit is very poor. Process 1 was not used for 4 of the events, as a better fit was achieved without this process.

The use of the proportional loss is therefore preferable.

Verification

The API and initial baseflow was determined for each calibration, as given in Table A8-26.

Table A8-26 Echunga Creek RRR Calibration Event API and Initial Baseflow

Event Date	Start Time	API (mm)	Initial Base Flow (m^3/s)
10/09/89	06:00	18.8	0.239
04/07/90	16:00	75.9	0.210
14/08/90	12:00	31.2	0.069
29/08/92	02:30	24.7	0.239
17/12/92	10:00	19.4	0.033
20/07/95	18:00	56.7	0.349

The IL2 was plotted against initial baseflow and API to determine if any relationship existed.

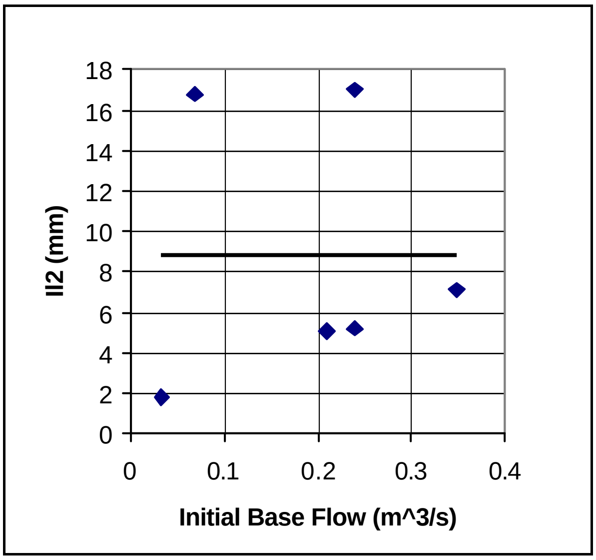


Figure A8-7 Echunga Creek IL2 vs Initial Baseflow

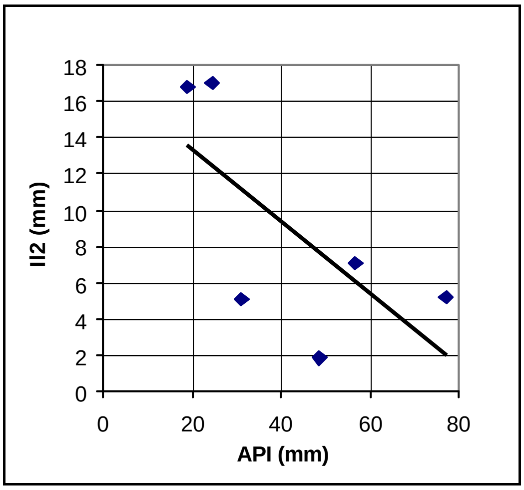


Figure A8-8 Echunga Creek IL2 vs API

As with Inverbrackie Creek there was no strong relationship between IL2 and API or initial baseflow (r^2 = 0.00 and 0.46), so verification was carried out using the weighted mean for all parameters, with the result given in Table A8-28.

Table A8-27 Echunga Creek Verification Parameters

IL1(mm)	IL2 (mm)	PL1	PL2	k	k_p1	k_p2
0.0	8.7	0.89	0.73	0.329	1.945	0.375

Table A8-28 Echunga Creek RRR Verification Results

Date	Gauged Peak Flow (m^3/s)	Predicted Peak Flow (m^3/s)	Predicted Peak / Gauged Peak	Gauged volume (m^3)	Predicted volume (m^3)	Predicted volume / gauged volume
02-07-92	7.48	13.92	1.86	122 500	374 300	3.06
06-07-93	13.62	22.03	1.62	304 000	1 140 000	3.75
07-07-91	4.37	8.44	1.93	115 500	240 600	2.08
10-07-95	6.06	8.13	1.34	223 700	343 800	1.54
28-09-96	22.76	12.07	0.53	903 500	554 800	0.61
30-07-96	38.24	10.30	0.26	1 895 900	1 050 000	0.55
		mean	1.26			1.93

Scott Creek

Calibration

Table A8-29 summarises the storm events used for calibration:

Table A8-29 Scott Creek RRR Calibration Events

Start Date	Start Time	Duration (hrs)	Rainfall (mm)	Runoff (mm)	Peak Flow (m^3/s)	Initial Baseflow (m^3/s)
14/09/91	18:00	36	22.0	6.1	6.0	0.188
16/12/92	22:00	40	41.4	10.5	6.1	0.371
05/07/93	09:00	36	69.2	4.2	3.6	0.060
11/07/95	12:30	24	25.8	10.1	10.0	0.135
20/07/95	23:00	75	67.8	36.9	8.7	0.307
02/08/96	13:30	75	88.0	36.8	15.2	0.447

PEST was used to minimise the value of the least squares of the differences between the observed and the predicted hydrographs, with the following results given in Table A8-30.

Table A8-30 Scott Creek RRR Calibration Results

Event date	PL1	IL2 (mm)	PL2	IL3 (mm)	PL3	k_p1	k_p2	k (hrs)	MEAN ERROR (m^3/s)
14/09/91	0.84	10.8	0.73	*	*	1.3	0.391	0.075	0.099
16/12/92	0.85	9.61	0.78	*	*	2.55	0.459	0.433	0.329
05/07/93	0.99	40.0	0.85	*	*	2.0	0.567	0.226	0.320
11/07/95	0.74	21.5	0.0	*	*	1.04	0.323	0.292	0.241
20/07/95	0.52	9.13	0.86	*	*	2.43	0.297	0.173	0.774
02/08/96	0.72	43.1	0.74	*	*	1.66	0.535	0.147	0.399

As before, a continuing loss for process 2 and 3 was also optimised, and the result given in Table A8-31.

Table A8-31 Scott Creek RRR Calibration With Continuing Loss

Event date	PL1	IL2 (mm)	CL2	IL3 (mm)	CL3	k_p1	k_p2	k (hrs)	MEAN ERROR (m^3/s)
14/09/91	0.68	50.0	8.62	*	*	1.929	0.537	0.122	0.475
16/12/92	0.97	40.0	5.46	*	*	1.038	0.398	0.433	0.207
05/07/93	0.79	10.0	4.73	*	*	1.002	0.301	0.056	0.240
11/07/95	0.65	5.75	11.1	*	*	9.659	0.756	0.562	0.376
20/07/95	0.81	16.8	0.54	*	*	1.727	0.551	0.323	0.354
02/08/96	0.39	>88.0	*	*	*	2.286	*	0.331	0.987

The standard error of estimate of the continuing loss versus the proportional loss varied, but the proportional loss case had the best fit, and the continuing loss case the worst fit, being the event of 02/08/96. This event had a better fit with only one contribution, being that of process 1 with a proportional loss.

Verification

The API and initial baseflow associated with each event was determined and is given in Table A8-32.

Table A8-32 Scott Creek RRR Calibration Event API and Initial Baseflow

Event Date	Start Time	API (mm)	Initial Base Flow (m^3/s)
14/09/91	18:00	46.0	0.188
16/12/92	22:00	16.6	0.371
05/07/93	09:00	20.0	0.060
11/07/95	12:30	40.1	0.135
20/07/85	23:00	64.2	0.307
02/08/96	13:30	36.3	0.447

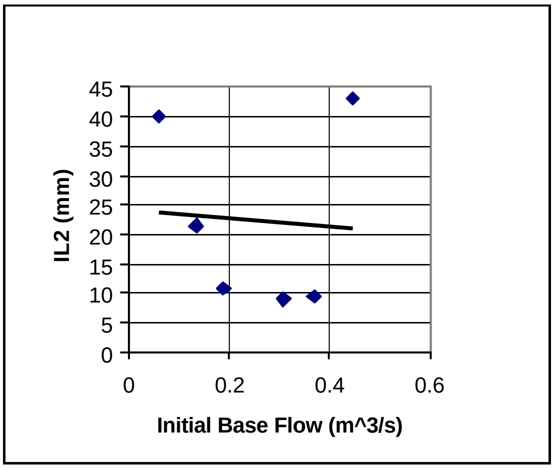


Figure A8-9 Scott Creek IL2 vs Initial Baseflow

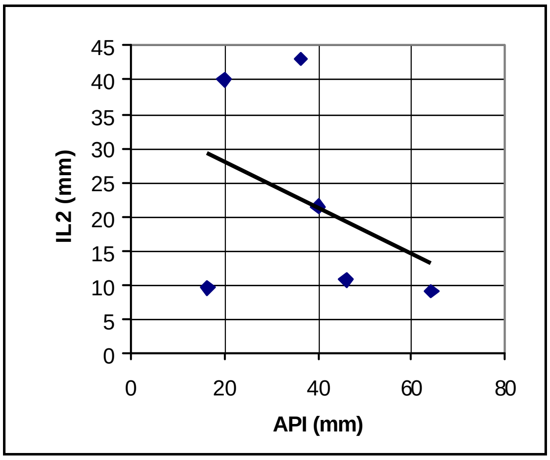


Figure A8-10 Scott Creek IL2 vs API

There were no strong relationships discernible between IL2 and API (r^2 = 0.004 and 0.15). Therefore the weighted mean values of all parameters were used for the verification, as given in Table A8-33.

Table A8-33 Scott Creek Verification Parameters

IL1 (mm)	IL2 (mm)	PL1	PL2	k	k_p1	k_p2
0.0	21.6	0.78	0.76	0.234	0.873	0.241

Verification was carried out as before, with the result given in Table A8-34

Table A8-34 Scott Creek RRR Verification Results

Date	Gauged Peak Flow (m^3/s)	Predicted Peak Flow (m^3/s)	Predicted Peak / Gauged Peak	Gauged volume (m^3)	Predicted volume (m^3)	Predicted volume / gauged volume
08-10-92	9.04	5.64	0.62	682 000	507 800	0.74
18-07-96	4.46	2.40	0.54	265 000	259 500	0.97
25-08-91	7.87	1.88	0.24	232 700	139 400	0.60
29-08-92	14.89	10.86	0.73	1 242 000	906 300	0.73
28-09-96	12.83	8.85	0.69	873 600	567 000	0.65
		Mean	0.56			0.74

Celia Creek

Calibration

Data were obtained for 14 periods having the highest flows in the duration of record, and calibration carried out with PEST. A weighting factor of 1 was used for all ordinates.

The events that were modelled were chosen at random. The following table summarises the events selected for calibration.

Table A8-35 Celia Creek RRR Calibration Events

Start Date	Start Time	Duration (hrs)	Rainfall (mm)	Runoff (mm)	Peak Flow (m^3/s)
10/03/92	12:00	72	72.9	29.1	31.9
27/01/93	12:00	125	381.4	155.4	50.0
20/01/95	12:00	48	15.6	17.8	26.9
04/03/96	08:15	125	119.9	29.8	13.7
09/04/96	12:00	48	76.3	12.9	14.3
01/01/97	00:30	100	381.0	256.7	100.7
30/01/97	00:00	96	65.8	31.6	37.0
19/02/97	00:00	192	311.1	142.6	35.7
01/03/97	00:15	48	160.6	84.7	149.7
15/01/98	18:30	90	113.5	22.2	18.9

Initial investigation and fit runs indicated that there were substantial differences between the pluviometer records at the three gauging stations. A Thiessen mean rainfall was used in the calibration runs, but some events did not model well. It was assumed that this was due to the mean pluviometer record not being representative of the true catchment rainfall.

Table A8-36 summarises the results of the calibration by PEST.

Table A8-36 Celia Creek RRR Calibration Results

Event date	PL1	IL2 (mm)	PL2	IL3 (mm)	PL3	k_p1	k_p2	k (hrs)	Mean Error (m^3/s)
10/03/92	0.95	76.8	0.48	*	*	0.800	2.030	0.770	2.43
27/01/93	0.87	105	0.45	*	*	0.000	0.642	1.321	3.96
20/01/95	0.49	2.1	0.15	*	*	0.999	1.038	0.854	1.04
04/03/96	0.79	20.3	0.92	*	*	1.712	0.692	0.203	2.04
09/04/96	0.86	>76.3	*	*	*	1.279	*	0.386	0.84
01/01/97	0.48	>381	*	93.3	0.87	1.390	*	0.247	10.4
30/01/97	0.48	>65.8	*	*	*	1.592	*	0.424	2.27
19/02/97	0.52	>311	*	*	*	2.224	*	0.641	5.47
01/03/97	0.60	25.2	0.61	*	*	1.896	0.155	0.313	3.64
15/01/98	0.65	>114	*	*	*	1.293	*	0.465	0.83

The results show considerable variation and some unusual values (for example the k_p1 of the event of 27/01/93 is zero). For this reason it was decided that the model would be divided into a 6 sub-area model, so that the rainfall from all three pluviometers could be applied differentially across the catchment.

The change to a six sub-area model meant that the PEST calibration was more complicated. A total of 25 parameters were required to be optimised, but most of these parameters were tied, that is their values were tied to other parameters. In effect there are only the same number of variables as the one sub-area model. It was also decided to use a two process model, on the basis that only one of the initial calibrations showed a process 3 contribution, and process 3 would not be expected in humid catchments such as Celia Creek (according to Dunne (1983). If the fit was not better for the event of 01/01/97 with two processes then consideration would be given to calibration with the third process.

The results are given in Table A8-37. The values given for the storage and lag parameters are the basic parameters, given for sub-catchment e, one of the 6 sub-catchments. That is the other sub-area storages and channel lags are tied to those given.

Table A8-37 Celia Creek Calibration Results - 6 Sub-area Model

Event date	PL1	IL2 (mm)	PL2	k1e	k2e	ke (hrs)	Mean Error (m^3/s)
10/03/92	0.63	>72.9	*	1.264	*	0.170	6.58
27/01/93	0.77	132.1	0.53	1.276	0.732	0.326	3.96
20/01/95	0.85	20.0	0.51	1.278	0.319	0.183	7.45
04/03/96	0.89	32.4	0.68	5.020	0.506	0.370	1.13
09/04/96	0.86	>76.3	*	0.960	*	0.169	1.00
01/01/97	0.59	0	0.76	1.390	0.597	0.011	10.3
30/01/97	0.0	29.5	0	0.823	0.620	0.340	3.84
19/02/97	0.16	27.8	0.99	3.63	2.04	0.109	4.95
01/03/97	0.57	29.6	0.65	1.440	0.304	0.046	3.31
15/01/98	0.97	45.8	0.57	0.693	0.977	0.239	1.12

Verification

The API and initial baseflow associated with each calibration event was determined, as listed in Table A8-38.

Table A8-38 Celia Creek RRR Calibration Event API and Initial Baseflow

Event Date	Start Time	API (mm)	Initial Base Flow (m^3/s)
10/03/92	12:00	76.8	1.07
27/01/93	12:00	99.5	0.16
20/01/95	12:00	135.0	1.20
04/03/96	08:15	63.9	0.33
09/04/96	12:00	42.2	0.27
01/01/97	00:30	196.8	0.70
30/01/97	00:00	173.3	4.31
19/02/97	00:00	90.5	1.65
01/03/97	00:15	128.6	4.93
15/01/98	18:30	73.4	0.13

The IL2 was plotted versus both API and initial baseflow to determine if a relationship existed.

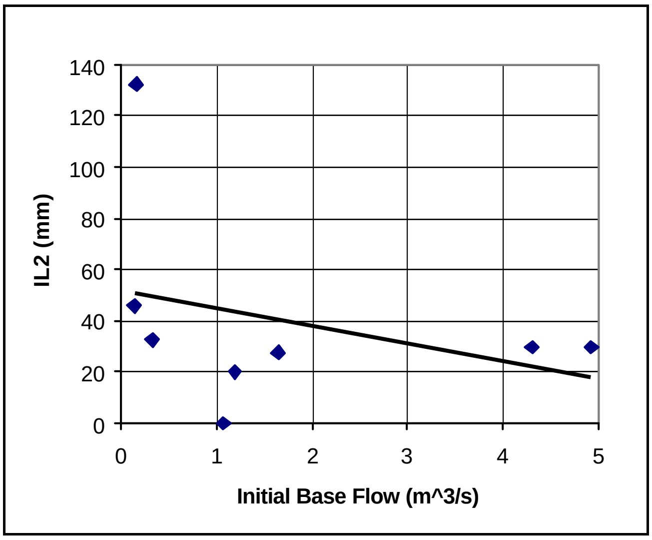


Figure A8-11 Celia Creek IL2 vs Initial Baseflow

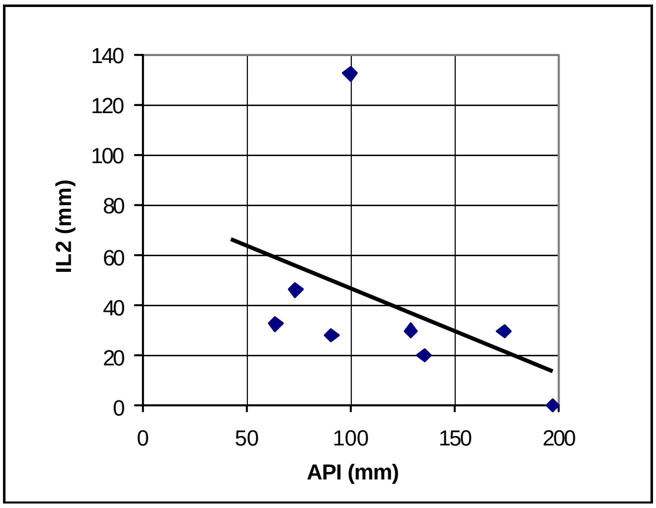


Figure A8-12 Celia Creek IL2 vs API

There was no strong relationship discernible between API or initial baseflow and IL2 (r^2 = 0.10 and 0.17), so verification was carried out with the weighted mean values for the basic parameters, including IL2, as follows:

Table A8-39 Celia Creek Verification Parameters

IL1(mm)	IL2(mm)	PL1	PL2	ke	k1e	k2e
0.0	29.3	0.69	0.63	0.167	1.193	0.468

The results are given in Table A8-40:

Table A8-40 Celia Creek Verification Results

Date	Gauged peak flow (m^3/s)	Predicted peak flow (m^3/s)	Predicted peak /gauged peak	Gauged volume (m^3)	Predicted volume (m^3)	Predicted volume/ gauged volume
13/03/92	20.3	16.8	0.83	719 000	619 900	0.96
25/02/93	18.2	14.9	0.82	1 774 200	1 814 800	1.02
25/02/94	68.9	52.5	0.96	7 419 900	6 304 500	0.85
04/03/94	58.3	30.2	0.52	2 672 700	1 777 000	0.66
30/01/96	18.8	16.7	0.89	1 572 100	2 045 200	1.30
20/01/97	43.6	54.0	1.24	6 018 300	5 021 900	0.83
		mean	0.84			0.92

Burra Creek

Calibration

Six events were chosen for calibration. The events are as follows:

Table A8-41 Burra Creek RRR Calibration Events

Start Date	Start Time	Duration (hrs)	Rainfall (mm)	Runoff (mm)	Peak Flow (m^3/s)
04/06/88	18:00	72	44.2	12.4	25.0
05/07/88	12:00	48	59.3	27.7	87.1
14/03/89	00:00	48	124.4	28.3	57.0
09/04/89	12:00	48	37.2	17.2	27.7
11/06/91	12:00	72	50.6	11.1	17.9
9/01/92	00:00	48	45.6	8.9	24.1
12/04/94	00:00	48	62.3	6.8	20.4

All events displayed reasonable fits. The calibration was initially undertaken on the assumption that two processes were occurring. The events having the largest three rainfalls were also calibrated on the assumption that three processes were occurring. The first of these (05/07/88) gave a much better level of fit, and had more consistent calibrated parameters than the 2 process calibration. For example the two process calibration gave a k_p2 of 0.001, more indicative of a process 3 contribution. The second (14/03/89) had high initial losses for both process 2 and 3 contributions, but all parameters were reasonable. The third (12/04/94) showed a high IL3 (58.8mm) compared with the event rainfall (62.3mm) resulting in little difference in calibrated parameters. It was assumed therefore that for verification the calibration on 3 processes would be used for 05/07/88 and 14/03/89, and the 2 process for 12/04/94.

The results of the calibration is thus:

Table A8-42 Burra Creek RRR Calibration Results

Event date	PL1	IL2 (mm)	PL2	IL3 (mm)	PL3	k_p1	k_p2	k (hrs)	Mean error (m^3/s)
04/06/88	0.86	20.6	0.75	> 44.2	n/a	2.06	0.430	0.201	0.603
05/07/88	0.81	19.3	0.64	47.8	0.59	2.32	0.677	0.083	0.803
14/03/89	0.91	59.6	0.81	105.5	0.74	2.51	0.246	0.226	1.130
09/04/89	0.73	10.1	0.73	> 37.2	n/a	2.06	0.466	0.260	0.829
11/06/91	0.93	18.9	0.76	> 50.6	n/a	3.10	0.551	0.248	0.348
09/01/92	0.94	11.4	0.81	> 45.6	n/a	1.85	0.531	0.284	0.774
12/04/94	0.96	45.9	0.74	> 62.3	n/a	1.04	0.276	0.279	1.203

Verification

Plots of IL2 versus API and initial baseflow for the events show no relationship (r^2 = 0.08 and 0.15). However what is shown is that the event of 14/03/89 has much higher initial loss than can be explained by the API for the event, which is not abnormally low.

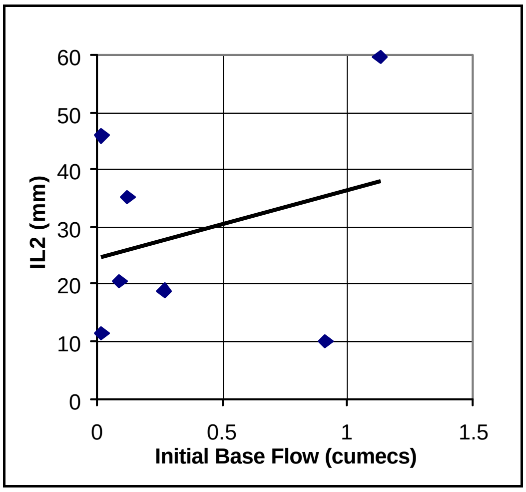


Figure A8-13 Burra Creek IL2 vs Initial Baseflow

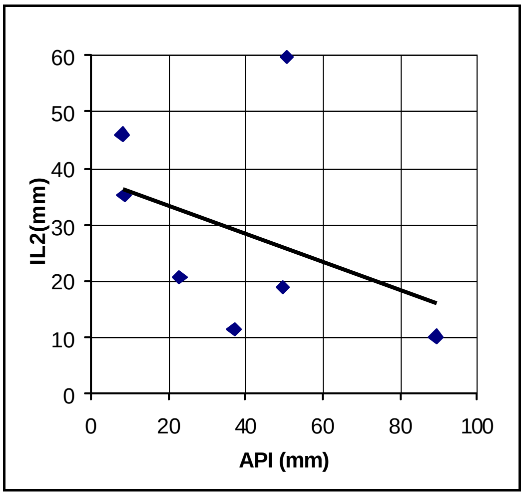


Figure A8-14 Burra Creek IL2 vs API

Verification was therefore carried out with weighted mean parameter values as follows:

Table A8-43 Burra Creek Verification Parameters

IL1(mm)	IL2(mm)	PL1	PL2	k	k_p1	k_p2
0.0	25.2	0.86	0.73	0.191	1.92	0.470

The results are given in Table A8-44. The event of 12/04/89 was a continuation from the calibration event of 9/04/89, so a verification run was also carried out using the fitted parameters from 9/04/89, and using zero loss for the initial for both process 1 and process 2:

Table A8-44 Burra Creek Verification Results

Date	Gauged peak flow (m^3/s)	Predicted peak flow (m^3/s)	Predicted peak /gauged peak	Gauged volume (m^3)	Predicted volume (m^3)	Predicted volume/ gauged volume
26/12/88	48.2	16.4	0.34	1 420 000	700 000	0.49
31/03/89	117.3	79.9	0.68	6 591 000	4 450 000	0.68
12/04/89	15.2	9.4	0.62	1 363 000	462 400	0.34
19/01/95	64.7	128.3	1.98	1 524 000	3 240 000	2.12
27/01/95	51.6	83.5	1.62	1 135 000	1 680 000	1.48
		mean	1.05			1.02

The event of 12/04/89 followed on directly after the event of 9/04/89. As another means of verification, the event of 12/04/89 was modelled using the calibrated parameters for 9/04/89, and assuming no IL2. The results were as follows:

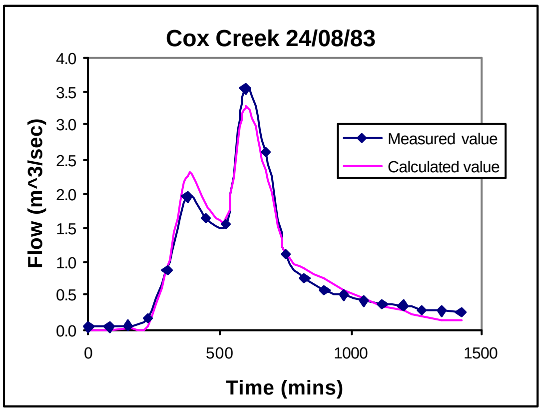
Table A8-45 Burra Creek 12/04/89 - Verification With Parameters of 9/04/89

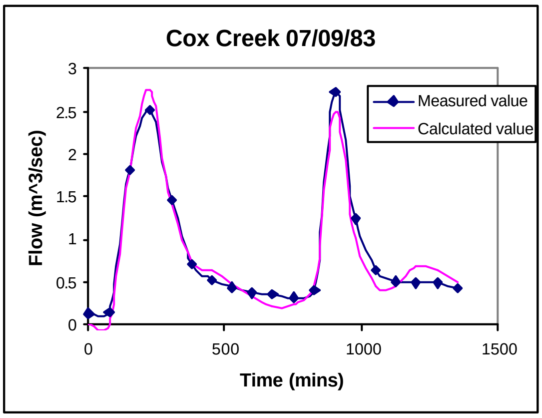
Date	Gauged peak flow (m^3/s)	Predicted peak flow (m^3/s)	Predicted peak /gauged peak	Gauged volume (m^3)	Predicted volume (m^3)	Predicted volume/ gauged volume
12/04/89	15.2	15.6	1.03	1 363 000	1 300 000	0.95

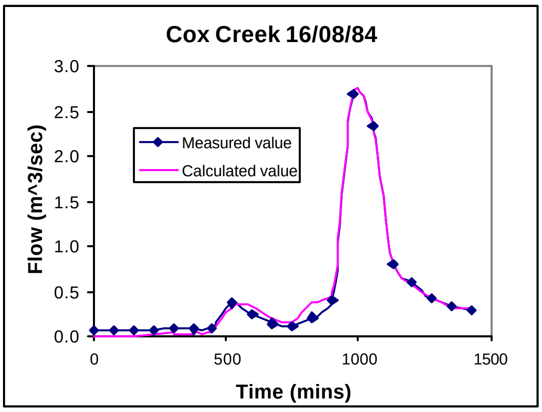
This resulted in a good fit.

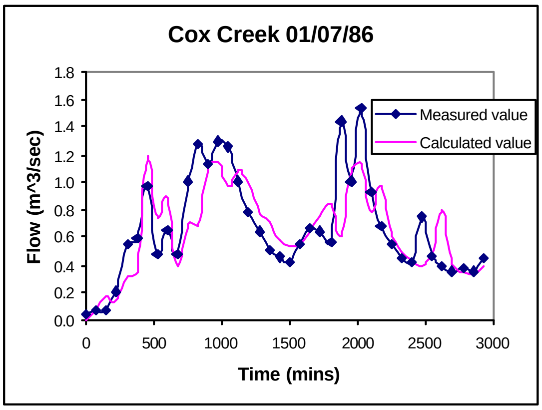
APPENDIX 9 – RRR Model Parameter Correlations

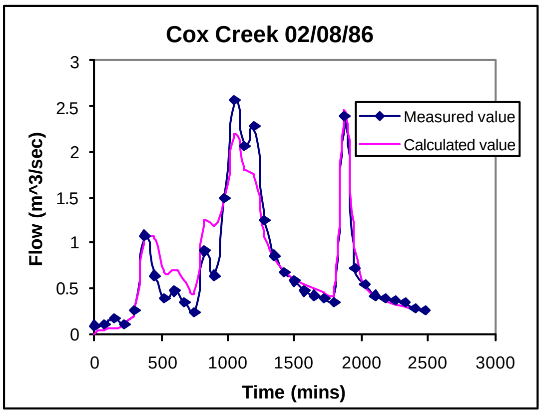
Cox Creek

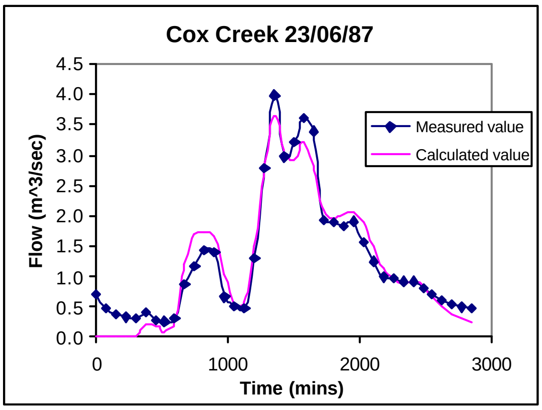

Cox Creek 24/08/83
Flow (m^3/sec)
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
0
500
1000
1500
Time (mins)
Measured value
Calculated value


Cox Creek 07/09/83
Flow (m^3/sec)
3
2.5
2
1.5
1
0.5
0
0
500
1000
1500
Time (mins)
Measured value
Calculated value

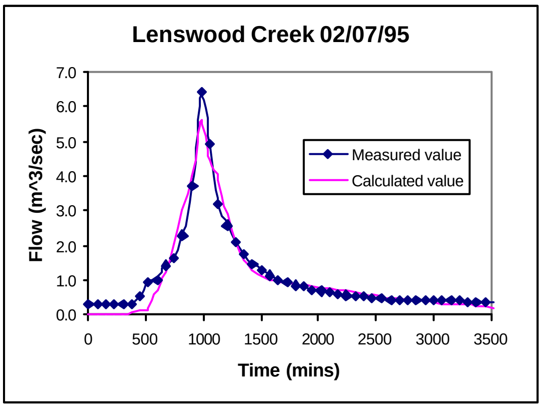

Cox Creek 16/08/84
Flow (m^3/sec)
3.0
2.5
2.0
1.5
1.0
0.5
0.0
0
500
1000
1500
Time (mins)
Measured value
Calculated value

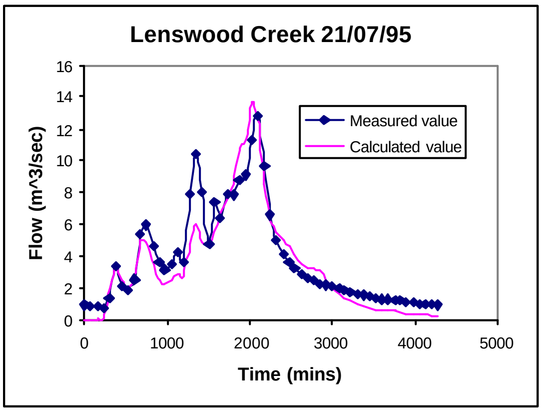

Cox Creek 01/07/86
Flow (m^3/sec)
1.8
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
0
500
1000
1500
2000
2500
3000
Time (mins)
Measured value
Calculated value

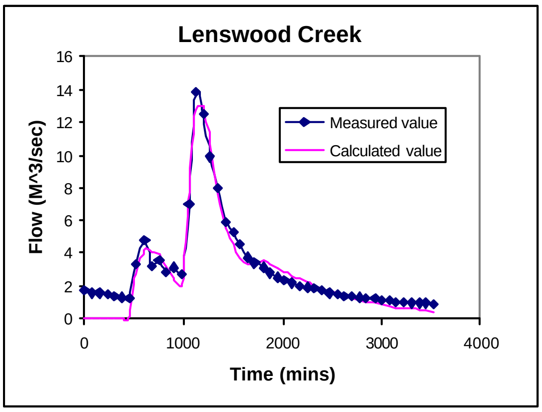

Cox Creek 02/08/86
Flow (m^3/sec)
3
2.5
2
1.5
1
0.5
0
0
500
1000
1500
2000
2500
3000
Time (mins)
Measured value
Calculated value

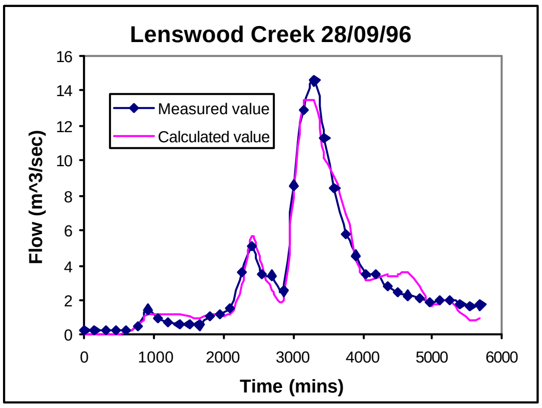

Cox Creek 23/06/87
Flow (m^3/sec)
4.5
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
0
1000
2000
3000
Time (mins)
Measured value
Calculated value

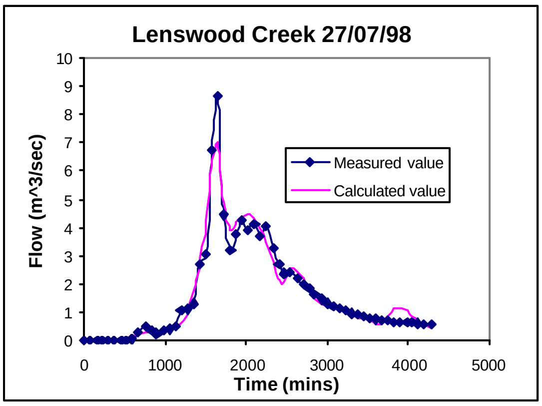
Lenswood Creek

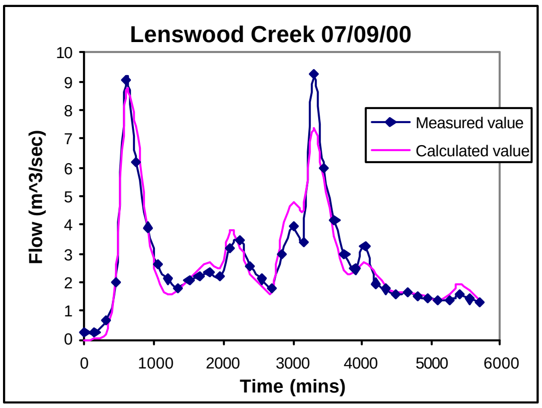




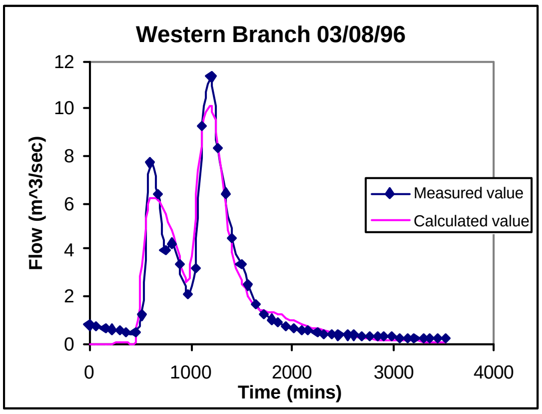


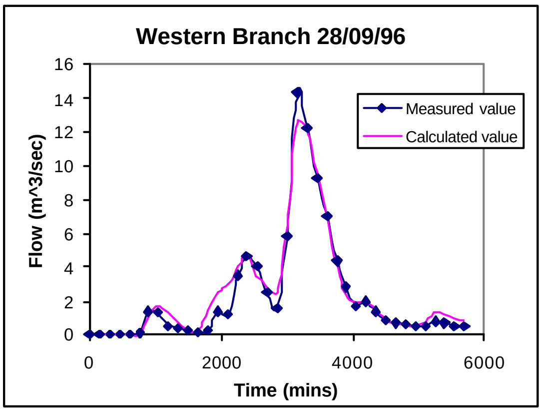


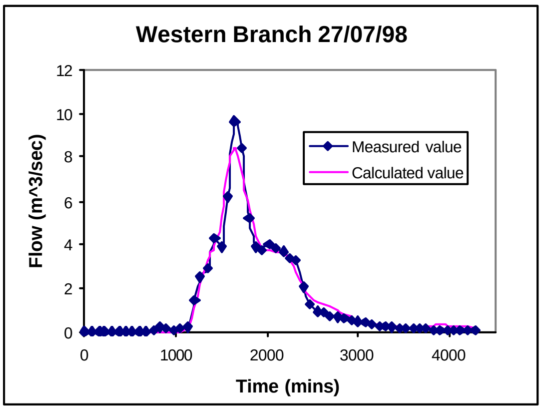


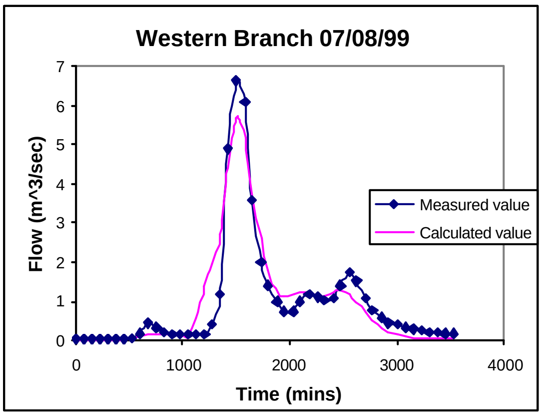


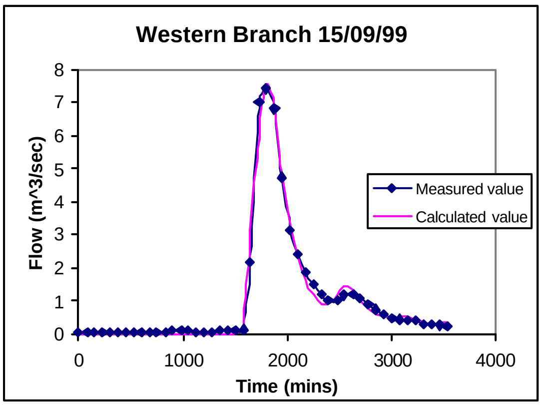
Western Branch

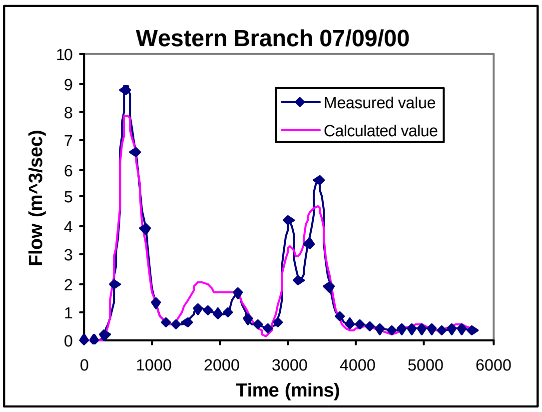




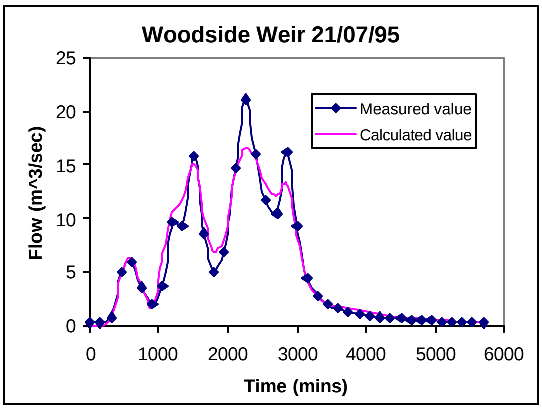


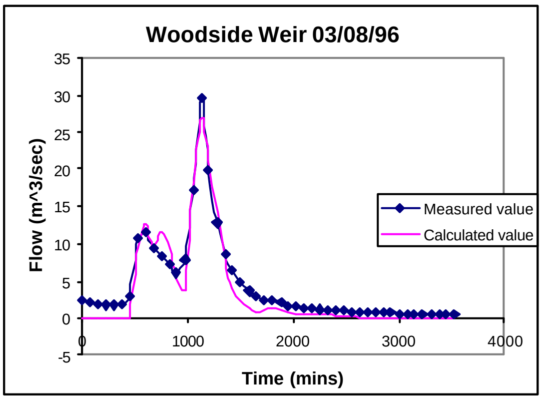


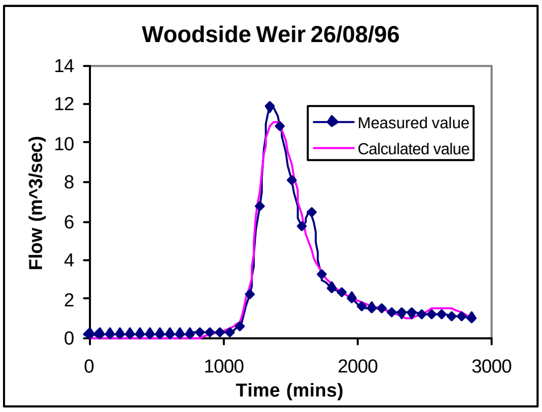


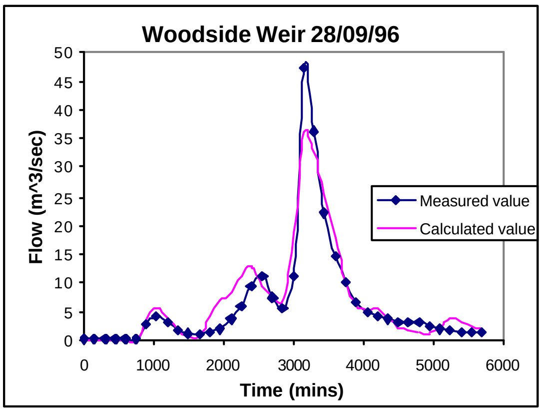


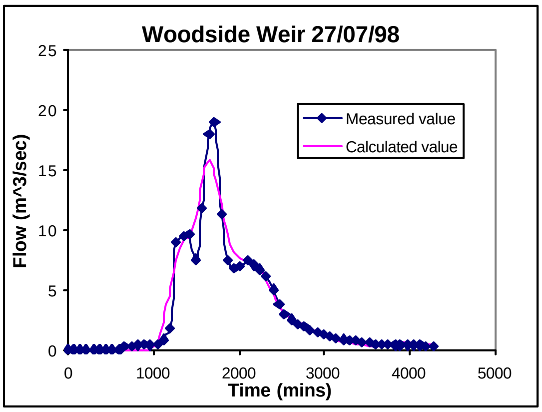
Woodside Weir

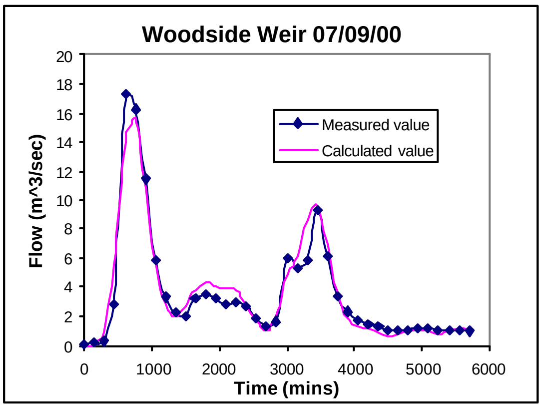




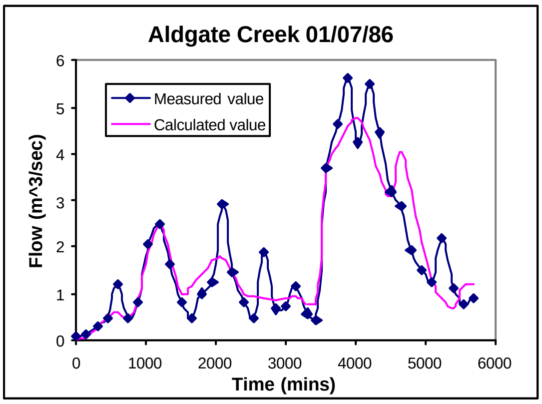


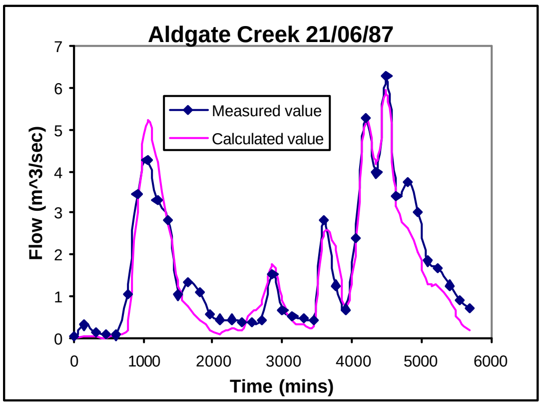


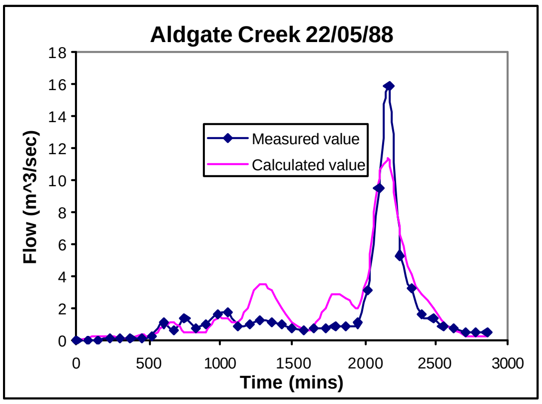


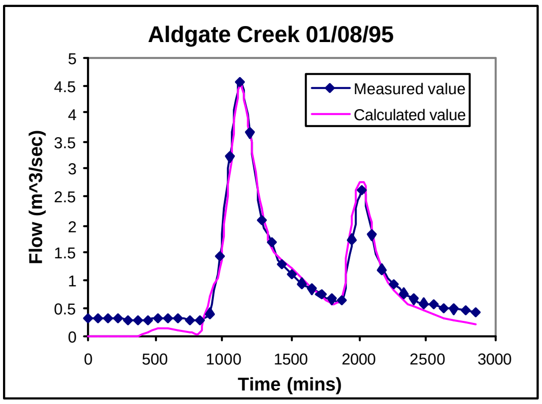


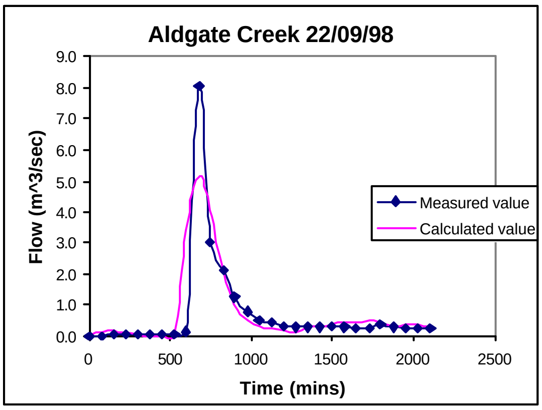
Aldgate Creek

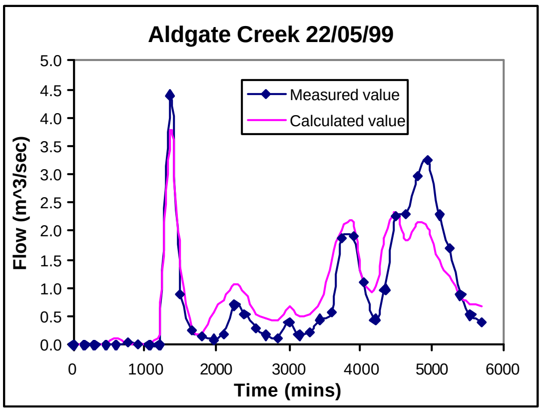


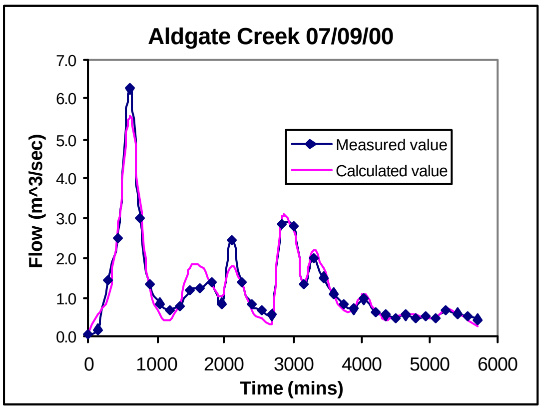




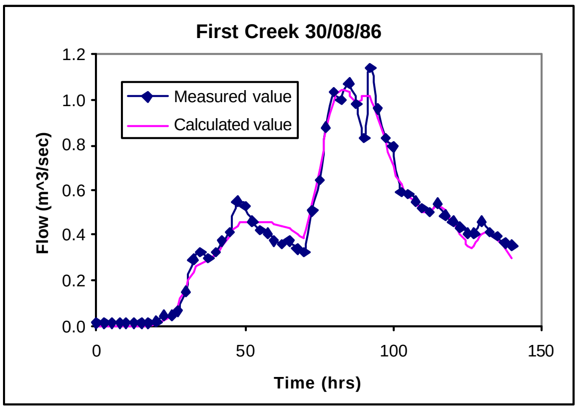


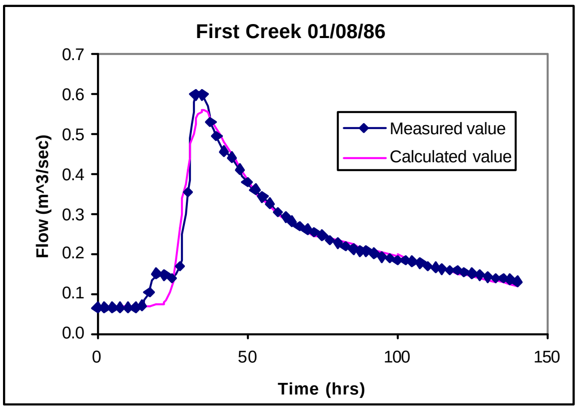


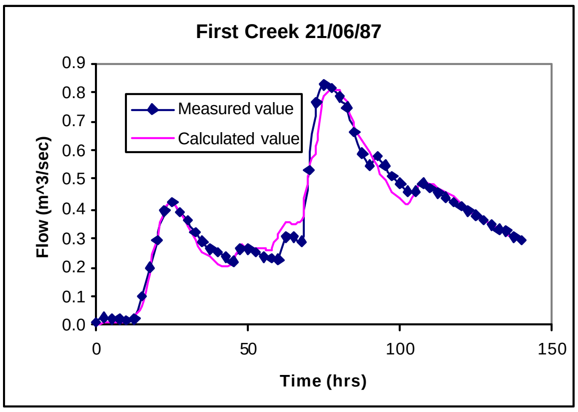


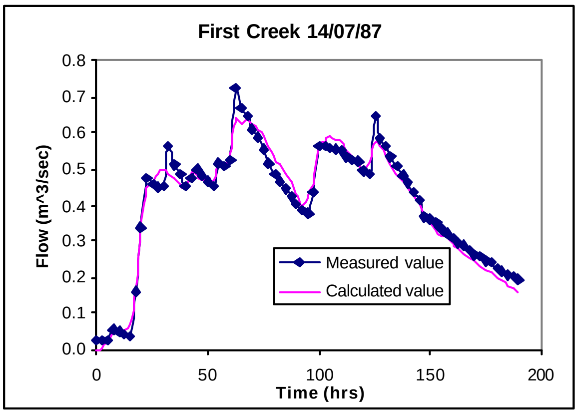

Aldgate Creek 07/09/00
Flow (m^3/sec)
Time (mins)
Measured value
Calculated value
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
0
1000
2000
3000
4000
5000
6000

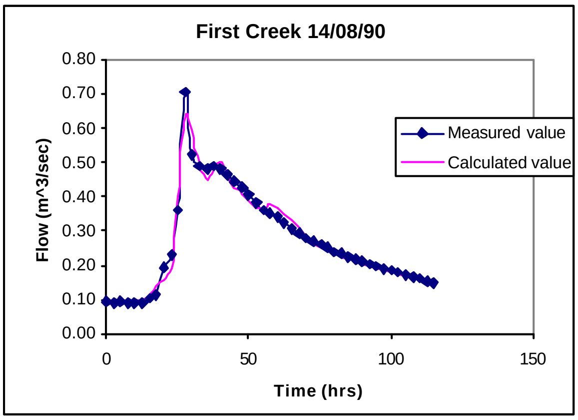
First Creek

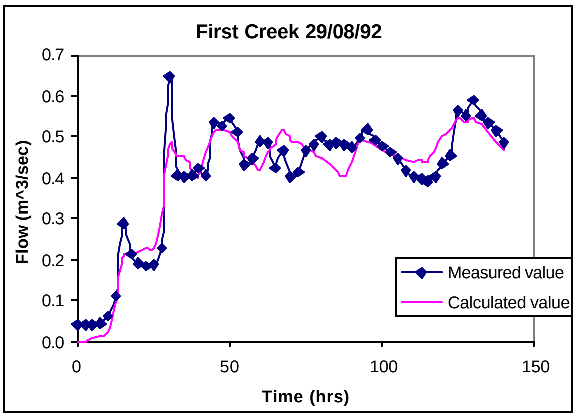

First Creek 30/08/86
Flow (m^3/sec)
Time (hrs)
Measured value
Calculated value
0.0
0.2
0.4
0.6
0.8
1.0
1.2
0
50
100
150

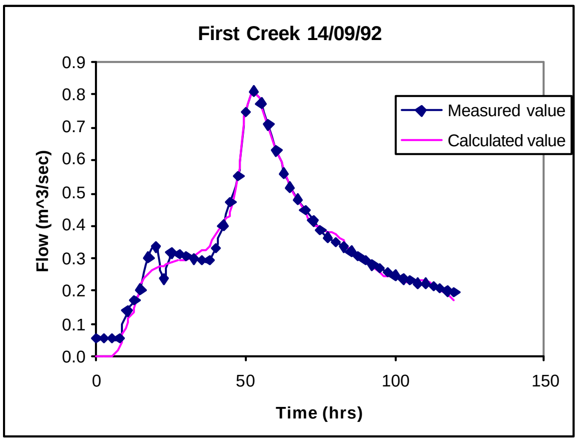

First Creek 01/08/86
Flow (m^3/sec)
Time (hrs)
Measured value
Calculated value
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0
50
100
150


First Creek 21/06/87
Flow (m^3/sec)
Time (hrs)
Measured value
Calculated value
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
0
50
100
150

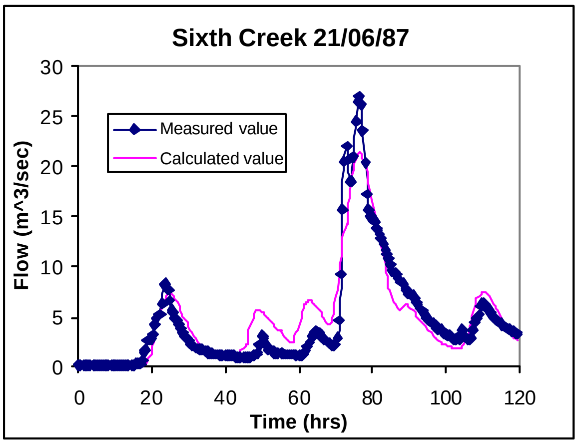

First Creek 14/07/87
Flow (m^3/sec)
Time (hrs)
Measured value
Calculated value
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0
50
100
150
200

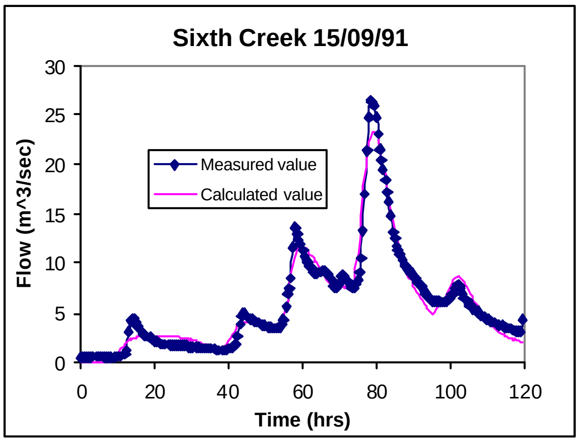

First Creek 14/08/90
Flow (m^3/sec)
Time (hrs)
Measured value
Calculated value

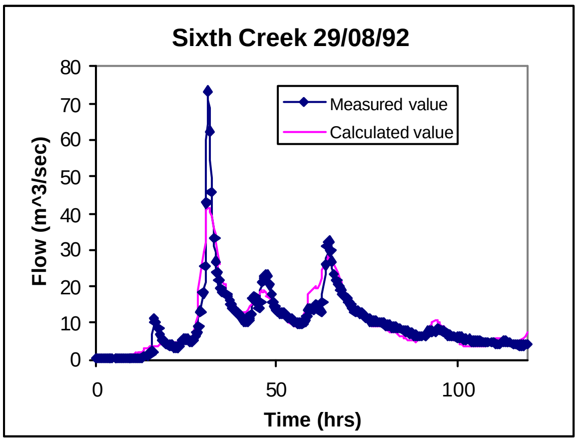

First Creek 29/08/92
Flow (m^3/sec)
Time (hrs)
Measured value
Calculated value

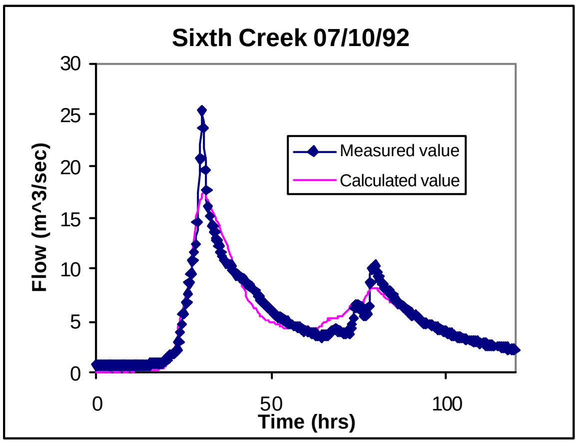

First Creek 14/09/92
Flow (m^3/sec)
Time (hrs)
Measured value
Calculated value

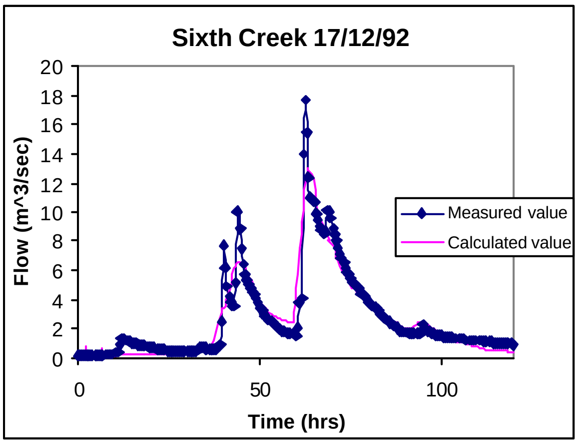
Sixth Creek

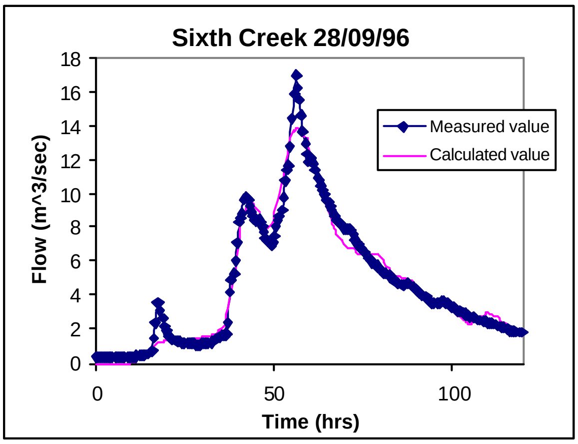

Sixth Creek 21/06/87
Flow (m^3/sec)
Time (hrs)
Measured value
Calculated value


Sixth Creek 15/09/91
Flow (m^3/sec)
Time (hrs)
Measured value
Calculated value


Sixth Creek 29/08/92
Flow (m^3/sec)
Time (hrs)
Measured value
Calculated value
0
10
20
30
40
50
60
70
80
0
50
100

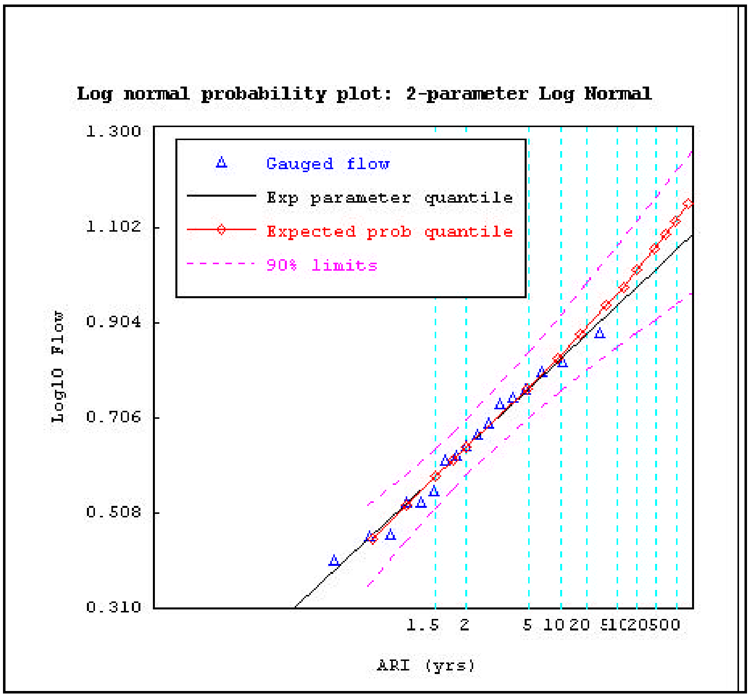

Sixth Creek 07/10/92
Flow (m^3/sec)
Time (hrs)
Measured value
Calculated value
0
5
10
15
20
25
30
0
50
100


Sixth Creek 17/12/92
Flow (m^3/sec)
Time (hrs)
Measured value
Calculated value
0
2
4
6
8
10
12
14
16
18
20
0
50
100

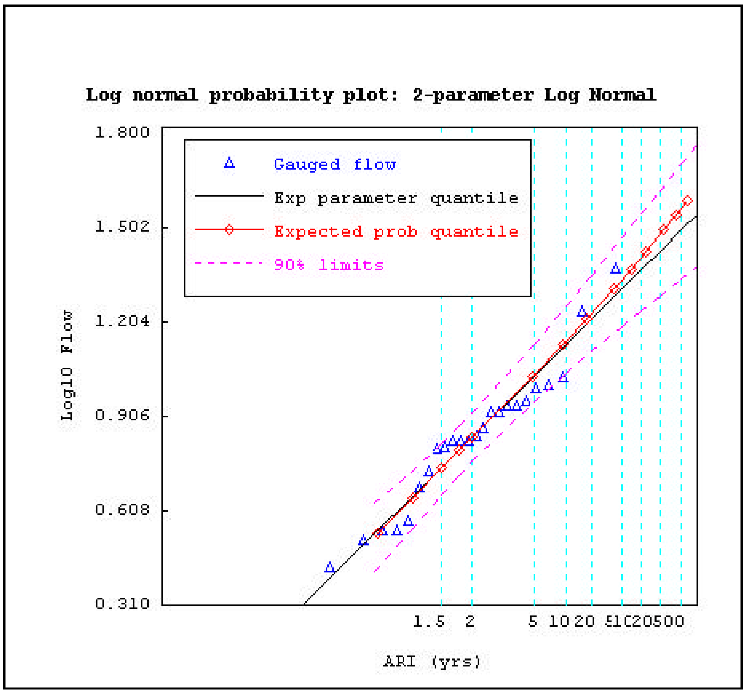

Sixth Creek 28/09/96
Flow (m^3/sec)
Time (hrs)
Measured value
Calculated value
0
2
4
6
8
10
12
14
16
18
0
50
100

FLOOD FREQUENCY ANALYSIS DISTRIBUTIONS

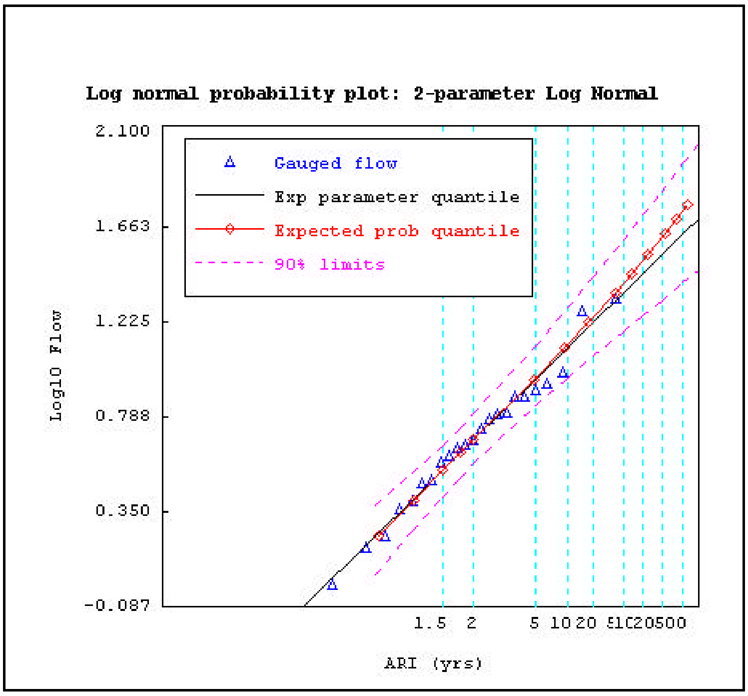
Cox Creek



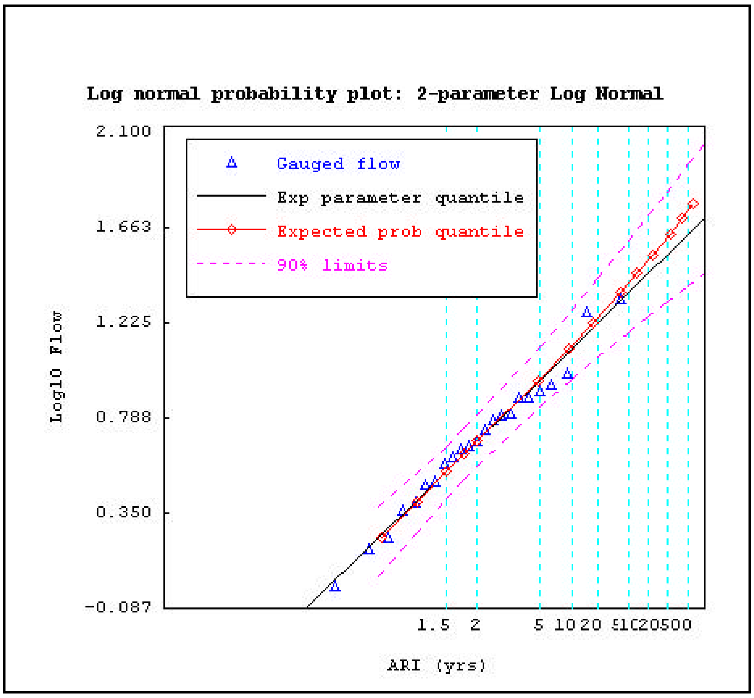
Aldgate Creek



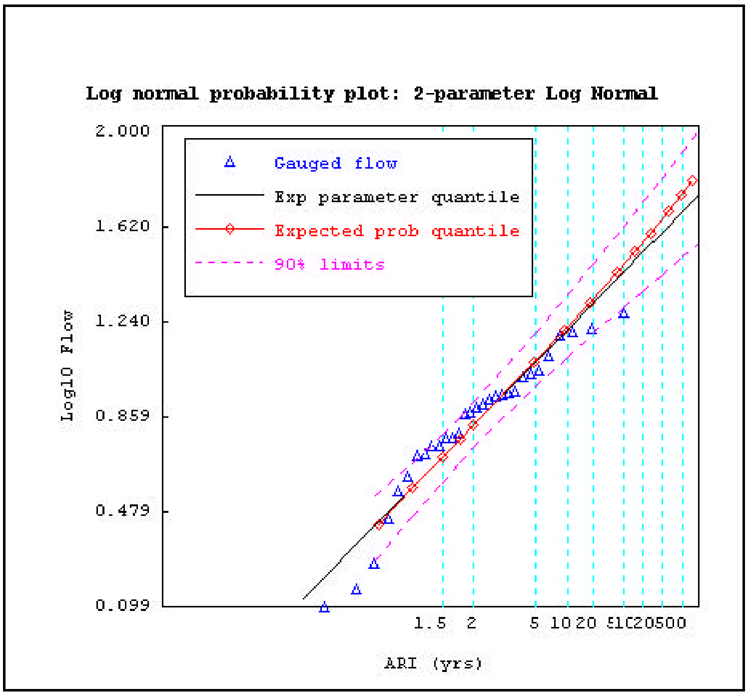
Inverbrackie Creek



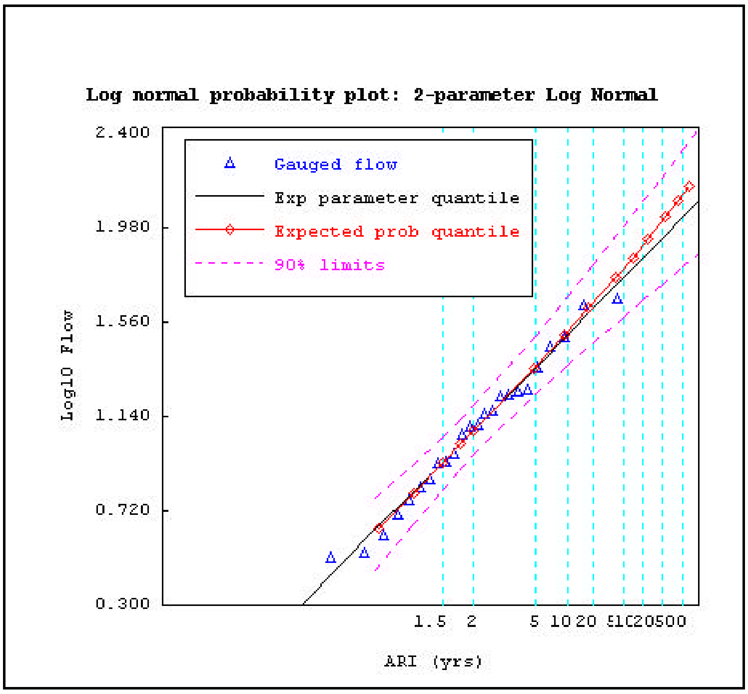
Lenswood Creek



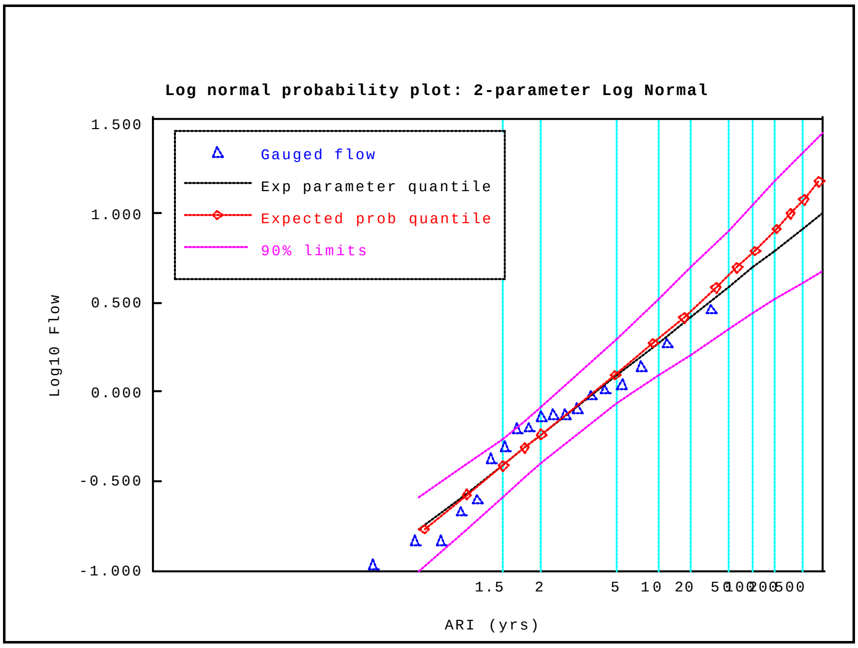
Scott Creek



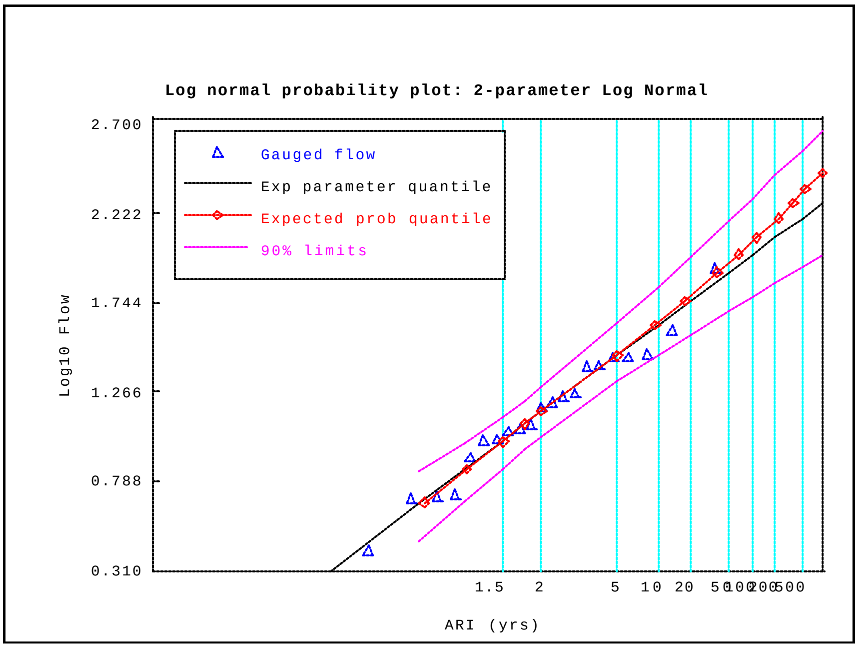
Echunga Creek



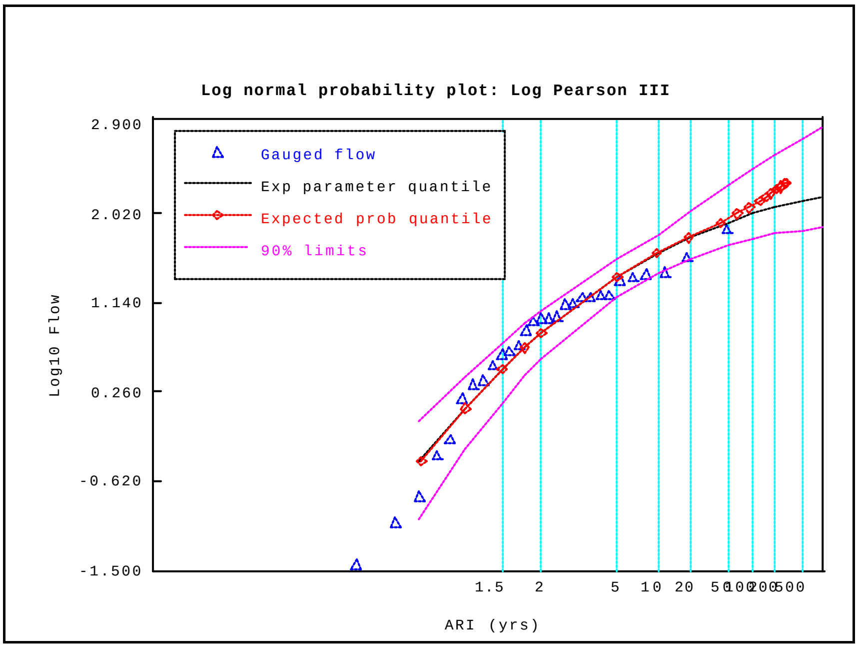
First Creek



Sixth Creek

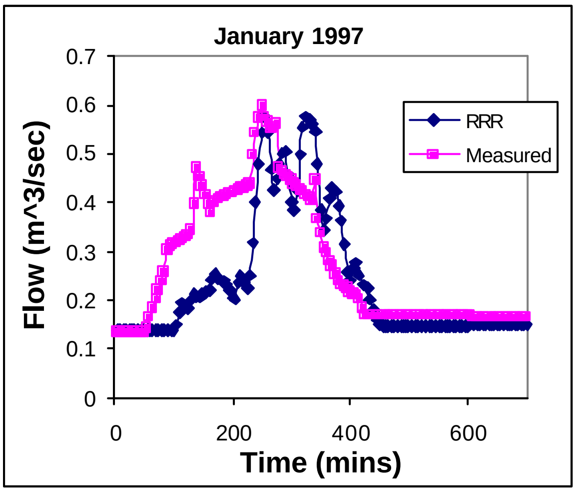


Torrens River

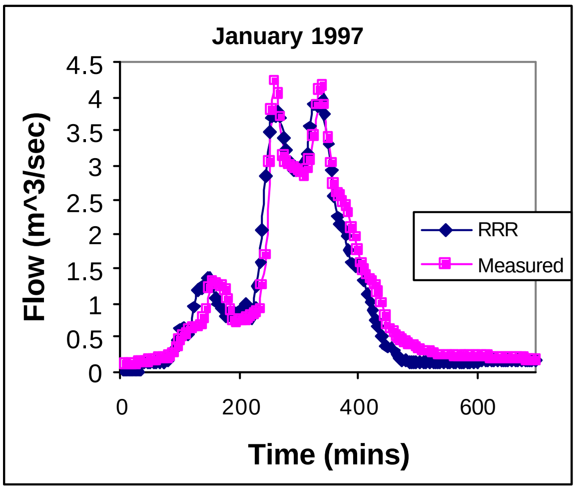


APPENDIX 10 – Keswick and Brownhill Creek

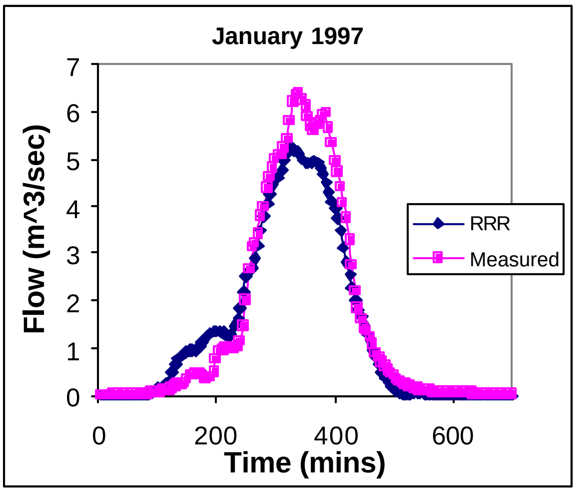
Keswick Creek



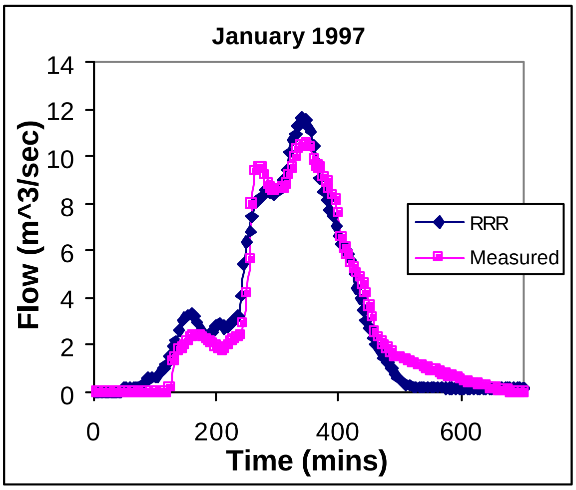
Ridge Park



Charles Street

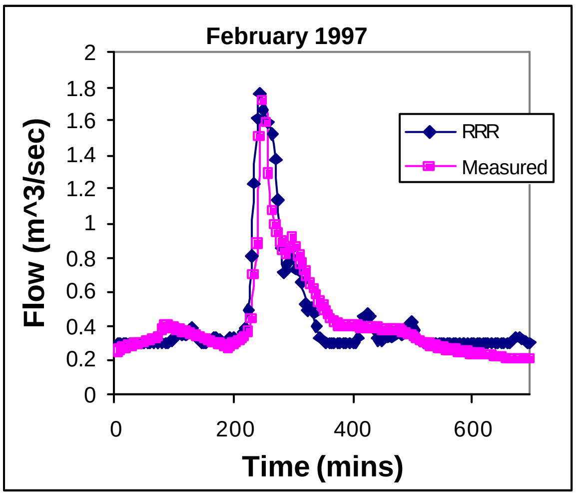


Roberts Street

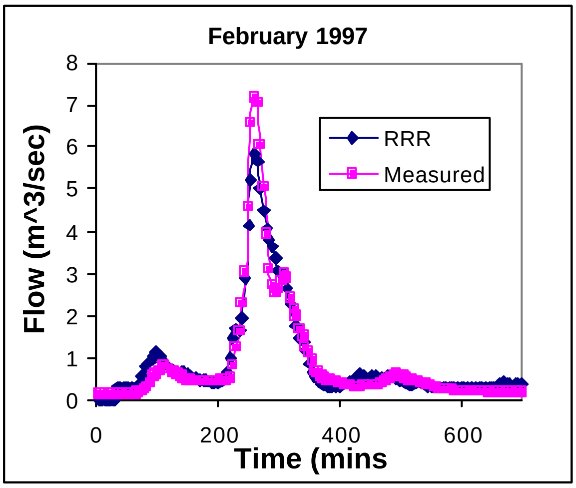


Keswick

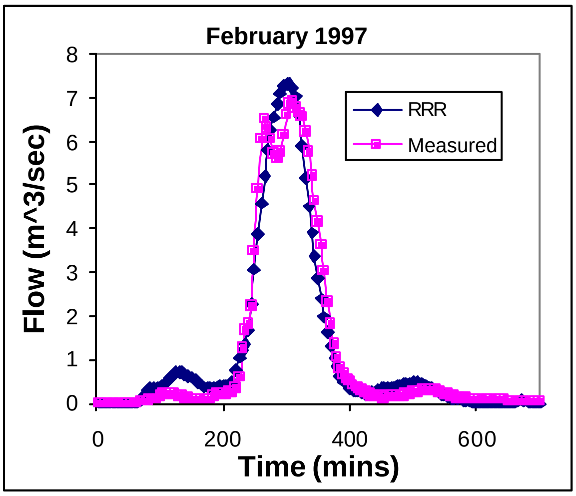
January 1997



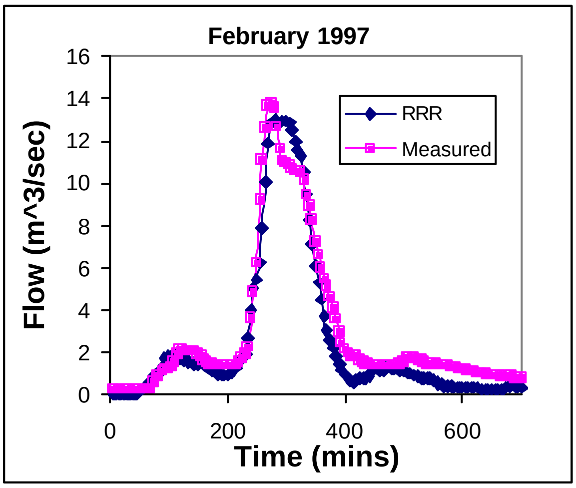
Ridge Park



Charles Street

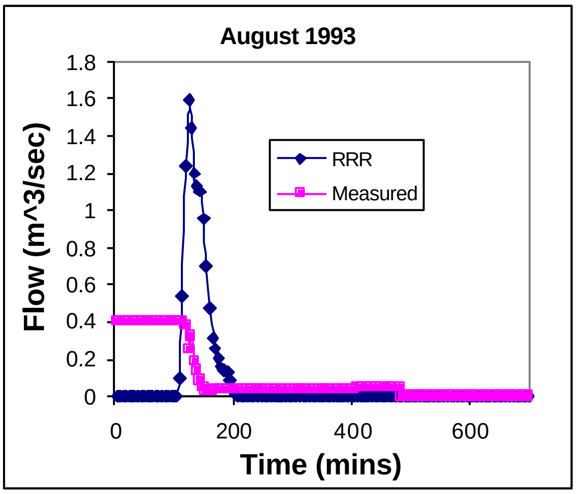


Roberts Street

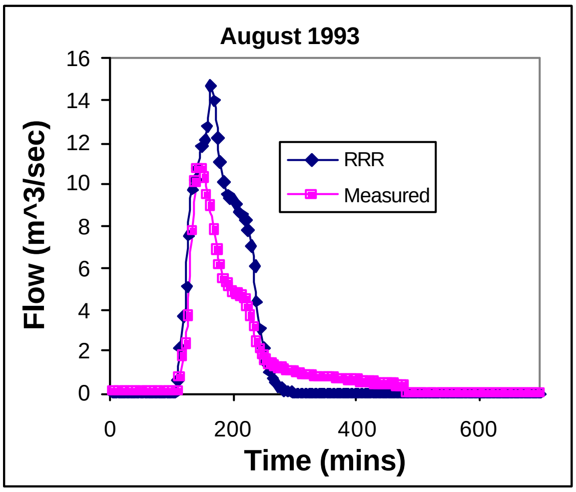


Keswick

February 1997

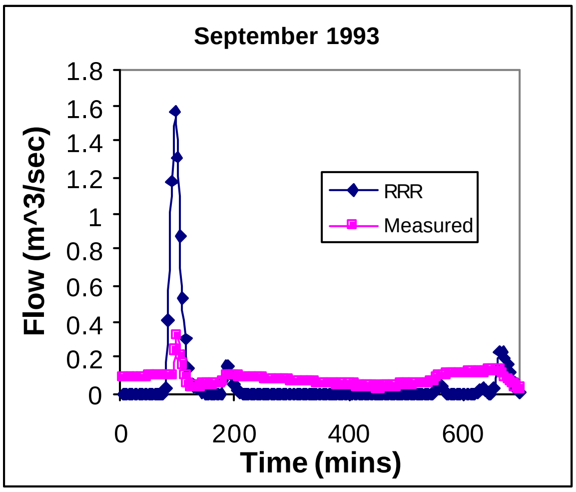


Ridge Park

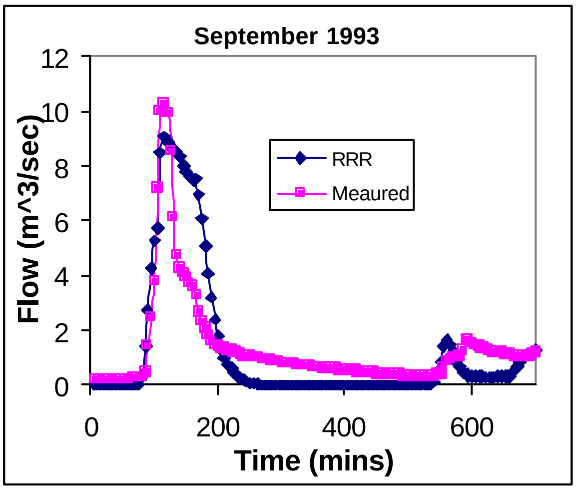


Keswick

August 1993

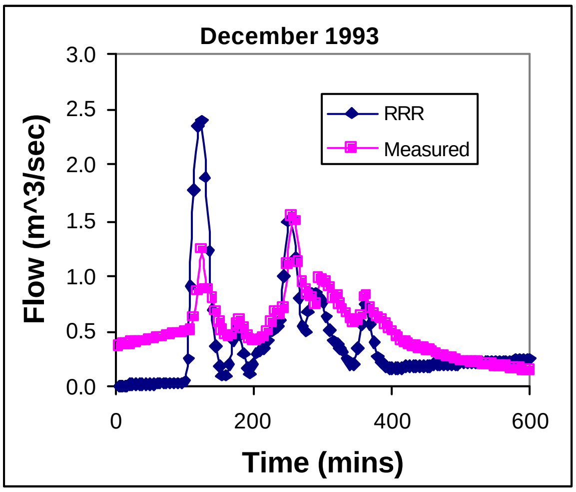


Ridge Park

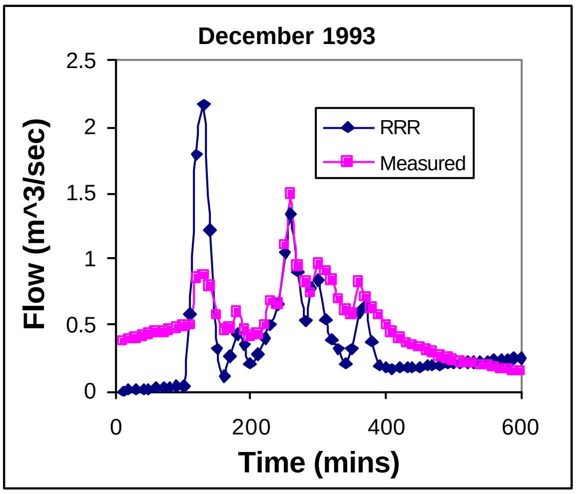


Keswick

September 1993

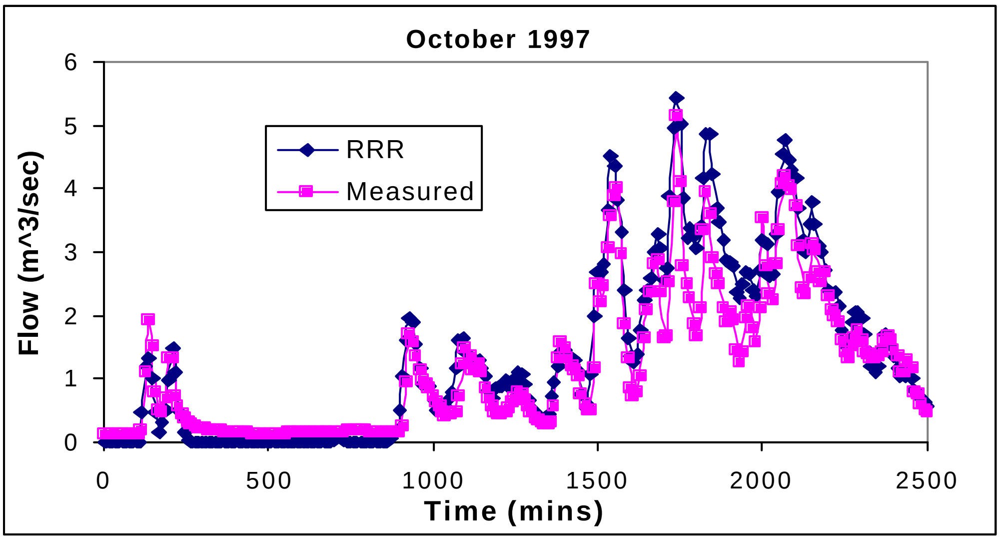


Ridge Park

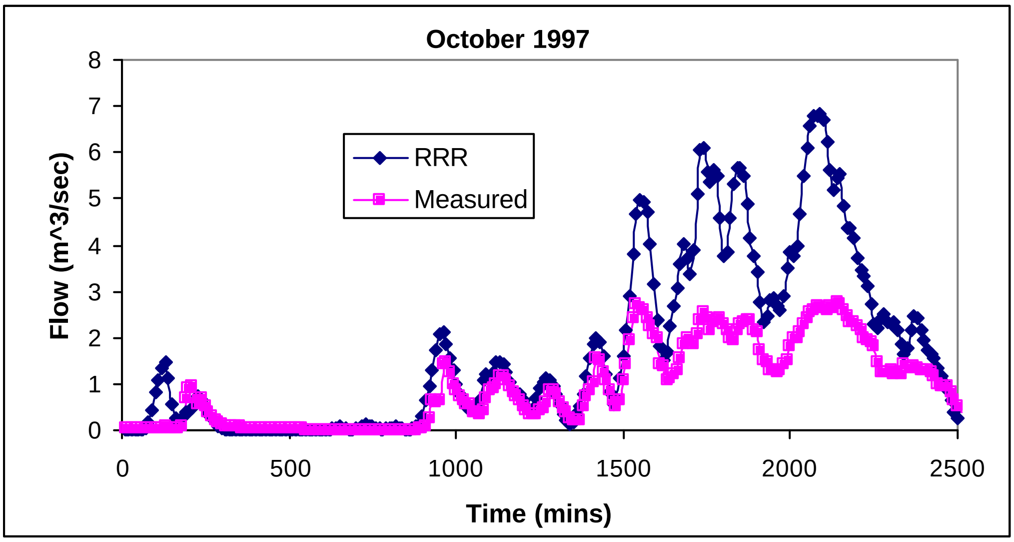


Keswick

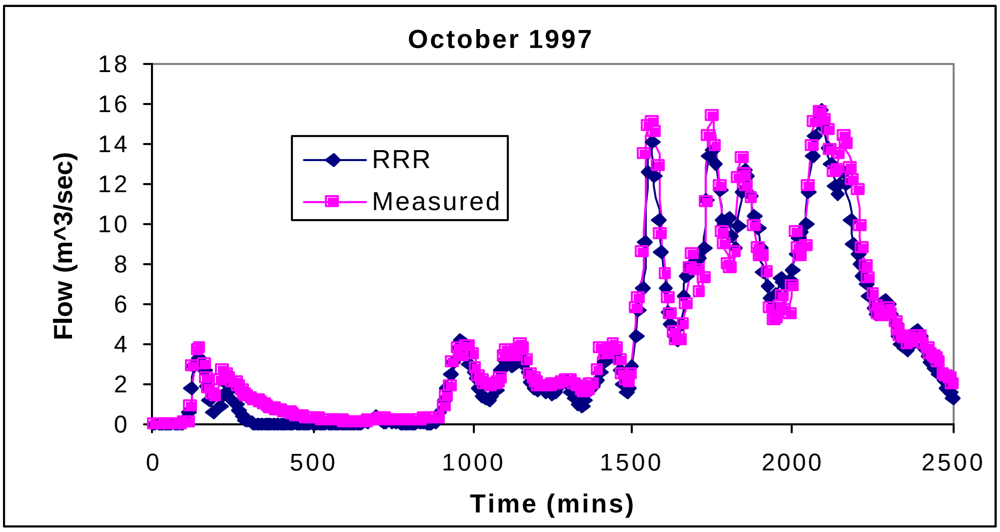
December 1993



Charles Street – October 1997

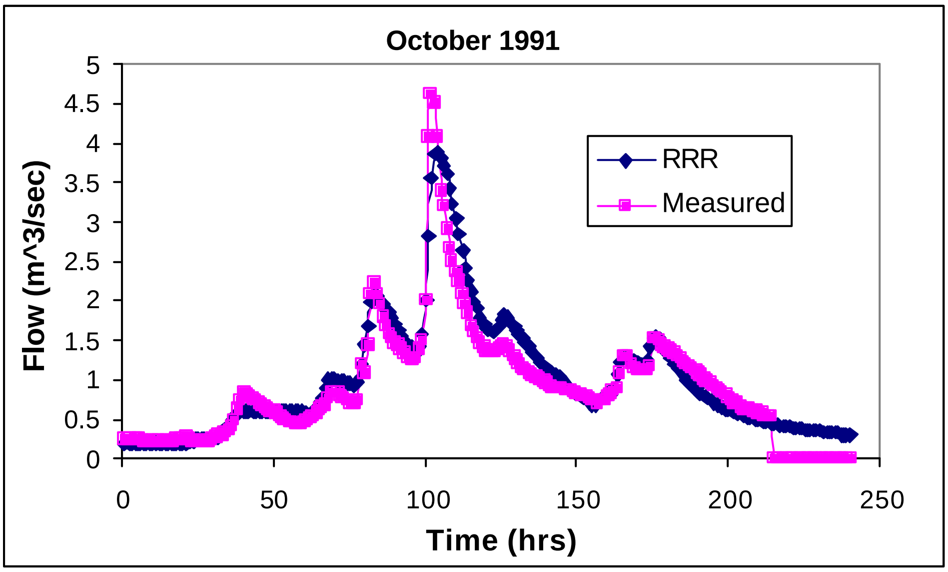


Roberts Street – October 1997

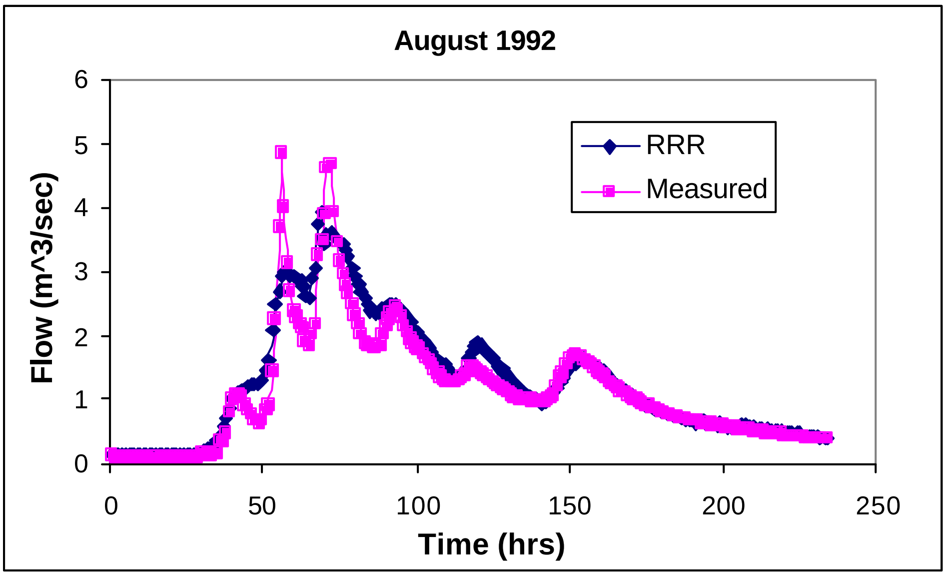


Keswick – October 1997

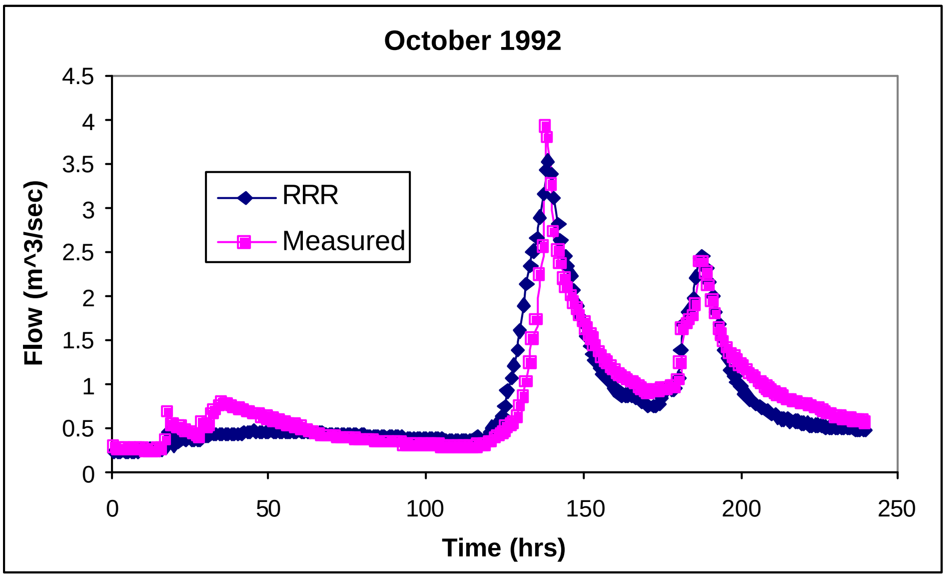
Brownhill Creek



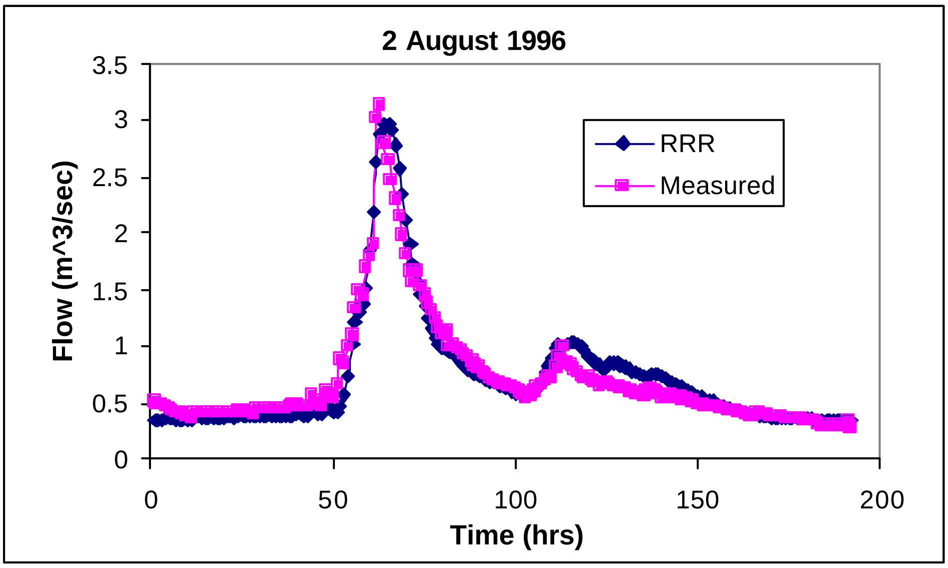
Scotch College September 1991



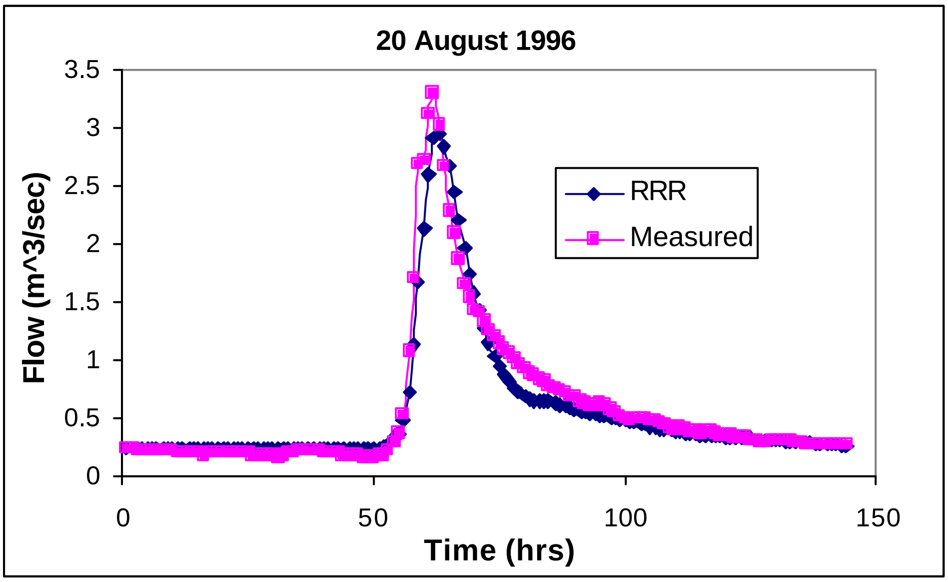
Scotch College, August 1992



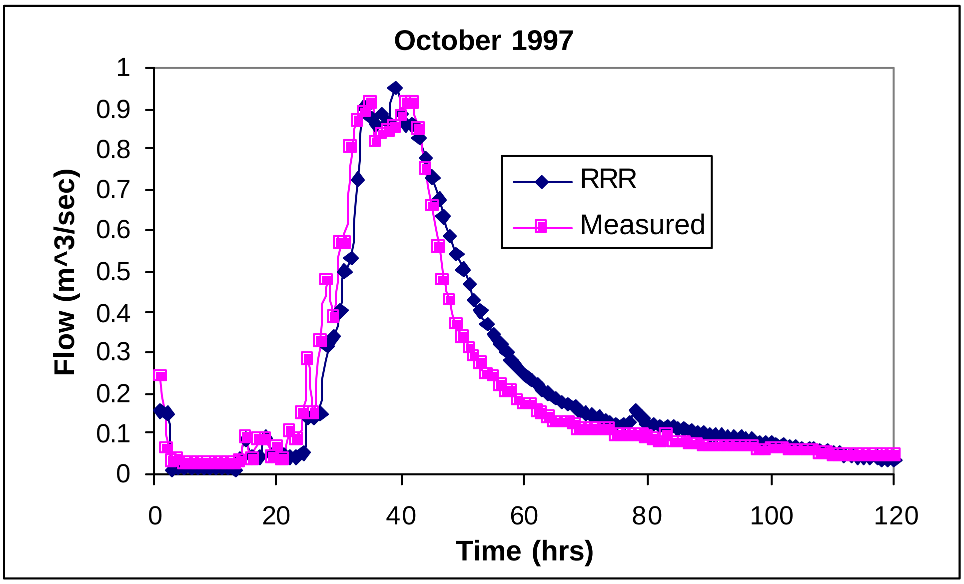
Scotch College, October 1992



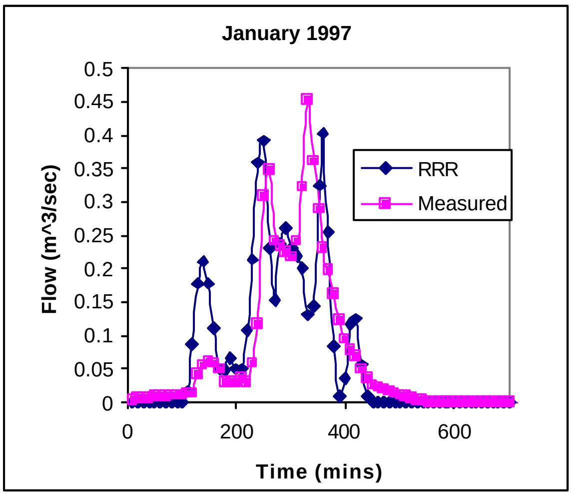
Scotch College August 1996



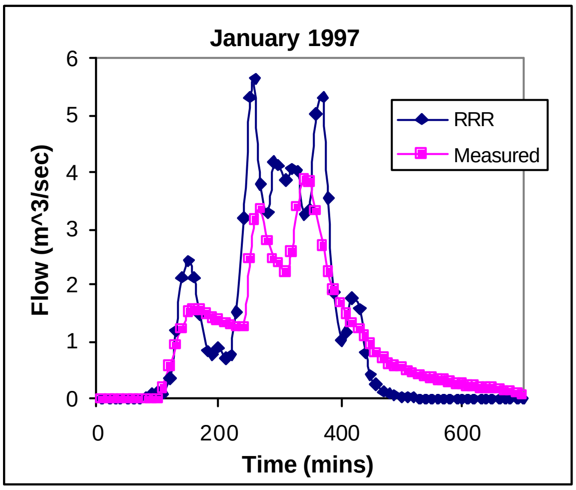
Scotch College August 1996



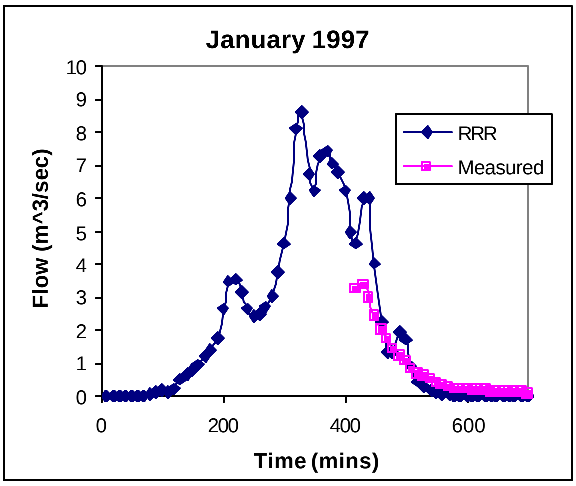
Scotch College October 1997



Scotch College

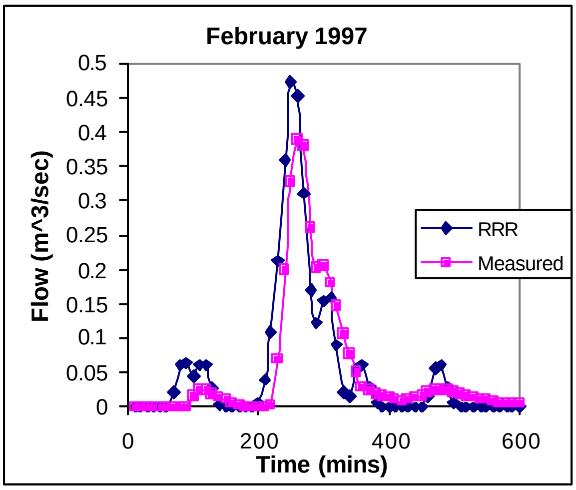


Hawthorn

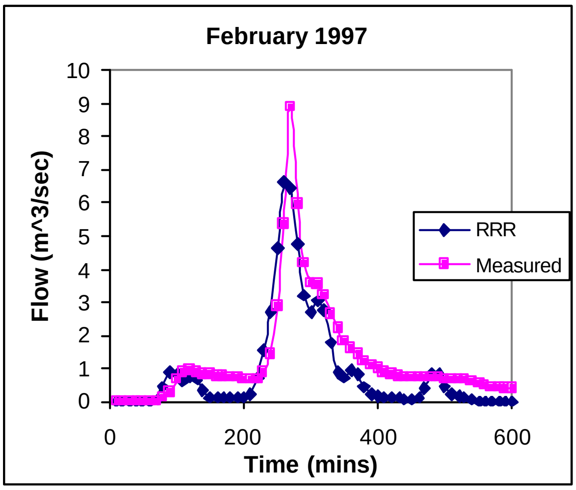


Keswick Ck Junction (part record missing)

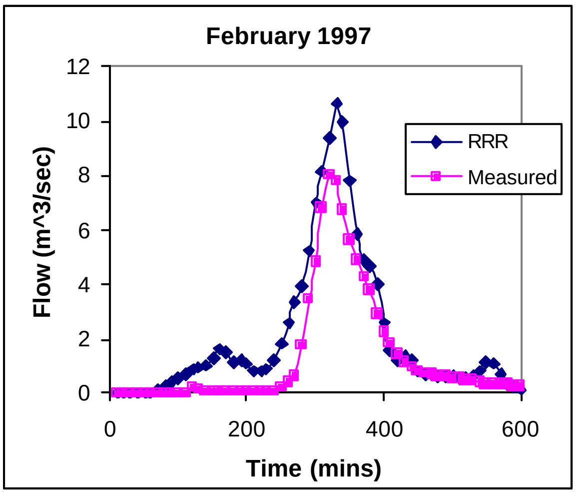
January 1997



Scotch College

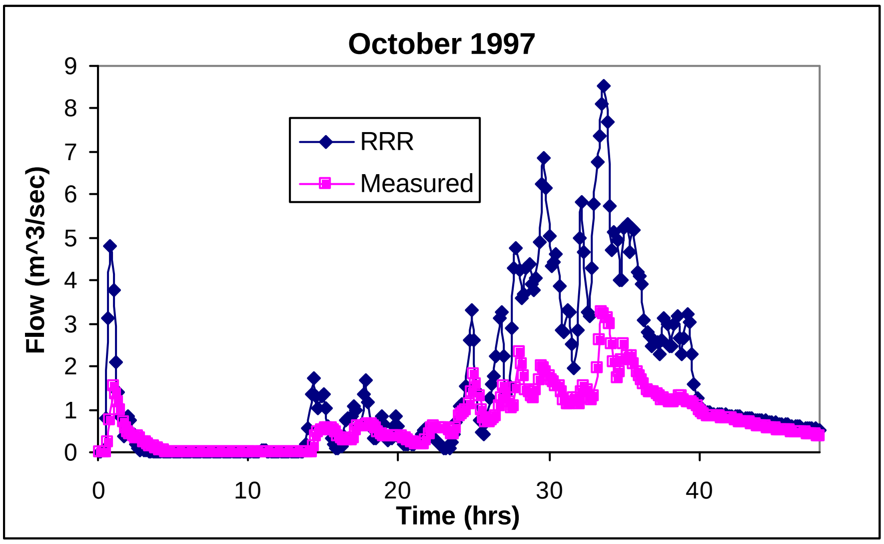


Hawthorn

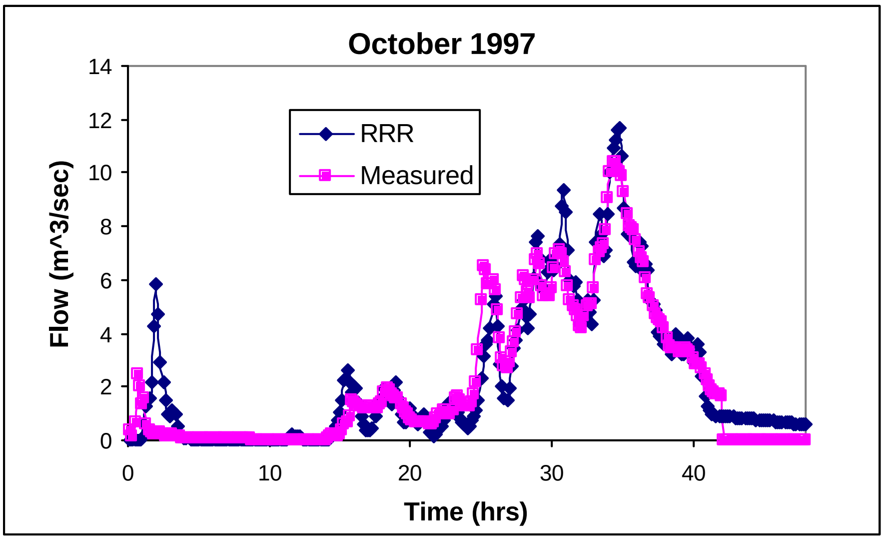


Keswick Creek Junction

February 1997



Hawthorn



Keswick Creek Junction

October 1997

APPENDIX 11 – PAPERS PUBLISHED RELATING TO THESIS

KEMP, D.J. "Calibration of the ILSAX and RAFTS Models on Two Urban Catchments in Adelaide, South Australia" I.E.Aust Water Down Under '94, Adelaide, November 1994 NCP No. 94/15 pp461-464.

KEMP, D.J. and DANIELL, T.M. "Towards Simple and Rational Urban Hydrology Modelling - A New Approach" I.E. Aust. Second International Symposium on Urban Stormwater Management, Melbourne, July 1995, NCP No. 95/03, pp195-200.

KEMP, D.J. and DANIELL, T.M. "A Proposal for a Rainfall - Runoff - Routing (RRR) Model" I.E.Aust Hydrology and Water Resources Symposium, Hobart, May 1996 NCP No. 96/05, pp15-20.

DANIELL, T.M, KEMP, D.J and DICKENS, J. "The Olary Floods February 1997 - Implications for South Australia" ANCOLD/NZSOLD Conference on Dams, August 1998.

KEMP, D.J. "Flood Hydrology Modelling of Keswick Creek using the RRR Model" Hydrastorm 98, 3rd International Symposium on Stormwater Management, Adelaide, September 1998 pp349-354.

KEMP, D.J. "The Old and the New – A Comparison of the Performance of the RORB, KINDOG and RRR Models on a Small Rural Catchment" Hydro 2000, I.E.Aust Hydrology and Water Resources Symposium, Perth, November 2000.

Kemp, D.J. (1994) Calibration of the ILSAX and RAFTS models on two urban catchments in Adelaide South Australia.
In: Water Down Under '94 Conference, Barton, ACT, pp. 461-464

NOTE:
This publication is included on pages 439-442 in the print copy of the thesis held in the University of Adelaide Library.

Kemp, D.J. & Daniell, T.M. (1995) Towards simple and rational urban hydrology modelling - a new approach.
Second International Symposium on Urban Stormwater Management, Melbourne, July 1995, pp. 195-200

NOTE:
This publication is included on pages 443-448 in the print copy of the thesis held in the University of Adelaide Library.

Kemp, D.J. & Daniell, T.M. (1996) A proposal for a rainfall - runoff - routing (RRR) model.
Hydrology and Water Resources Symposium, Hobart, May 1996, pp. 15-20

NOTE:
This publication is included on pages 449-455 in the print copy
of the thesis held in the University of Adelaide Library.

Daniell, T.M., Kemp, D.J. & Dickens, J. (1998) The Olary floods February 1997 - Implications for South Australia.
ANCOLD/NZSOLD Conference on Dams, August 1998.

NOTE:
This publication is included on pages 456-466 in the print copy of the thesis held in the University of Adelaide Library.

Kemp. D.J. (1998) Flood hydrology modelling of Keswick Creek using the RRR model. *Hydrastorm 98, Third International Symposium on Stormwater Management, Adelaide, September 1998, pp. 349-354*

NOTE:
This publication is included on pages 467-472 in the print copy of the thesis held in the University of Adelaide Library.

Kemp. D.J. (2000) The old and the new - acomparison of the performance of the RORB, KINDOG and RRR models on a small rural catchment.
Hydro 2000, Hydrology and Water Resources Symposium, Perth, November 2000, pp. 1036-1041

NOTE:
This publication is included on pages 473-478 in the print copy of the thesis held in the University of Adelaide Library.