MODELLING OF URBAN STORMWATER DRAINAGE SYSTEMS USING ILSAX

BY

SUNIL THOSAINGE DAYARATNE

B.Eng. (Peradeniya), M.Eng. (Roorkee), M.Eng. (Melbourne)

CPEng., MIEAust., C.Eng., MIE (SL), MAWWA, LMIAH, LMSLAAS

THESIS SUBMITTED IN FULFILMENT OF THE REQUIREMENT FOR THE DEGREE OF DOCTOR OF PHILOSOPHY

SCHOOL OF THE BUILT ENVIRONMENT

VICTORIA UNIVERSITY OF TECHNOLOGY, AUSTRALIA

AUGUST 2000

To:

Ranjani, Kushan & Maduwanthi

# ABSTRACT

Over the last few decades, the world has witnessed rapid urbanisation. One of the many complex problems resulting from increased urbanisation is related to management of stormwater from developed areas. If stormwater is not managed properly, it may lead to flooding of urban areas, and deterioration of water quality in rivers and receiving waters. Urban drainage systems are used to manage urban stormwater.

For design of effective and economic urban drainage systems, it is important to estimate the design flows accurately. Many computer based mathematical models have been developed to study catchment runoff (or flows) in urban environments. These models may be used in different stages of the projects such as screening, planning, design and operation. Each stage may require a different model, although some models can be used for several of these stages.

A customer survey was conducted in May 1997 to study the current practice in Victoria (Australia) on stormwater drainage design and analysis, as part of this thesis. The survey was restricted to city/shire councils and consultants, who are engaged in design and analysis of urban drainage systems. The results of the survey showed that 95% of respondents used the Statistical Rational method. Also, it was revealed that most respondents were reluctant to use stormwater drainage computer models, since there were no adequate guidelines and information available to use them especially for ungauged catchments. According to 5% of the respondents, who used models, ILSAX was the most widely used stormwater drainage computer model in Victoria. The 1987 edition of the Australian Rainfall-Runoff (ARR87) suggests the ILSAX model as one of the computer models that can be used for stormwater drainage design and analysis. Due to these reasons, the ILSAX model was used in this study in an attempt to produce further guidance to users in development and calibration of ILSAX models of urban drainage systems.

In order to use the ILSAX model, it is necessary to estimate the model parameters for catchments under consideration. The model parameters include loss model parameters (i.e.

infiltration and depression storage parameters) and other parameters related to the catchment (such as percent imperviousness, soil cover and conveyance system parameters). Some of these parameters can be estimated from available maps and drawings of the catchment. The ideal method to determine these parameters (which cannot be reliably determined from available maps and drawings) is through calibration of these models using observed rainfall and runoff data. However, only few urban catchments are monitored for rainfall and runoff, and therefore calibration can be done only for these catchments. At present, there are no clear guidelines to estimate the model parameters for ungauged catchments where no rainfall-runoff data are available. In this PhD project, first the ILSAX model was calibrated for some gauged urban catchments. From the results of calibration of these catchments, regression equations were developed to estimate some model parameters for use in gauged and ungauged urban stormwater catchments.

Before calibrating the ILSAX model for gauged catchments, a detailed study was conducted to;

- select the most appropriate modelling option (out of many available in ILSAX) for modelling various urban drainage processes,
- study the sensitivity of model parameters on simulated storm hydrographs, and
- study the effect of catchment subdivision on storm hydrographs.

This detailed study was conducted using two typical urban catchments (i.e. one 'small' and one 'large') in Melbourne metropolitan area (Victoria) considering four design storms of different average recurrence intervals (ARI). Three storms with ARI of 1, 10 and 100 years, and one with ARI greater than 100 years were considered in the study. The results obtained from this detailed study were subsequently used in model calibration of the study catchments. The results showed that the runoff volume of 'large' storm events was more sensitive to the antecedent moisture condition and the soil curve number (which determines soil infiltration) and less sensitive to the pervious and impervious area depression storages. However, for 'small' storm events, the runoff volume was sensitive to the impervious area depression storage. The peak discharge was sensitive to pipe roughness, pit choke factor, pit capacity parameters and gutter characteristics for both 'small' and 'large' storm events.

The results also showed that the storm hydrograph was sensitive to the catchment subdivision.

The accuracy of rainfall-runoff modelling can be adversely influenced by erroneous input data. Therefore, the selection of accurate input data is crucial for development of reliable and predictive models. In this research project, a number of data analysis techniques were used to select good quality data for model calibration.

For calibration of model parameters, parameter optimisation was preferred to the trial and error visual comparison of observed and modelled output responses, due to subjectivity and time-consuming nature of the latter approach. It was also preferred in this study, since the model parameters obtained from calibration were used in the development of regional equations for use in gauged and ungauged catchments. Therefore, it was necessary to have a standard method which can be repeated, and produced the same result when the method is applied at different times for a catchment. An optimisation procedure was developed in this thesis, to estimate the model parameters of ILSAX. The procedure was designed to produce the 'best' set of model parameters that considered several storm events simultaneously. The PEST computer software program was used for the parameter optimisation. According to this procedure, the impervious area parameters can be obtained from frequent 'small' storm events, while the pervious area parameters can be obtained from less-frequent 'large' storm events.

Twenty two urban catchments in the Melbourne metropolitan area (Victoria) were considered in the model parameter optimisation. Several 'small' and 'large' storm events were considered for each catchment. However, it was found during the analysis that the selected 'large' storm events did not produce any pervious area runoff, and therefore it was not possible to estimate the pervious area parameters for these catchments. The Giralang urban catchment in Canberra (Australia) was then selected to demonstrate the optimisation procedure for estimating both impervious and pervious area parameters, since data on 'small' and 'large' storm events were available for this catchment. The calibration results were verified using different sets of storm events, which were not used in the calibration, for all catchments. The optimised model parameters obtained for each catchment were able to produce hydrographs similar to the observed hydrographs, during verification. The impervious area parameters obtained from optimisation agreed well with the information obtained from other sources such as areal photographs, site visits and published literature. Similarly, the pervious area parameters obtained for the Giralang catchment agreed well with the values given in the published literature.

If ILSAX is to be used for ungauged drainage systems for which no storm data are available, then the model parameters have to be estimated by some other means. One method is to estimate them through regional equations, if available. These regional equations generally relate the model parameters to measurable catchment properties. In this study, analyses were conducted to develop such regional equations for use in ungauged residential urban catchments in the Melbourne metropolitan area. The Melbourne metropolitan area was considered as one hydrologically homogeneous group, since the urban development is similar in the area. The equations were developed for the land-use parameters of directly connected impervious area percentage (DCIA) and supplementary area percentage (SA), and the directly connected impervious area depression storage (DS_i). Several influential catchment parameters such as catchment area, catchment slope, distance from the Central Business District to the catchment and household density were considered as independent variables in these regional equations.

A regional equation was developed for DCIA as a function of the household density. A similar equation was also developed to determine SA as a function of household density. DCIA was obtained from the model parameter optimisation using rainfall-runoff data (i.e. calibration), while SA and household density were obtained from the available drawings and field visits. These two equations showed a very good correlation with household density and therefore, DCIA and SA can be estimated accurately using these two equations. The city/shire councils generally have information on the household density in already-developed urban areas and therefore, these two equations can be used to estimate DCIA and SA for these areas. For new catchments, these equations can be used to estimate DCIA and SA based on the proposed household density.

The directly connected impervious area depression storage (DS_i) is the only ILSAX model loss parameter that was obtained from the calibration, and this is the loss parameter that is more sensitive for 'small' storm events of the urban drainage catchments. A regional equation was attempted for this parameter by relating with the catchment slope, since the catchment slope was found to have some correlation with DS_i according to past studies. However, the results in this study did not show a correlation between these two variables. Therefore, based on the results of this study, a range of 0 - 1 mm was recommended for DS_i. Because of the recommended range for DS_i, the sensitivity of DS_i against DCIA was revisited and found that DS_i was less sensitive compared to DCIA, in simulating the peak discharge and time to peak discharge for both 'small' and 'large' storm events. However, there is a little impact for runoff volume and hydrograph shape for 'small' storm events. Therefore, defining a range for DS_i is justified for modelling purposes and the user can choose a suitable value within this range from engineering judgement.

DECLARATION

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

Sunil Thosainge Dayaratne

31 August 2000

ACKNOWLEDGMENTS

First and foremost I would like to express my appreciation and my sincere gratitude to my supervisor Associate Professor, Dr. Chris Perera without whose constant support and help this thesis could not have been completed. His inspiration, enthusiasm and encouragement have made this research successful. I am grateful to him for always been available for discussion (especially given his extra ordinarily busy schedule) and for always managing to re-frame periods of frustration in positive light. I would also like to thank Chris for providing me with so many opportunities to broaden my Engineering experience, and for creating a flexible and enthusiastic research environment.

I am thankful to Dr Andrews Takyi (Canada) for his contribution to this research in early stages as the co-supervisor.

I am indebted to School of the Built Environment, Victoria University of Technology (VU) for awarding me the ARC Urban Stormwater Postgraduate Research Scholarship for my studies, and for use of their facilities. I also pay my sincere thanks to all the fellow postgraduate students, technical staff and administrative staff of VU, especially the research office staffs for their friendliness and willingness to help.

I would also like to thank the all city/shire council staff involved in this project for providing necessary information.

I am deeply conscious of the importance in my family, without whose emotional support I would not have tackled this research. I would memorise respectfully my late parents who gave me a priceless education and a respect for learning. Special thanks also due to my ever loving wife Ranjani, who has been a constant source of encouragement and who has provided me with the sense of well being necessary to complete this project and also boldly took up the challenge of grass widowhoodship for last few years. Last but not least; I thank my two beautiful children Kushan and Maduwanthi those who missed their dad a lot and their tolerance during this research project.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

LIST OF ABBREVIATIONS

Abbreviation	Meaning
ACT	Australian Capital Territory
AMC	Antecedent Moisture Condition
API	Antecedent Precipitation Index
ARI	Average Recurrence Interval
ARR	Australian Rainfall & Runoff
ARR87	Australian Rainfall and Runoff 1987 Edition
CN	Curve Number
DCIA	Directly connected impervious area
DRM	Deterministic Rational method
GIS	Geographic Information System
GPT	Gross Pollutant Trap
HGL	Hydraulic Grade Line
hhd	Household density
IFD	Intensity-Frequency-Duration curves
ILSAX	ILSAX urban stormwater drainage design & analysis program
MCBD	Melbourne Central Business District
RFM	Rational Formula method to compute peak discharge
RR Plot	Rainfall-Runoff depth plots
SA	Supplementary area
SCS	U.S. Soil Conservation Service
SRM	Statistical Rational method
SWMM	Storm Water Management Model
TIA	Total impervious area
USA	United States of America
USAFEMA	USA Federal Emergency Management Authority
VU	Victoria University of Technology, Australia

LIST OF PUBLICATIONS

This thesis is the result of four years of research work since September 1996 at School of the Built Environment, Victoria University of Technology. During this period, following research papers related to this project were published.

Dayaratne, S.T. and Perera, B.J.C. (2000), "Estimation of Impervious Area Parameters for Use in Urban Drainage Models", 3rd International Hydrology and Water Resources Symposium, Hydro2000, 20-23 November 2000, Perth, Australia, (Abstract accepted; paper under preparation).

Dayaratne, S.T. and Perera, B.J.C. (1999), "Parameter Optimisation of Urban Stormwater Drainage Models", 8th International Conference on Urban Storm Drainage, Sydney, Australia, 30 August-3 September 1999, pp 1768-1775.

Dayaratne, S.T. and Perera, B.J.C. (1999), "Towards Regionalisation of Urban Stormwater Drainage Model Parameters", 25th Hydrology and Water Resources Symposium and 2nd International Conference on Water Resources and Environment Research, Queensland, Australia, 6-8 July 1999, pp 825-830.

Maheepala, U., Dayaratne, S.T. and Perera, B.J.C. (1999), "Diagnostic Checking and Analysis of Hydrologic Data of Urban Stormwater Drainage Systems", 25th Hydrology and Water Resources Symposium and 2nd International Conference on Water Resources and Environment Research, Queensland, Australia, 6-8 July 1999, pp 813-818.

Dayaratne, S.T., Perera, B.J.C. and Takyi, A. (1998), "Sensitivity of Urban Storm Event Hydrographs to ILSAX Model Parameters", International Symposium on Stormwater Management, HydraStorm'98, Adelaide, Australia, 27-30 September 1998, pp 343-348.

Perera, B.J.C., Takyi, A.K., Nichol, E.R., Maheepala, U.K. and Dayaratne, S.T. (1998), Urban Stormwater Monitoring Project, First Progress Report for City of Hobsons Bay, Victoria University of Technology, Australia.
(Nine similar reports for nine other city/shire councils in Victoria were prepared)

CHAPTER 1

INTRODUCTION

1.1 URBAN HYDROLOGY

Urban hydrology is defined as the interdisciplinary science of water and its interrelationships with urban people (Jones, 1971). Perhaps the most obvious definition of urban hydrology would be the study of the hydrological processes occurring within the urban environment. It is a relatively new science, with the bulk of its knowledge accumulated since early 1960s. The beginning of urban hydrology can be traced to the time shortly after the automobile became the major means of transportation in the United States. Roads were paved to facilitate travel, allowing the growth of the suburbs where the commuter escaped the congestion of inner-city life. The result was the rapid creation of large impervious areas, producing significant problems such as regular flooding, inadequate drainage facilities, erosion, sedimentation and deterioration of water quality in receiving water bodies. Urbanisation of a rural catchment can cause a dramatic change to the hydrology and in particular to the peak flows. The science of urban hydrology was born out of the necessity to understand and control these problems.

Australia is a large continent of some 7,700 million square kilometers with a population of about 18 million. However, 80-85% of this population lives in 3.3% of the nation's land area. The level of urbanisation in Australia is estimated to be 85% of already-developed areas, with 63% of this is in nation's twelve major urban cities. Each of these 12 cities has a population of at least 100,000. The United Nations has estimated that the level of urbanisation for developed countries is about 73% (Fleming, 1994).

Storm drainage has become a central issue in urban planning and management, particularly in developed countries with substantial urban infrastructure in place. The magnitude of investments required to construct, operate and maintain urban storm drainage facilities and the potential for significant adverse social and environmental impacts mandate the use of the best possible methods for planning, analysis and design.

1.2 EFFECT OF URBANISATION ON STORM RUNOFF

The increase in population density and building density exert the most obvious influence on hydrological processes in an urban area. Modification of the land surface during urbanisation alters the stormwater runoff characteristics. The major modification which alters the runoff process is the impervious surfaces of the catchment such as roofs, side walks, roadways and parking lots, which were previously pervious.

Another factor is the natural channels, which were in existence before urbanisation, are often straightened, deepened and lined to make them hydraulically smoother. Gutters, drains and storm drainage pipes are laid in the urbanised area to convey runoff rapidly to stream channels. These increase flow velocities, which directly affect the timing of the runoff hydrographs. The combined effect of all these changes is to reduce the lag time of runoff. Since a larger volume of runoff (due to urbanisation) is discharged within a shorter time interval, the peak discharge inevitably increase.

The amount of waterborne waste increases in response to the growth in population and building density. The quality of stormwater runoff deteriorates as contaminants are washed from streets, roofs and paved areas. The disposal of both solid and waterborne wastes may also have an adverse effect on groundwater quality. The degradation of the quality of flows in both the drainage networks serving the urban area and the underlying aquifers, gives rise to major hydrological problems.

Urbanisation also considerably affects the climate of the area. It has been found that precipitation, evaporation and local temperature increase due to urbanisation (Hall, 1984). The urban atmosphere is characterised by a marked abundance of dust particles along with sulphur dioxide and other gases. These contaminants not only reduce the clarity of the atmosphere, thereby decreasing the amount of incoming radiation and sunshine, but also provide an excess of condensation nuclei that may change the nature of city fogs and affect the characteristics of precipitation. Increase of population density and impervious area leads to higher absorption of incoming radiation. Due to urbanisation the evaporation may reduce as transpiration (lack of vegetation) and soil moisture (loss of pervious areas) reduce. Reduction of evaporation increases the sensible heat resulting a temperature increase.

Urban catchments are rarely stationary with time (i.e. urbanisation takes place over a period of years). This can be seen in the Melbourne metropolitan area in Australia over the last few decades. During late seventies and early eighties, the allotment sizes remained the same but a trend was seen for larger dwellings of average areas around 180 to 280 m^2. Increases in the number of vehicles per property and changes to outdoor living styles caused considerable increases in paved areas in the properties in terms of garages and driveways, amounting to about 320 to 400 m^2 of impervious surfaces. These figures equated to about 40% to 50% of the allotment being impervious (Giancarro, 1995). During late eighties and early nineties, as a result of State and Federal Government policies, dealing with urban consolidation, the average size of residential allotments began to significantly reduce, increasing impervious areas because of the reduced garden size.

1.3 URBAN STORMWATER MANAGEMENT

Before 1980s, stormwater was considered as a nuisance and the main objective of stormwater management was to dispose stormwater as quickly as possible to receiving water bodies. This meant that no matter how large the rainfall or its duration, the drainage system was expected to remove runoff as quickly as possible, in an attempt to restore maximum convenience to the community in the shortest possible period of time. No consideration was given to stormwater as a valuable resource. Furthermore, the receiving water bodies were adversely affected due to poor quality stormwater.

In recent times, stormwater has been considered as a resource due to scarcity of water resources. Stormwater is a significant component of the urban water cycle, and its improved management offers potentially significant environmental, economic and social benefits. Urban stormwater management objectives now pursue the goal of ecological sustainable development and better environmental outcomes. This objective results in vastly improved stormwater quality. One of these technologies is the infiltration technology incorporating soaking wells, pervious tanks and biologically engineered soil filter medium. Infiltration techniques may provide an effective solution to overcoming stormwater contamination. One other technique is the reuse of stormwater. Reuse of treated stormwater can be considered as a substitute for other sources of water supply for non-potable uses.

In Australia, stormwater pollution was not considered as a serious problem until the 1980s. Stormwater management in Australia has developed greatly over the last fifteen years. In 1995, authorities in Melbourne took a particular interest in litter and gross pollutants, and all states introduced or tightened erosion and sedimentation controls (Robinson and O'Loughlin, 1999). Soil erosion and sediment controls, and stormwater treatment devices are now commonly used.

Structural and non-structural stormwater management measures often need to be combined to manage the hydrology of urban runoff and to remove stormwater pollutants. One group of stormwater management measures that has proved effective in removing stormwater pollutants associated with fine particulates (such as suspended solids, nutrients and toxicants) is constructed wetlands and ponds. Constructed wetlands also satisfy urban design objectives, providing passive recreational and landscape value, wildlife habitat and flood control. A gross pollutant trap is an another structural pollution control measure that traps litter and sediment to improve water quality in receiving waters. Community involvement in clean up programs and source controls, re-vegetation programs of disturbed land, and minimal bare soil in urban gardens (especially those on sloping land) are some of the non-structural measures.

The stormwater drainage network represents a large capital investment and hence due consideration should be given to its design, management and maintenance. For example, Cullino (1995) reported that the drainage network in Waverley, Victoria (Australia) was significantly under capacity due to recent greater building densities, and an expenditure of about $200 million as at 1995 was required for the existing underground drainage network to be replaced or augmented to cope with a five year storm event. To achieve the best practice in the design and whole-life cycle management of stormwater infrastructure requires the adoption of appropriate design standards dealing with major and minor storms, and the encouragement of practices to extend the retention time in the stormwater systems.

The Australian Rainfall and Runoff (The Institution of Engineers, 1987), referred to as ARR87 in this thesis, provides guidelines for design of stormwater drainage systems. These guidelines are based on the limited information and data available at the time of preparation of ARR87. However, the ARR87 encourages the use of innovative solutions in urban stormwater management and allows the designers to deviate from the guidelines when additional good quality data and design information are available.

1.4 SIGNIFICANCE OF THE RESEARCH

Urban flooding is a major social and economic problem in Australia. According to a study conducted by the Department of Primary Industries and Energy (1992), flood damage costs Australia around $300 million per year as at 1992, with about 200,000 urban properties across Australia prone to flooding due to a 100 year flood. For example, on 17 February 1972, an intense thunderstorm flooded many city buildings in Melbourne, causing extensive damage. The Australian newspaper on 19 February 1972 reported that the estimated damage due to this flood was in excess of $ 1 million as at 1972.

In addition to these massive economic costs of flood destruction, there are also major social disruptions associated with emotional disturbance, relocation, counselling and loss of important private and personal articles, and in some cases loss of human life. However, flooding is one of the most manageable of natural disasters, if flood prone areas are identified and suitable flood mitigation strategies are implemented.

The most practical way of identifying flood prone areas, and the effectiveness of flood mitigation strategies is by the application of mathematical models, which consider complex hydrological and hydraulic processes of these areas. The hydrologic models compute peak flows and/or flood hydrographs, which are required to design the system components of drainage systems to minimise flood damage. If there are errors in peak flows and/or flood hydrographs, the drainage system will be either undersized or oversized. The former results in flooding of the urban areas and causes inconvenience to the residents in the flood affected area. The latter produces an uneconomical design, which is equally undesirable.

Thus, for the design of an efficient and economic urban drainage system, it is important to estimate the design flows and/or flood hydrographs accurately.

When dealing with urban drainage design, in some cases the full flood hydrograph is not required. Simple peak flow design methods in particular the Statistical Rational method are sufficient to design inlets, pipes, gutters and channels in locations where rainfall variability and/or storage effects can be neglected. This is often the case for small urban catchments. In these design methods, it is assumed that the calculated peak discharge has the same average recurrence interval (ARI) as the design rainfall. These peak flow design methods can be considered as simple mathematical models.

In most cases, the design of urban drainage systems involves consideration of flood storage, permanent storage, off-channel storage, inter-drainage diversions, pumping installations and silting of drains. This requires knowledge of flood hydrographs instead of just flood peak. The full hydrograph can be obtained from the rainfall-runoff models such as ILSAX (O'Loughlin, 1993), RAFTS (WP Software, 1991), RORB (Laurenson and Mein, 1990), SWMM (U.S. Environmental Protection Agency, 1992) and WBNM (Boyd *et al.*, 2000). To use these rainfall-runoff models, it is necessary to estimate the model parameters and land-use parameters for the urban catchments under consideration. The parameters include those of the loss models (mainly infiltration and depression storage parameters) and characteristics of the catchment (such as directly connected impervious area, supplementary area and pervious area). The ideal method to determine model parameters is to calibrate the models using observed rainfall and runoff data. However, only few urban catchments are monitored for rainfall and runoff. This is mainly due to large cost associated with monitoring of these catchments. On the basis of an urban stormwater monitoring program conducted by Victoria University of Technology (VU) during 1996-99, it was estimated that the initial capital cost (as at 1997) was about $35,000 to gauge and monitor a typical 100 hectare urban catchment. This included one pluviometer and three flow meters. However, this cost did not cover the cost of installation, maintenance and downloading of data. Thus, it is expensive and impractical to monitor every urban catchment. Therefore, it is important to develop methods to estimate the model parameters accurately to use rainfall-runoff models for ungauged urban catchments.

ILSAX (O'Loughlin, 1993) is a relatively simple mathematical model and is widely used by the local government authorities and consultants in Australia for design and analysis of urban drainage systems. However, there are no adequate guidelines especially for estimating the ILSAX model parameters for ungauged urban catchments. This was addressed in this research project, among other issues. First, the ILSAX model was calibrated for selected gauged catchments considering appropriate catchment subdivision and observed storm events. From the results of calibration of these catchments, the regional equations were developed to estimate model parameters for use in ungauged urban stormwater catchments.

The results of this research project will guide users of ILSAX to develop reliable and accurate computer models for urban drainage systems. This will allow the users to compute the flow hydrographs more accurately than the currently available methods, which in turn allows the designers to analyse the flood mitigation strategies accurately. These procedures will assist in the formulation of effective flood mitigation strategies and in the development of suitable capital work programs to reduce flood damage and lessen social disruptions due to urban flooding.

1.5 AIMS OF THE PROJECT

The main aim of this research project is to develop improved methodologies for design and analysis of urban stormwater drainage systems using the ILSAX model and to provide further guidance to users on development of these models. To achieve the main aim of the study, the following specific tasks were undertaken.

(i) Collect and analyse data such as storm and runoff events, land-use conditions, soil and vegetation details for the 22 study catchments in Melbourne metropolitan area in Victoria (Australia) and one study catchment in Canberra (Australia).

(ii) Select suitable storm events of the study catchments for model parameter calibration.

(iii) Identify the most suitable ILSAX modelling options (out of several options available) to model various hydrological and hydraulic processes, to determine the appropriate catchment subdivision for calibration of models, and to determine the most sensitive model parameters.

(iv) Calibrate the ILSAX model parameters for the study.

(v) Develop regional equations to estimate model parameters and land-use parameters for use in hydrograph modelling of ungauged urban catchments in Melbourne metropolitan area.

(vi) Provide guidance for estimation of the ILSAX model parameters and modelling strategies for both gauged and ungauged urban stormwater catchments.

The scope of this research project was limited to the ILSAX model. Although the DRAINS model (O'Loughlin and Stack, 1998) is a more recent version of the ILSAX model, it was not used in this study, since the project was started prior to the release of DRAINS. However, the procedures and guidelines developed for ILSAX can be extended to DRAINS, since the hydrology and hydraulics of the two models are similar. Similarly, other urban drainage models [e.g. SWMM (U.S. Environmental Protection Agency, 1992), CIVILCAD (Surveying and Engineering Software, 1997) and MOUSE (Danish Hydraulic Institute, 1988)] use similar principles to those of ILSAX in modelling various urban drainage processes. Therefore, there is a possibility of using the results of this study with the other urban drainage computer models.

1.6 OUTLINE OF THE THESIS

Chapter 2 reviews the literature on modelling techniques used in current urban catchment models in simulating various urban drainage processes. They include loss modelling, overland flow modelling, pipe and channel modelling, and modelling of runoff through storages. The chapter also includes a discussion on possible errors in modelling these processes.

Chapter 3 discusses the results of a survey conducted among stormwater drainage practitioners in government and consulting offices in Victoria. The survey investigated the current practice used in design and analysis of urban stormwater drainage systems. It also identified the widely used urban drainage models in Victoria and the problems faced by the practitioners in using these computer models.

Chapter 4 describes the ILSAX model parameters and different options available in ILSAX to simulate various hydrological and hydraulic processes. Estimation of the ILSAX model parameters is also reviewed in this chapter.

The collection of hydrologic and physical data required for modelling of urban stormwater drainage systems of the study catchments is described in Chapter 5. It also discusses diagnostic checks used to check accuracy and consistency of the rainfall-runoff data.

A detailed study was carried out to study the different modelling options available in the ILSAX model, the sensitivity of model parameters with respect to simulated storm hydrographs and the effect of catchment subdivision on storm hydrographs using two urban catchments in Melbourne metropolitan area. The details of this study and the results are discussed in Chapter 6.

Chapter 7 describes the two-stage inner/outer optimisation procedure developed to optimise model parameters of the ILSAX model and the results of model parameters for study catchments. Calibration results were also compared with the information obtained from other sources and were verified using independent storm events. These details are also given in this chapter.

Chapter 8 reviews the literature on regionalisation techniques that can be used for urban catchment models to regionalise model parameters. Also the development of regional equations for impervious area model parameters is presented in this chapter.

A summary of the research study and the conclusions drawn from the study are presented in Chapter 9. Recommendations for future research work arising from the research in this thesis are also outlined in this chapter.

CHAPTER 2

URBAN STORMWATER DRAINAGE MODELLING

2.1 INTRODUCTION

Combined sewers were constructed in many cities of the United States before 1900 without recognising the need for segregation and treatment of domestic and industrial wastes from storm runoff (Hall, 1984). Although these systems still exist in older municipalities in the U.S., separate sewers have dominated the construction during the 20th century. Separate systems for stormwater drainage and sewerage are almost universal in Australia.

The main purpose of urban drainage systems is to collect stormwater and convey it to receiving waters, with minimal nuisance, danger or damage, at least in the conventional drainage systems. However, in recent times emphasis has been shifted from disposal of stormwater to total management of stormwater, considering stormwater as a resource (CEPA, 1993). In addition to collection and disposal of stormwater, several other objectives are considered in total management of stormwater. These objectives include: limiting pollutants entering receiving waters through water quality control measures such as wetlands; minimising other adverse impacts of urbanisation (e.g. erosion and sedimentation); water conservation in semi-arid and arid areas; integration of large-scale drainage works into overall town planning schemes with multipurpose land-use (such as drainage, recreation or transportation), and reuse of stormwater.

The design methods for urban drainage systems include a wide range from rule-of-thumb methods to computer models. The Statistical Rational method has been commonly used in Australia for computing flows for urban drainage design. However, there is an increased tendency in recent times to use computer models to analyse complex drainage systems. These models generally consider the major hydrological and hydraulic processes of urban drainage systems such as interception, infiltration (from pervious surfaces), depression storage, overland flow, gutter flow and pipe flow. These computer models can be used for both storm event modelling and continuous simulation. Storm event modelling which considers the generation of flood hydrographs due to a storm is important in urban drainage design. The continuous modelling, which deals with modelling of the drainage system over a long period, is important in estimating stormwater yield, which can be reused.

Urban hydrologic models have two major components: runoff generation and runoff routing. The runoff generation component is responsible for partitioning rainfall into surface runoff and losses, while the runoff routing component routes the surface runoff from the catchment to the outlet. Runoff from an urban drainage catchment consists of an initial runoff from impervious areas (such as roofs, buildings, roads and parking lots), which flows into the storm drainage system. There is also a delayed response accompanying infiltration and storage, which occurs in pervious areas that are flat or gently sloping such as gardens, parks or playgrounds. The pervious and impervious surfaces in an urban catchment can be expected to behave quite differently, both in terms of rainfall losses and travel lag-times. Most urban drainage models consider these pervious and impervious areas separately.

This chapter first describes the main features of an urban drainage system, followed by a brief description of the computer models that can be used for hydrograph modelling of these systems. Then, a review of the techniques used by different computer models in modelling of various components of the urban drainage rainfall-runoff process is presented. Finally, the likely errors produced by these models are discussed in this chapter.

2.2 COMPONENTS OF URBAN DRAINAGE SYSTEM

A stormwater drainage system describing processes and components are shown in Figure 2.1. The main parts of an urban drainage system are:

- Property drainage,
- Street drainage (including both piped and surface flows),
- Trunk drainage (consisting of large conduits, usually open channels located on lands reserved for drainage purpose), and
- Receiving water bodies.

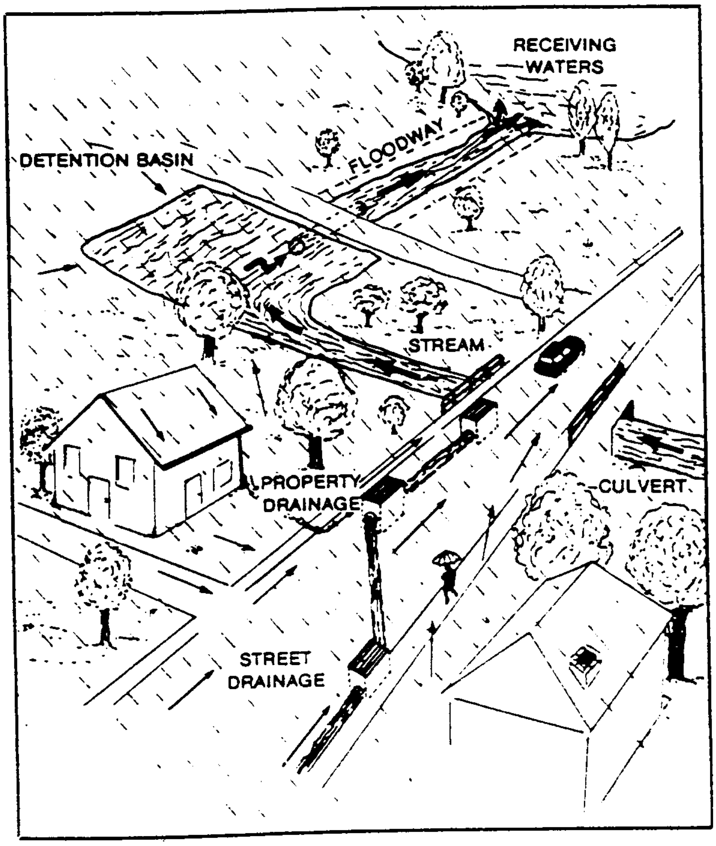


Figure 2.1: Urban Drainage System
(The Institution of Engineers, 1987)

2.2.1 Property Drainage

Properties consist of two types of hydrologically important surfaces namely impervious and pervious areas. Impervious areas consist of surfaces such as house roofs, backyard sheds, garages, driveways, access roads, parking places, tennis courts etc. Pervious areas in urban catchments consist of areas such as residential backyards, parks, playgrounds etc. The stormwater from both impervious and pervious surfaces is connected to the drainage system. The roof is the main impervious portion of a property.

The roof drainage system consists of gutters, down pipes, receiver boxes and runoff inlets. Roof gutters have different shapes such as rectangular and trapezoidal gutters. The gutters generally discharge freely into a receiver box, the depth of which can be selected so as to match the use of a downpipe of convenient size. The receiver box is then connected by downpipes. It should be noted that the receiver boxes are used only for large buildings such as office buildings, schools, factories etc., and not for small houses.

2.2.2 Street Drainage

Streets are normally drained by a network of gutters, pits and pipes. The street drainage system collects runoff from road surfaces as well as land adjoining streets, and discharged to trunk drains. In addition, in some cases runoff from properties is also disposed to the street drainage system.

The stormwater from the street gutter system enters the underground drainage system through inlets located in street gutters. There are three types of inlets in use, namely kerb inlets, gutter inlets and combined inlets. The kerb inlet has a vertical opening to catch the gutter flow. Although the gutter may be depressed slightly in front of the kerb inlet, it offers no obstruction to traffic. The gutter inlet is an opening covered by a horizontal grate through which stormwater enters. The disadvantage of the gutter inlet is that debris collecting on the grate may block the inlet (Hammer, 1977). Combined inlets, composed of both kerb and gutter openings, are also common in urban drainage systems.

Street grade, kerb design and gutter depression define the best type of inlet for a particular situation. Nevertheless, minimising traffic interference and reducing inlet blockage often take precedence over hydraulic efficiency in selecting the type of inlet.

2.2.3 Trunk Drainage

The trunk drainage system consists of large conduits or open channels (concrete or earth) which carry runoff from the local street drainage system to receiving waters. They are generally located in a dedicated drainage easement. Trunk drains serve several sub-areas, which are physically large, and therefore, the overflows are likely to cause direct damage and prolonged inconvenience (The Institution of Engineers, 1987).

2.2.4 Retention and Detention Basins

Retention basins, often known as water quality control ponds at least in recent times, are small lakes located in-stream or off-stream along urban waterways. They can be extremely effective in removing pollutants, since they allow a range of physical, chemical and/or biological processes to take place, which improve water quality. Retention basins hold stormwater for considerable periods, which cause stormwater to be in the hydrologic cycle via infiltration, percolation and evapotranspiration.

Detention basins are commonly known as retarding and compensating basins in Australia. The detention basins hold runoff for short time periods to reduce peak flow rates and later release into natural or artificial watercourses. Therefore, the volume of stormwater runoff is relatively unchanged from the original volume. They may be sport fields of specific sizes and shapes (e.g. football and cricket grounds). The sides of basins are usually sloping earth embankments, suitable for spectator use if they are sport fields.

Both retention and detention basins have been used extensively throughout Australia and overseas, to provide economical and practical solutions to a range of drainage problems.

2.2.5 Receiving Water Bodies

The major receiving water bodies that are considered in urban drainage include rivers, lakes, bays, the sea and ground water storages.

2.3 URBAN DRAINAGE COMPUTER MODELS

The Statistical Rational method is still being widely used as the preferred method both in Australia and overseas for pipe design in urban drainage systems instead of the more advanced runoff routing procedures which produce hydrographs (Goyen *et al*., 1989). Although the hydrograph methods were not well established in 1960s, even at that time the practitioners derived hydrographs using the Rational method. They generated triangular and trapezoidal hydrographs using the peak discharge from Rational method and time of concentration by other methods.

As reported in Aitken (1975), there had been many attempts to use overseas computer models directly or to modify overseas models to suit Australian urban catchments. At that time, two problems were found in selecting overseas computer models for use in Australia. The first problem was the existence of separate systems for urban stormwater and sewage water collection in Australia as opposed to single systems used in these models. The second problem was the soil types underlying the urban areas in different cities in Australia (with high variations in infiltration characteristics), which were different to those that were used in the development of the overseas models. However, these problems are not issues anymore since most overseas drainage models are flexible enough to cater for the above two problems. The examples are SWMM (U.S. Environmental Protection Agency, 1992) and MOUSE (Danish Hydraulic Institute, 1988). In some cases, the overseas computer programs are successfully modified to suit the Australian conditions. A good example of this is the ILSAX (O'Loughlin, 1993) model.

Several urban drainage computer models are used in Australia and overseas. A survey conducted as part of this thesis and described in Chapter 3 indicated that ILSAX (O'Loughlin, 1993), RAFTS (WP Software, 1991), RatHGL (WP Software, 1992), CIVILCAD (Surveying and Engineering Software, 1997), RORB (Laurenson and Mein, 1990), SWMM (U.S. Environmental Protection Agency, 1992), WBNM (Boyd *et al.*, 2000) and HEC-RAS (Hydrologic Engineering Centre, 2000) were used in Victoria as urban drainage computer models for design and analysis. The order given above is based on usage of these models in Victoria based on the above survey. ILSAX was widely used followed by RAFTS etc.

The other widely used urban drainage models in Australia and overseas in modelling event hydrographs are AUSQUAL (White and Cattell, 1992), STORM (U.S. Army Corps of Engineers, 1977), MOUSE (Danish Hydraulic Institute, 1988), DR3M (Alley and Smith, 1990) and HSPF (Johanson *et al.*, 1984). The models AUSQUAL, STORM, DR3M and HSPF were specially developed for water quality simulation in urban drainage systems while MOUSE was developed for both hydraulic and water quality simulation.

These models are briefly described in the following sections, except the ILSAX model. The ILSAX model, which is used in this thesis, is described in detail in Chapter 4. In addition to the above computer models, the Statistical Rational method is described, since it is widely used in Australia. In the following sections, the Statistical Rational method is described first, followed by the computer models based on Statistical Rational method. Then the pipe drainage models are discussed. Finally, the other catchment and hydraulic models are presented.

2.3.1 Statistical Rational Method (SRM)

The Rational Formula method (RFM) is a simple mathematical rainfall-peak runoff model. Two forms of RFM have been used in the past, namely deterministic and statistical. Aitken (1975) in his study found that the RFM as a deterministic model was of little value in the urban situation. As a statistical model (SRM), it was found to have some merit. However, considerable care has to be taken in the selection of the runoff coefficient, which is always subjective and depends on personal judgement. The ARR87 recommends the SRM.

Several major assumptions are made in SRM. They are:

(i) The design storm is uniform in intensity over the catchment in both time and space,

(ii) The rainfall duration is equal to the time of concentration of the catchment,

(iii) The peak runoff is a fraction of the average rainfall rather than the residual after abstraction of losses,

(iv) The return period of the peak discharge is equal to that of the rainfall intensity, and

(v) Rainfall runoff response is linear.

SRM is given in Equation 2.1.

$$Q_{peak} = \frac{C_y \cdot I \cdot A}{360} \tag{2.1}$$

where Q_{peak} is the peak discharge (m^3/s),

C_y is the runoff coefficient corresponding to return period y,

A is the catchment area (ha), and

I is the average rainfall intensity (mm/h) of a storm with return period y and storm duration t_c.

For single land-use catchments, losses are assumed to be the same for the whole catchment. Therefore, the runoff coefficient is a function of return period and fraction of imperviousness. If the catchment consists with different land-uses having different losses, then the area-weighted runoff coefficient should be computed (Argue, 1986). The time of concentration in urban catchments can be calculated by adding of property time, gutter time and pipe flow time.

The main shortfalls of SRM are:

- The subjectivity of the catchment runoff coefficient (although there are guidelines given in ARR87 based on limited data),
- Uniformly distributed storms are rarely experienced over the catchment,
- Storms are not uniform in intensity,
- The return period of runoff and rainfall would rarely agree,
- The catchment time of concentration may be unknown or at best variable,
- It is applicable only to small catchments, and
- Only peak discharge can be estimated.

2.3.2 Models Based on Statistical Rational Method

2.3.2.1 Wallingford Procedure

Hall (1984) presented a complete description of the Wallingford procedure, which sets out a modified version of the Rational Formula method for homogeneous areas up to 150 hectares. This method allowed for the routing effects by introducing a routing coefficient in addition to volumetric runoff coefficient. The Wallingford Rational Formula method, applied mostly in the UK, is presented in Equation 2.2. This equation can be applied either actual storm or design storm.

$$Q_P = 2.78\ C_V\ C_R\ I\ A \tag{2.2}$$

where C_V is the volumetric runoff coefficient,
C_R is the routing coefficient (a value 1.3 is recommended for design),
Q_P is the peak discharge (l/s),
I is the average rainfall intensity (mm/h), and
A is the total catchment area (ha).

In Equation 2.2, $C_V = PR / 100$, where PR is the percentage runoff of the catchment (as a ratio of rainfall) and estimated from Equation 2.3.

$$PR = 0.829\ IMP + 25.0\ SOIL + 0.078\ UCWI - 20.7 \tag{2.3}$$

where IMP is the directly connected impervious area (%),
SOIL is the soil index (map available for U.K.), and
UCWI is an antecedent wetness index.

If the catchment consists of only impervious areas, then

$$C_V = PR / IMP \tag{2.4}$$

where PR is computed from Equation 2.3. This method has been used by one shire council in Victoria at the time of the survey (i.e. May 1997) for one catchment. In this application, the user assumed a value for SOIL index for this catchment based on U.K. soil indices, but considering similar characteristics of the soil.

2.3.2.2 RatHGL

RatHGL model (WP Software, 1992) uses the Statistical Rational method (Rat) for modelling hydrology and the Hydraulic Grade Line (HGL) analysis for modelling hydraulics of the urban drainage systems. RatHGL considers all possible area/time of concentration combinations above each node (i.e. partial area effects) and computes the maximum flow. In estimating the HGL at each node, friction energy losses along the pipes are estimated using the Colebrook-White or Darcy-Weisbach equations. Pit junction energy losses in the form of empirical pit pressure change coefficients are applied to entrance/exit flows and pit configurations.

2.3.3 Pipe Drainage Models

2.3.3.1 SWMM

SWMM (U.S. Environmental Protection Agency, 1992) is a comprehensive computer model for simulation of urban runoff quantity and quality in storm and combined sewer systems. SWMM stands for Storm Water Management Model. All aspects of the urban hydrologic and quality cycles are simulated, including surface runoff, transport through the drainage network, storage and treatment. Like most hydrologic models, SWMM subdivides the overall catchment into subcatchments, predicting runoff from the subcatchments on the basis of their individual properties, and combining their outflows using a flow routing scheme. SWMM can also simulate backwater effects.

In SWMM, subcatchments are represented mathematically as spatially lumped, nonlinear reservoirs, and their outflows are routed via the channel/pipe. Subcatchments are subdivided into three subareas, impervious area with and without depression storage, and pervious areas with depression storage. Flow from one subarea is not routed over another subarea. Overland flow is generated from each of the three subareas by approximating them as nonlinear reservoirs. This nonlinear reservoir is established by combining the continuity equation with Manning's equation. Infiltration from pervious areas can be computed by either Horton or Green-Ampt equation.

Flow routing in channel/pipes is also performed through a nonlinear reservoir by combining the continuity equation with Manning's equation.

2.3.3.2 MOUSE

MOUSE (Danish Hydraulic Institute, 1988) stands for **M**odelling **O**f **U**rban **SE**wers and is a hydrologic-hydraulic model applicable only for modelling of urban catchments. This model is used extensively for sewerage design in Australia compared to the design of stormwater drainage networks (Lindberg and Car, 1992). The hydrologic part of the model deals with simulation of runoff using two methods: a simple method based on time-area diagram and a complex method based on kinematic wave theory and continuity equation. The hydraulic part of the model simulates flow routing in closed conduits or open channels. Three options are available in MOUSE to compute depth and velocity of flow. The first is the kinematic wave method, which is mostly applied to part full flow conditions. The second is the diffusive wave method, which considers backwater and surcharge in the systems. The last is the dynamic wave method, which provides a full hydrodynamic solution. MOUSE, like SWMM, is well-suited for analysing the hydraulic performance of complex looped sewer systems including overflows, storage basins and pumping stations. Water quality modelling and prediction is also included in the MOUSE model.

2.3.3.3 CIVILCAD

CIVILCAD (Surveying and Engineering Software, 1997) is a multipurpose design computer package. It was mainly a design tool for road design, although it provides facilities for drainage design. However, this package is rarely used only for drainage design by city/shire councils in Victoria (personal communication with R. Silva, Buloke

Shire Council, Victoria, 1999). The drainage module of CIVILCAD performs the following basic functions:

- Perform hydrological calculations to calculate surface runoff, gutter flow and pipe flow.
- Design the pipes interactively to obtain the optimum combination of diameters, slopes and depths of pipes.
- Perform backwater analysis to ensure satisfactory hydraulic performance.
- Produce reports of calculations including tables and figures (both hydrographs and longitudinal sections).

The CIVILCAD uses ILSAX model procedures to compute rainfall excess. The hydraulic grade line method is used for pipe flow analysis.

2.3.4 Other Catchment and Hydraulic Models

2.3.4.1 RAFTS

The RAFTS (WP Software, 1991) model has been used in Australia since 1980s. RAFTS stands for Runoff Analysis and Flow Training Simulation. RAFTS simulates runoff hydrographs at defined points throughout the catchment for specific rainfall events (both observed and design). RAFTS is suitable for modelling of catchments ranging from rural to fully urbanised. The model is capable of analysing catchments comprising natural waterways, formalised channels, pipes, retarding and retention basins, and any combination of these. There are no specific limitations on the catchment size. It has been successfully used for on-site detention and on catchments up to 20,000 km^2 (WP Software, 1991). RAFTS can be used in event or continuous mode, with appropriate rainfall inputs.

Like most rainfall-runoff models, RAFTS requires the catchment to be sub-divided into several subcatchments. Each subcatchment is then divided into 10 subareas within RAFTS based on lines of equal travel time or isochrones. Runoff from each subarea is routed using the Laurenson's (1964) runoff routing procedure to obtain the outflow hydrograph of a subcatchment. RAFTS can model pervious and impervious areas separately. However, it does not consider directly connected impervious area and supplementary area separately as in ILSAX and SWMM. RAFTS uses initial loss-continuing loss model or Philip's infiltration equation to simulate the excess runoff.

Pipe flow is determined using Manning's equation. Overflow is computed as the portion of the total subcatchment inflow, which cannot flow through the pipe because of inadequate capacity. Pit inlet capacity restriction is not considered in this model. For flood routing through pipes and trunk drainage system, the Muskingum procedure is used. As an alternative to channel routing where physical data is lacking, RAFTS allows a simple channel lagging procedure whereby the flood hydrograph is simply lagged by an appropriate time with zero attenuation. Lag times are calculated in RAFTS using flow velocity computed from the Manning's equation. Puls' level pool routing procedure is used in the retarding and retention basins.

2.3.4.2 RORB

RORB (Laurenson and Mein, 1990) is a general runoff and streamflow routing program that can be used to calculate flood hydrographs from rainfall and other channel inputs. The typical application of this model requires the subdivision of the catchment into several subcatchments. Each subcatchment is represented by a node. Nonlinear storages (which are called reach storages) are connected between adjacent nodes to model the flow from one subcatchment to another. For each subcatchment, RORB computes the rainfall excess, which is routed through the reach storage. The initial loss-continuing loss model or initial loss-proportional loss can be used to compute the rainfall excess. Areal variability of loss parameters from one subarea to another is provided through the fraction imperviousness incorporated in the loss model for urban catchments.

The RORB model accommodates the effect of urbanisation by weighting reach length by a factor. This factor is used to scale the reach length, which is assumed to be a surrogate for the catchment and channel lag.

The main drawback of using RORB for urban catchments is the lumping of all impervious areas in a subcatchment without considering directly connected and supplementary areas separately. It does not model the pipe hydraulics.

2.3.4.3 WBNM

The WBNM (Boyd *et al.,* 2000) model is an event based nonlinear runoff routing model, capable of modelling runoff from small and large catchments. In WBNM, a catchment is divided into a number of subcatchments and is represented by a separate storage element. Each urbanised subcatchment is divided into pervious and impervious subareas, with separate rainfall losses to compute the rainfall excess. Five alternative loss models (i.e. initial loss-constant loss rate, initial loss-loss rates varying in steps, initial loss–runoff proportion, Horton continually varying loss rate and Green-Ampt varying loss) are available in WBNM to model rainfall losses. Overland flow in each subcatchment is modelled by a nonlinear reservoir with time-lag. Three options available for channel routing are:

a) Nonlinear routing using a "channel factor" selected to reflect the increased flow velocities in the "improved" channel.
b) Muskingum routing, with its parameters selected based on the translation and attenuation properties of the reach.
c) Time-lag method, in which the upstream hydrograph is delayed through the reach by a specified time (but without attenuation) to produce the downstream hydrograph.

2.3.4.4 AUSQUAL

AUSQUAL (White and Cattell, 1992) was developed as a tool for use in water quality management in Australia. **AUS** refers to Australian conditions, **Q** indicates that it is an essentially a quality model while **UAL** states that the model is based on unit area loading of constant concentration. The hydrological simulation in AUSQUAL is based on the subdivision of the catchment into several subcatchments, to account for different land uses. Each subcatchment considers three different flow path namely overland, gutter/pipe and trunk drainage. AUSQUAL determines the hydrologic response of the catchment by generating a series of triangular hydrographs for each subcatchment using the time-area model, based on times of concentration of the three flow paths. After generation of the individual subcatchment hydrographs, each hydrograph is routed to the catchment outlet by applying the appropriate time lag to the hydrograph considering a representative velocity and the flow path length.

The hydraulic component of AUSQUAL is weak and therefore, it is not basically set up to analyse the detailed pipe networks.

2.3.4.5 STORM

The U.S. Army Corps of Engineers (1977) developed **Sto**rmwater **R**unoff **M**odel (STORM) to analyse quantity and quality of runoff from urban and nonurban catchments. STORM was primarily developed to evaluate the stormwater storage and treatment capacity required to reduce untreated overflows below specified values. Computations of treatment, storage and overflow proceed in an hourly basis by simple runoff volume and pollutant mass balance for the entire catchment. Since this model runs on hourly time step, this model is not suitable for small catchments where time of concentration is less than one hour.

STORM is a continuous simulation model. This model is basically a planning model and therefore, not suitable for detailed quantity or quality modelling. Runoff can be determined in one of three ways. They are the runoff coefficient method, the U.S. Soil Conservation Service (SCS) curve number technique and a method which combines the above two.

2.3.4.6 DR3M

DR3M (Alley and Smith, 1990) stands for **D**istributed **R**outing **R**ainfall-**R**unoff **M**odel. This model simulates the quantity and quality of surface runoff from urban catchments. The model can be operated as a lumped parameter model or as a distributed parameter model. The DR3M model considers separate volumes of pervious and impervious area rainfall excess. Infiltration losses from pervious area are computed using the Green-Ampt infiltration equation. Two types of impervious surfaces namely directly connected and supplementary area are considered by the model in computing impervious area rainfall excess like in ILSAX and SWMM. The kinematic wave method is used to generate runoff from rainfall over subcatchments and to route through drainage system. This model can produce runoff hydrographs and quality pollutographs at any location of the drainage system.

2.3.4.7 HSPF

HSPF (Johanson *et al.*, 1984) stands for Hydrocomp Simulation Program in Fortran. It can be used as an event model or continuous model. HSPF is essentially an agricultural model, improved to handle the impervious areas. In this model, the rainfall excess computations are based on the Stanford watershed model (Craford and Linsley, 1966). Impervious area water balance is similar to that in the pervious areas without infiltration. It cannot model detailed pipe networks. Therefore, this model is not suited to analyse detail drainage systems except as a planning model. The model can be used on large catchments of area up to several thousands square kilometers, as a planning model.

2.3.4.8 HEC-RAS

HEC-RAS (US Army Corps of Engineers, 2000) is widely used in Australia for flood profile calculations. In urban drainage context, HEC-RAS can be used to design and analyse of trunk drainage systems. HEC-RAS is designed to perform one-dimensional hydraulic calculations for a full network of natural and constructed channels. The basic computational procedure is based on the solution of the one-dimensional energy equation. Energy losses are evaluated by Manning’s equation. The momentum equation is used in situations where the water surface profile is rapidly varied.

2.4 MODELLING APPROACHES USED IN COMPUTER MODELS

Most urban catchment models use hydrologic and hydraulic computations such as loss modelling, overland flow routing and pipe routing in simulating the runoff response. Different methods are used in different models in computing these hydrologic and hydraulic responses. These methods are summarised in Table 2.1 for the models described in Section 2.3 (including ILSAX).

In Table 2.1, models are described as event or continuous models in the second column. The continuous model uses daily rainfall and daily evaporation data to provide a continuous accounting of soil moisture on a daily basis. In event models, model is set up for an event such as a flood event. Evaporation losses are modelled in continuous models, but generally not in event models. The third column defines the scheme used for modelling pervious and impervious area. In some models, two areas are analysed separately (Separate), while in the other models they are lumped (Lumped). The other columns define the methods used for loss modelling, overland flow routing and pipe flow routing.

2.4.1 Loss Modelling

Storm loss for an event is defined as the amount of precipitation that does not appear as direct runoff. The storm loss includes moisture intercepted by vegetation (interception loss), percolated into soil (infiltration) or retained by surface storage (depression). It can occur from both impervious and pervious surfaces. These losses can be modelled by four different loss components: impervious area depression storage (impervious area initial loss), pervious area depression storage (pervious area initial loss), pervious area continuous loss, and evaporation loss from both impervious and pervious surfaces. However, in storm event hydrograph modelling, evaporation from pervious and impervious areas can be neglected, as it is insignificant compared to other losses.

Table 2.1: Modelling Methods Used in Different Models

Model	Continuous or Event Model	Impervious and Pervious Area Lumped or Separated	Loss Model	Overland Flow Routing Method	Pipe Routing Method	Can Water Quality Parameters be Simulated?	Output
Wallingford	Event	Lumped	Statistical Rational method	Not considered	Not considered	No	Only peak discharge
RatHGL	Event	Lumped	Statistical Rational method	Not considered	Hydraulic grade line	No	Hydrologic and hydraulic information at each node
CIVILCAD	Event	Separate	Horton	Time-area	Manning's Equation	No	Hydrographs at each pit can be modelled.
ILSAX	Event	Separate	Horton	Time-area	Manning's equation	No	Hydrographs at each pit can be modelled.
SWMM	• Event or • Continuous	Separate	• Horton • Green-Ampt	Nonlinear reservoir	Kinematic wave	Yes	Hydrograph at each pit
MOUSE	• Event or • Continuous	Lumped	Horton	• Time-area • Kinematic wave	• Kinematic wave • Diffusive wave • Dynamic wave	Yes	Can be generated runoff hydrographs at defined points

Table 2.1 Continued ……..

Model	Continuous or Event Model	Impervious and Pervious Area Lumped or Separated	Loss Model	Overland Flow Routing Method	Pipe Routing Method	Can Water Quality Parameters be Simulated?	Output
RAFTS-XP	• Event or • Continuous	Separate	• Philip's equation or • ARBM model	Laurenson's (1964) runoff routing procedure	• Muskingum method or • Time-lag	No	Can be generated runoff hydrographs at defined points
RORB	Event	Lumped	Runoff coefficient	Nonlinear	Nonlinear	No	Can be generated runoff hydrographs at defined points
WBNM2000	Event	Separate	• Initial loss-constant loss rate • Initial loss-loss rate varying insteps • Initial loss-runoff proportion • Horton • Green-Ampt	Nonlinear reservoir with time lag	• Nonlinear • Time-lag • Muskingum	No	Can be generated runoff hydrographs at defined points

Table 2.1 Continued ……..

Model	Continuous or Event Model	Impervious and Pervious Area Lumped or Separated	Loss Model	Overland Flow Routing Method	Pipe Routing Method	Can Water Quality Parameters be Simulated?	Output
AUSQUAL	• Event • Continuous	• Lumped • Separate	• Runoff coefficient • Boughton model	Time-area	Time-lag	Yes	One hydrograph for single catchment
STORM	• Event or • Continuous	Lumped	• Runoff coefficient • SCS method	Not considered	Not considered	Yes	One hydrograph for single catchment
DR3M	• Event or • Continuous	Separate	Green-Ampt	Kinematic wave	Kinematic wave	Yes	Can be generated runoff hydrographs at defined points
HSPF	• Event or • Continuous	Separate	Stanford model	Kinematic wave	Kinematic wave	Yes	One hydrograph for single catchment
HEC-RAS	Not applicable	• Manning's equation • Gradually varied flow	No	Can be generated water surface profile			

Computer models simulating the rainfall runoff process (including urban drainage models) have focused mainly on catchment hydraulics, in particular, overland flow, channel flow and large storage areas. Less emphasis has been placed on losses (especially infiltration), which can have a greater bearing on the accuracy of results than the catchment hydraulics (Priestley *et al.*, 1997). The paucity of infiltration on initial losses constitutes one of the greatest weaknesses in Australian flood design (Pilgrim and Robinson, 1988). However, as Mein and Goyen (1988) stated, the uncertainties of soil behaviour and the areal variability of soil properties do not justify the use of anything more than the simplest loss model in rainfall-runoff modelling. In some urban drainage computer models, storm losses from impervious and pervious areas are modelled separately. Examples are DR3M (Alley and Smith, 1990), SWMM (U.S. Environmental Protection Agency, 1992), ILSAX (O'Loughlin, 1993) and CIVILCAD (Surveying and Engineering Software, 1997). However, the models such as RORB (Laurenson and Mein, 1990), RAFTS (WP Software, 1991) and STORM (U.S. Army Corps of Engineers, 1977) lump these two areas together.

2.4.1.1 Impervious and pervious area depression storage

Depression storage is a volume that must be filled prior to the occurrence of runoff on both pervious and impervious areas and can be considered as an initial loss. It represents a loss caused by such phenomena as interception, surface wetting and surface ponding. The losses from impervious areas are simple to calculate. Simply, the depression storage is subtracted from rainfall hyetograph to compute the rainfall excess. The impervious area depression storage is depleted only by evaporation. However, in storm event modelling, the evaporation loss is insignificant and therefore the impervious area depression storage is assumed to be a constant in most urban drainage models. Typical values would be 0 to 2 mm for impervious area depression storage.

The pervious area depression storage is subject to infiltration and evaporation, and therefore it is continuously and rapidly replenished. However, it is small compared to infiltration losses. Therefore, the pervious area depression storage is also assumed to be constant in most urban drainage models. Typical values would be 2 to 10 mm for pervious area depression storage (O'Loughlin, 1993).

2.4.1.2 Pervious area infiltration loss

Several equations have been developed for modelling the process of water entry into soil from surface at a point. Some of them are based on fitting empirical equations to infiltration data; others are analytical/numerical solutions to the complex equations describing the water movement in soils, with various simplifying assumptions. The Horton equation is one such empirical equation, while the Philip and Green-Ampt equations are examples of theoretical equations used in urban drainage models. Ideally, the parameters of these equations should be estimated using the results of field infiltrometer tests for at several sites of the catchment due to several antecedent moisture conditions.

Other types of infiltration loss models are spatially lumped models, which have been conceptualised. Some of the most frequently used methods for spatially lumped losses include constant loss rate, initial loss-continuing loss, proportional loss, antecedent precipitation index and SCS curve procedure (Nandakumar *et al.*, 1994). From these methods, initial loss-continuing loss, constant loss rate (i.e. runoff coefficient) and SCS methods have been used in urban drainage computer models. Such conceptualised models do not consider the spatial variability or the temporal pattern of storm losses adequately. The model parameters of these types are estimated using the total catchment runoff. However, the spatially lumped loss models are widely used because of their simplicity and ability to approximate catchment runoff behaviour (Nandakumar *et al.*, 1994). In some computer models, a component of the loss model is represented by a point infiltration equation. For example, the ILSAX model uses initial loss-continuing loss model with its continuing loss model represented by the Horton equation, which considers average conditions over the entire catchment. Point infiltration methods and spatially lumped models used in urban drainage models are described in the following sections.

(a) Horton equation

Horton (1940) observed that infiltration capacity decreased with time until it reached a constant value, and described this process by the exponential equation given in Equation 2.5.

$$f_p = f_c + (f_0 - f_c)^{-k.t} \qquad (2.5)$$

where f_p is the infiltration capacity of soil (m/s),
f_c is the minimum or ultimate value of f_p (m/s),
f_0 is the maximum or initial value of f_p (m/s),
t is the time from beginning of storm (s), and
k is the decay coefficient (s^{-1}).

Due to its simplicity, several hydrologic models such as SWMM (U.S. Environmental Protection Agency, 1992), ILSAX (O'Loughlin, 1993) and MOUSE (Danish Hydraulic Institute, 1988) use the Horton equation in infiltration loss modelling. The infiltration parameters (i.e. f_0, f_c and k) are generally obtained from calibration considering the average conditions over the entire catchment or from tables, which relate the parameters to soil type (for ungauged catchments) and initial moisture content. Estimating these parameters has never been an easy task. Literature values vary largely and cause a great deal of uncertainty. Theoretically, the parameters depend on soil type, vegetation and soil moisture content.

The Horton equation is generally applicable for shallow ponded conditions (i.e. infiltration at potential rate). Therefore, the applicability of this equation during intermittent rainfall is questionable, although the above models have neglected this effect.

(b) Green-Ampt model

The Green and Ampt (1911) infiltration model is probably the most physically-based model used to determine the losses due to infiltration. The most significant advantage of the Green-Ampt model over the other similar models is that the parameters have a physical basis. The Green-Ampt model uses Darcy's Law assuming ponded conditions, a constant matric potential at the wetting front, and uniform moisture content and conductivity to model infiltration. Mein and Larson (1971) showed that the Green-Ampt model could be presented by a two-stage model given by Equations 2.6 and 2.7 for a constant intensity rainfall at the surface (rather than ponded conditions).

$$\text{For } F < F_s\text{:} \quad f = i \quad \text{and;} \quad F_s = \frac{S_u(IMD)}{\frac{i}{k_s} - 1}, \qquad \text{for } i > k_s \tag{2.6}$$

F_s is not calculated for $i \leq k_s$,

For $F \geq F_s$: $f = f_p$ and,

$$f_p = k_s\left(1 + \frac{S_u(IMD)}{F}\right) \tag{2.7}$$

where i is the rainfall intensity (m/s),

f is the infiltration capacity (m/s),

f_p is the infiltration capacity into soil (m/s),

F is the cumulative infiltration volume (m^3/m^2),

F_s is the cumulative infiltration volume required to cause surface saturation (m),

S_u is the average capillary suction at the wetting form (m of water),

IMD is the initial moisture deficit for this event (m/m), and

k_s is the saturated hydraulic conductivity of soil (m/s).

Infiltration is thus related to the volume of water infiltrated as well as to the moisture conditions in the surface soil zone. Like the Horton equation, the parameters (i.e. k_s, S and IMD) can be estimated based on available field data (i.e. hydraulic conductivity of soil, soil porosity, actual moisture content and capillary suction) or through calibration using rainfall/runoff data. In the SWMM model, the infiltration from pervious areas may be computed by either the Horton or Green-Ampt equations.

(c) Phillip equation

Philip (1957) obtained a two-term solution (i.e. ‘S $t^{1/2}$’ and ‘A t’ in Equation 2.8) to the Richards’ equation for a homogeneous soil with uniform initial moisture content and ponded conditions at the surface. This is a physically based model having two parameters (i.e. *S* and *A*).

$$F = S\,t^{1/2} + A\,t \qquad (2.8)$$

where	F	is the cumulative infiltrated volume,
	S and A	are functions of soil water diffusivity, initial water content and ponded depth, and
	t	is the time from start of storm event.

The RAFTS model (WP Software, 1991) uses the Phillip equation to compute rainfall excess. Parameters can be estimated using the measurements of soil properties or by model calibration using rainfall/runoff data. Sorptivity (*S*) can be estimated from the infiltrometer tests. The parameter *A* can be estimated from the measurements of saturated hydraulic conductivity.

(d) SCS method

The Soil Conservation Service’s (SCS) runoff equation, which was in common use since mid 1950’s in the USA, was developed for estimating direct runoff from storm rainfall. This equation was developed based on more than 20 years of rainfall-runoff studies of small rural catchments (U.S. Soil Conservation Service, 1985). It was originally developed to estimate runoff volume and peak discharge for design of soil conservation works and flood control projects, but later extended to estimate the complete hydrograph (Kumar and Jain, 1982). This method is used in STORM (U.S. Army Corps of Engineers, 1977) model.

The SCS method uses physical characteristics of catchment, soil group and soil wetness conditions. This method was originally proposed for rural catchments, but after some modifications, is now applicable on urban catchments. The SCS method uses a curvilinear relationship between accumulated runoff and accumulated rainfall. The curve number

(CN) is defined in terms of soil type, antecedent moisture condition, land-use treatment and hydrological condition of the catchment. The basic equations used in this method for a rainfall event are:

$$Q = \frac{(P-Ia)^2}{P-Ia+S} \qquad \text{for } P \geq Ia \qquad (2.9)$$

$$Q = 0 \qquad \text{for } P \leq Ia \qquad (2.10)$$

where Q is the accumulated runoff (mm),

P is the accumulated precipitation (mm),

Ia is the initial abstraction, represents all initial losses (depression storage, interception and infiltration during the filling of depression storage) that occur prior to the time when runoff begins (mm), and

S is the total soil moisture capacity for storage of water (mm) and is related to pre-storm and watershed characteristics.

To eliminate the necessity of estimating both variables, *Ia* and *S*, an empirical relationship was derived using records of rainfall and runoff from experimental watersheds, as follows:

$$Ia = 0.2\,S \qquad (2.11)$$

Also the curve number (*CN*) and soil moisture capacity (*S*) are related by the following equation:

$$S = \frac{1000}{CN} - 10 \qquad (2.12)$$

Therefore, substituting Equations 2.11 and 2.12 into Equation 2.9, Q can be derived as:

$$Q = \frac{(P + 2 - \frac{200}{CN})^2}{(P + 8 + \frac{800}{CN})} \qquad (2.13)$$

Therefore, if CN is known for a catchment, Q can be computed for known rainfall event (P). CN values have been published for most of the regions in the United States based on extensive field studies. However, no such data are available for Australian catchments.

Aitken (1973) reported that the method had given poor predictions for runoff events in Australian catchments with bias towards overestimating the runoff volume, and that the prediction was very sensitive to the selection of CN. Pilgrim (1989) concluded that in the absence of locally derived CN values, the method could not be recommended as a satisfactory design method for Australian catchments.

2.4.1.3 Evaporation loss

For modelling of individual storm events, the evaporation is relatively unimportant. However, for long-term analysis of the urban water budget (through long-term simulation of urban catchments), the evaporation is just as important as for rural catchments (Bedient and Huber, 1992).

2.4.2 Overland Flow Modelling

Overland flow is a major process in simulating runoff hydrographs of an urban drainage system. Overland flow deals with both impervious and pervious areas. However, the overland flow is not modelled in all urban drainage models. Instead, the rainfall excess and routing are lumped together and the inflow runoff hydrograph is computed at a pit without considering overland flow routing. Examples are the Rational formula method and the SCS method. In the models where overland flow is modelled explicitly, the rainfall excess is computed first and then this rainfall excess is routed over catchment surface into a pit. Different approaches have been used to model the overland flow component in these urban catchment models. These methods are applied for both pervious and impervious areas. They include:

- Time-area routing with linear time-area diagram,
- Linear and nonlinear reservoir representation, and

- Muskingum routing approach.

2.4.2.1 Time-area method

The time-area method is used in ILSAX (O'Loughlin, 1993) and STORM (U.S. Army Corps of Engineers, 1977) models for overland flow routing. The details of this method are given in Section 4.2.1. Therefore, the details are not repeated here.

2.4.2.2 Linear and nonlinear reservoir representation

The overland flow component over catchment surface can be represented by linear or nonlinear reservoirs. Nonlinear reservoir model is a two-parameter (n, K) model, based on successive storage routing without translation among storages. The model uses storage and continuity equations, as follows.

Storage Equation: $$S = K\, q^n \tag{2.14}$$

Continuity Equation: $$dS/dt = I - Q \tag{2.15}$$

Combining Equations 2.14 and 2.15, the overland flow routing equation is obtained (i.e. Equation 2.16).

Routing Equation: $$I - Q - n\, K\, Q^{n-1}\, (dQ/dt) = 0 \tag{2.16}$$

where
- S is the storage (m^3),
- I is the inflow (m^3/s),
- Q is the outflow (m^3/s),
- n is the number of reservoirs,
- K is the storage coefficient, and
- t is time (s) from starts of runoff.

Single linear reservoir is a special case with $n = 1$ and K equals to the time lag (hours) between hyetograph and hydrograph.

Linear or nonlinear routing is used in SWMM (U.S. Environmental Protection Agency, 1992), WALLRUS (Hydraulics Research Ltd., 1991) and RAFTS (WP Software, 1991) models for modelling overland flow routing.

2.4.2.3 Muskingum routing approach

The Muskingum method is a different representation of a nonlinear reservoir described in the previous section. The storage function for the catchment is used in the continuity equation given in Equations 2.17-2.21. This method was used in RAFTS (WP Software, 1991).

$$(i_1+i_2).\Delta t/2 - (q_1+q_2).\Delta t/2 = S_2-S_1 \qquad (2.17)$$

where i_1, i_2 are inflows at beginning and end of routing period (m^3/s),

Δt is routing interval (h),

q_1, q_2 are outflows from the storage at beginning and end of routing period (m^3/s), and

S_1, S_2 are storages volume at beginning and end of routing period ($hrs.m^3/s$).

Substituting S_2 and S_1 in equations: $S_1 = k_1 q_1$ and $S_2 = k_2 q_2$ to Equation 2.17:

$$q_2 = c_0 .i_2 + c_1 i_1 + c_2 q_1 \qquad (2.18)$$

where $c_0 = c_1 = \Delta t / (2k_2+\Delta t)$ (2.19)

$c_2 = (2k_1-\Delta t) / (2k_2+\Delta t)$ (2.20)

i_1, i_2 are inflows at beginning and end of routing period (m^3/s),

Δt is routing interval (h),

k_1, k_2 are storage delay times as a function of q defined by Equation 2.21,

q_1, q_2 are outflows from the storage at beginning and end of routing period

(m^3/s), and

S_1, S_2 are storages volume at beginning and end of routing period (hrs.m^3/s).

An iterative solution to Equation 2.18 is required to the interrelation between c_o, c_1, c_2, k_2 and q_2 in RAFTS the Newton Raphson iteration is used. K_1 and K_2 defined via Equation 2.21.

$$k = B\, q^n \tag{2.21}$$

2.4.3 Modelling of Pipe and Channel Flows

Methods that have been used to model pipe and channel flow include unsteady flow models, steady flow models, time-lag method, linear and nonlinear reservoir routing, and applying Muskingum routing parameters derived from recorded hydrographs. The theory of the latter two methods for pipe/channel routing is the same as for catchment overland flow routing described in Sections 2.4.2.2 and 2.4.2.3.

2.4.3.1 Unsteady flow models

This method uses different simplifications of the complete dynamic equation (i.e. Saint-Venant equation) of flow, as shown in Equation 2.22.

$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x}\left(\frac{Q^2}{A}\right) + gA\frac{\partial Y}{\partial x} + gAS_f - gAS_o = 0 \tag{2.22}$$

where t is the time,

x is the longitudinal direction measured horizontally,

A is the flow cross-sectional area normal to x,

Q is the discharge through A,

Y is the depth of flow,

S_o is the channel slope,

S_f is the friction slope, and

g is the gravitational acceleration.

In Equation 2.22, the first term, known as the local acceleration term, is important, when flow is highly unsteady. The second term, the convective acceleration term, is important when the flow changes rapidly with respect to space (i.e. rapidly changing water surface over the length). The third, the pressure term, is important when there is a pressure gradient caused by changing depth of nonuniform flow. The fourth term represents the resistance due to bed friction. The last term denotes the gravity driven force due to bed slope. The Kinematic wave equation considers only the last two terms of Equation 2.22.

In the last few decades, researchers have extensively used the finite difference technique (Chiang and Bedient, 1986) to solve the full dynamic Saint-Venant equation in urban runoff models. Although this approach provides very accurate hydraulic simulations, it requires extensive input data and involves excessive computer time to generate the results. Kinematic or full dynamic equation can be used with SWMM (U.S. Environmental Protection Agency, 1992), DR3M (Alley and Smith, 1990), and WALLRUS (Hydraulics Research Ltd., 1991) models.

2.4.3.2 Steady flow models

A widely used steady flow model in urban drainage computer models is the Manning's equation. The ILSAX (O'Loughlin, 1993) model used this method for pipe routing.

2.4.3.3 Time-lag method

In this method, the lag time is computed as time taken for the maximum discharge to travel the pipe/channel reach from upstream to downstream. The upstream hydrograph is then lagged by this travel time to obtain the downstream hydrograph. This method does not consider attenuation effects due to storage in the pipe/channel and therefore, the hydrograph shape and peak are the same for upstream and downstream hydrographs. The time-lag method can be used in the ILSAX model for pipe routing, as one of the options.

2.4.4 Modelling of Flow through Storages

Most urban drainage models use modified Puls and Muskingum routing methods to model flow through storages. These methods are based on the continuity equation and a relationship linking outflow rates from the storage to various water levels in the storage. A description of these methods can be found in any standard hydraulics or hydrology textbook (e.g. Chow, 1973; Mutreja, 1990).

2.5 LEVEL OF ACCURACY IN URBAN CATCHMENT MODELS

Very little literature is available on the level of accuracy of urban drainage computer models. Several comparative studies on these models were found, but the level of accuracy of these models was not presented in most of these studies. The main purpose of this section is to review studies that compare urban catchment models and to quantify the level of accuracy associated with each model. Although the level of accuracy depends on each individual process of the rainfall-runoff chain, only the hydrographs at the outlets were considered in the review, since outputs in relation to each of these processes were not available.

Two groups of comparative studies were found in the literature in relation to urban drainage computer models. The first group (called calibrated studies in this review) covers studies, where the models were calibrated for a catchment and then verified using independent data sets for the same catchment. The second group (referred to as non-calibrated studies in this review) covers studies, where the models were not calibrated and the parameter values were assumed or taken from other sources, or calibration and verification of models were not clearly explained in the literature.

Section 2.5.1 and 2.5.2 review the calibrated and non-calibrated studies respectively in a qualitative manner. Section 2.5.3 presents a quantitative assessment of the level of accuracy associated with urban drainage models conducted by the author using the results of calibrated and non-calibrated studies.

2.5.1 Calibrated Studies

2.5.1.1 Kidd (1978a) study

In Kidd (1978a), alternative optimisation strategies and objective functions, different loss models and different surface routing models were considered. Data from the Netherlands, Sweden and UK consisting of 188 rainfall-runoff events on 18 urban catchments were used. The catchment areas varied from 78 m^2 to 20,000 m^2, and the impervious fraction of subcatchments varied from 50% to 100%.

Comparison of Objective Functions

Six different objective functions were investigated in Kidd's study. They are given in Table 2.2. These objective functions can be used to calibrate model parameters under different objectives. ISE and BISE deal with the overall shape of the hydrograph, while VOL considers only the runoff volume. PISE can be used to optimise model parameters matching observed and modelled hydrograph ordinates close to peak discharge without considering small discharges. The other objective functions consider peak and time to peak discharge.

Table 2.2: Six Different Objective Functions Used in Kidd's (1978a) Study

Objective Function	Formula
Integral Square Error (ISE)	$\text{ISE}=\left\{\sqrt{\sum(Q_o - Q_m)^2} / \sum Q_o\right\}\times 100\%$
Biased Integral Square Error (BISE)	$\text{BISE}=\left\{\sqrt{\sum(Q^2{}_o - Q^2{}_m)} / \sum Q_o\right\}\times 100\%$

Error in Peak Estimate (PEAK)	PEAK= $\{(P_o - P_m) / P_o\} \times 100\%$
Error in Predicted Volume (VOL)	VOL= $\{(V_o - V_m) / V_o\} \times 100\%$
Time to Peak (TTP)	TTP= $\{TP_o - TP_m\}$
Partial Integral Square Error (PISE)	PISE= $\left\{\sqrt{\sum (Q_o - Q_m)^2} / \sum Q_o\right\} \times 100\%$ For $Q_o > P_o /2$

where Q_o is the observed discharge ordinate,
Q_m is the modelled discharge ordinate,
P_o is the observed peak discharge,
P_m is the modelled peak discharge,
V_o is the observed runoff volume,
V_m is the modelled runoff volume,
TP_o is the observed time to peak, and
TP_m is the modelled time to peak.

The constant proportional loss model and the Nash Cascade surface routing model were used for three catchments (out of 18 catchments referred in the study) in this part of the comparison. In the constant proportional loss model, the depression storage (DEPSTO) is first subtracted, then the remaining losses distributed as a fixed proportion of the remaining rainfall period. This is a two parameter (i.e. DEPSTO and the runoff coefficient) model. Nash Cascade surface routing model is also a two parameter (i.e. n and K) model. In this model, storage routing is performed using n equal linear reservoirs each with routing constant equal to K. Kidd (1978a) discussed these models in details. Seven storm events were considered for each of the three catchments and model parameters estimated in two ways.

a) A single 'best' parameter set considering all events to satisfy the particular objective function.
b) A parameter set for each event. These parameter sets were then used to produce an average parameter set.

Thus for each catchment, eight sets of parameter values (7 sets for seven events including the single 'best' parameter set considering all events and 1 set for average) were obtained for each objective function. By using each of these eight parameter sets, the hydrographs were simulated for different storm events which were not used in calibration (3, 4 and 4 events for the three catchments) as verification events and the values of objective (error) functions computed for each event. Each parameter set was ranked according to the value obtained for each error function for an event. Since the results varied from one storm to another, the ranks for error functions were summed for each catchment in Kidd's study, and the best objective functions were determined from these summed values.

Based on the results of verification, the PISE 'best' parameter set had given the best result a greater number of times, with the BISE 'best' parameter set the second, the ISE 'best' parameter set the third, and the ISE average set the fourth.

Comparison of Loss Models

Three different loss models (i.e. phi-index, constant proportional loss and variable proportional loss) were studied in Kidd (1978a) using an arbitrarily chosen surface routing model (i.e. nonlinear reservoir which has two parameters, n and K). These loss models were discussed in details in Kidd (1978a). The parameters of the loss models are given in Table 2.3.

Table 2.3: Parameters of selected loss models
(Kidd, 1978a)

Model Name	Number of Parameters	Parameters
Constant Proportional Loss (Kidd, 1978a)	2	DEPSTO, PC
Phi-index (Kidd, 1978a)	2	DEPSTO, PHI

Variable Proportional Loss (Kidd, 1978a)	3	ALPHA, Z_o, Z_E

where DEPSTO is the depression storage of pervious area,
PC is the runoff coefficient,
PHI is the constant rate of loss value,
Z_o is the fraction of loss at start of storm,
Z_E is the fraction of loss at end of storm, and
ALPHA is the constant.

Three catchments were selected for comparison of loss models. Kidd (1978a) does not say whether these three catchments were same as in the study of 'comparison of objective functions'. These catchments had an only single inlet (pit). Therefore, the runoff from these catchments represented only overland flow. The value of depression storage (DEPSTO) was found from a previously derived regression equation. PC, PHI, ALPHA, Z_o and Z_E values were estimated by equalling the volume of net rainfall and the volume of runoff. The exponent (n) of the nonlinear reservoir routing model ($S = K .Q^n$) was fixed at 0.67. For each catchment, the parameter K was optimised for each loss model considering a number of storm events. The average K was then estimated for each catchment. This value of K was used to compute the ISE objective (error) function for ten independent events of the three catchments for different loss models.

Based on the results of the study, it was found that the Phi-index loss model was significantly inferior to the other two loss models in terms of runoff volume. No significant difference could be observed between constant proportional and variable proportional loss models.

Comparison of Overland Flow Routing Models

Seven surface routing models (i.e. linear reservoir, nonlinear reservoir, nonlinear with time lag, Nash cascade, Muskingum, time of entry and unit hydrograph) were compared in this study. These routing models were discussed in Section 2.4.2 and the previous section of this thesis, except the unit hydrograph models. The unit hydrograph method assumes that the catchment response is linear. The hydrograph ordinates for any rainfall depth can be computed by multiplying the unit hydrograph ordinates by rainfall depth.

Three catchments were initially used to study sensitivity of overland flow routing model parameters. Kidd (1978a) does not say whether these three catchments were same as in the previous two comparisons. The overland flow routing models had one to three model parameters. The constant proportional loss model was used and the loss parameters were derived for each storm by matching the runoff volume of modelled and observed hydrographs. The sensitivity of overland flow routing model parameters was studied and the most sensitive parameter was identified for each overland flow routing model.

Fourteen catchments of the eighteen catchments were selected to calibrate the above most sensitive parameter, while the other routing parameters were set at reasonable values. As earlier, the constant proportional loss model was used and the loss parameters were derived for each storm by matching the runoff volume of modelled and observed hydrographs. The most sensitive parameter of each model was then optimised for the 14 catchments. These most sensitive parameters were then used in a regional study to correlate them with catchment characteristics.

The remaining four catchments were then considered and the most sensitive parameters of the overland flow routing models were determined from the above regional expressions. As earlier, the constant proportional loss model was used and the loss parameters were derived for each storm by matching the runoff volume of modelled and observed hydrographs. The value of the objective (error) function was computed for each of these catchments to compare the routing models. A general conclusion drawn from the analysis was that the nonlinear models performed better than the linear ones.

2.5.1.2 Heeps and Mein (1973a,b) study

Three urban catchment models, the Road Research Laboratory model - RRL (Terstriep and Stall, 1969), the University of Cincinnati Urban Runoff Model - UCUR (Papadakis and Preul, 1972) and the Storm Water Management model - SWMM (U.S. Environmental

Protection Agency, 1992) were applied to an urban catchment in Australia (Heeps, 1973, Heeps and Mein, 1973a,b). The catchment considered was Yarralumla Creek at Mawson in Canberra. The RRL model is an early version of the ILSAX model (O'Loughlin, 1993). The main difference between RRL and ILSAX is that RRL ignores pervious areas. Interception, depression storage and evaporation are likewise omitted. Gutter flow is not permitted in RRL. The UCUR model routes the flows overland, and through gutters and pipes using continuity and Manning's resistance equation. Infiltration is accounted for using Horton's equation and surface retention is related to depression storage using an exponential equation. The drainage area is divided into subcatchments. Starting with overland flow, the excess rainfall is routed through successive components of the drainage system.

Initially, a qualitative comparison was made on the ability of each model to predict the outflow hydrograph using data on several storms. The degree of subdivision of the catchment required by each model was also examined. Three subdivisions were considered namely, coarse subdivision (neglecting all lateral drains and considering each subcatchment to contribute directly to the inlets of the main drain), medium subdivision (considering first order lateral drains as well as the main drain) and fine subdivision (considering all drains).

No data were available for infiltration and depression storage of this catchment, and therefore the infiltration and depression storage parameters were estimated by calibrating the models using one storm event. These values were then used with five other storm events to compare the models. The modelled hydrographs were visually compared with the observed hydrographs in the assessment.

The results of Heeps and Mein study showed that SWMM consistently produced the best results. Furthermore, the RRL method was found to be inadequate. From the results of five independent storms that were not used in the calibration, the runoff volume was underestimated by RRL and overestimated by UCUR. SWMM also overestimated the runoff volume except in one event. Underestimation of runoff volume in RRL may be due to neglect of the pervious area runoff. All models gave the highest error in peak runoff compared to runoff volume and time to peak discharge.

It was also found in Heeps and Mein study that the degree of subdivision of the catchment had a significant influence on the peak discharge predicted by each of the models. However, no recommendations were given regarding the best subdivision to be used in design and analysis.

2.5.1.3 Dayaratne (1996) study

Dayaratne (1996) studied the accuracy of ILSAX, SWMM and Deterministic Rational method (DRM) using different calibration and verification storm events. The Giralang catchment in Canberra was used in the study. The calibration of the ILSAX and DRM model parameters was done by trial and error. The SWMM model parameter calibration was done through optimisation using the Pattern Search (Monro, 1971) technique.

The model parameters estimated from calibration yielded different results for different storm events showing storm dependency on model parameters. The calibration also produced different parameter values for different objective functions.

Analysis of ten observed storms events during calibration and verification, indicated that both the ILSAX and SWMM models gave higher errors in simulating runoff volume, peak discharge and time to peak discharge for ‘small’ rainfall events (< 1 year ARI) compared to medium events. For medium size rainfall storm events (2-10 year ARI), both these models were equally good in simulating peak discharge and time to peak discharge but giving relatively high errors in simulating runoff volume. Generally, RFM model gave the highest error in peak discharge compared to SWMM and ILSAX.

2.5.1.4 Other studies

Black and Aitken (1977) used the twenty-one parameter Australian Representative Basin Model - ARBM (Australian Water Resources Council, 1973) and the sixteen parameter Hydrocomp Simulation Programming - HSPF (Johanson *et al.*, 1984) model to simulate the behaviour of two urban catchments in Australia. The catchments considered were

Yarralumla Creek at Mawson (Canberra) and Vine Street (Melbourne). ARBM models urban catchments as well as rural catchments. The pervious and impervious areas are considered separately. Nineteen parameters are used to represent the catchment characteristics and each of these parameters has a physical significance. The nonlinear runoff routing method developed by Laurenson (1964) was used to route the combined surface runoff from impervious and pervious areas. Two extra parameters are required for this routing method. HSPF was described briefly in Section 2.3.4.7.

These two models were calibrated with observed hydrographs at the catchment outlets and tested using independent rainfall-runoff data sets. The results of flood peak estimates of these two models were shown as scatter diagrams for both calibration and verification (Black and Aitken, 1977). Thirty eight and forty events from Yarralumla Creek and Vine Street catchments respectively were selected for verification. For calibration events, the ARBM results appeared to be slightly better than those of HSPF, which consistently over-estimated the peak of small events. In the verification, ARBM had a tendency to under-estimate flood peaks frequently while the HSPF continued to over-estimate peaks of small events and under-estimate the peaks of large events (Black and Aitken, 1977). Based on statistics on 78 storm events, the maximum errors in flood peak by ARBM and HSPF models were -2.1% and +4.2% respectively. For the runoff volume, the maximum errors were +26.3% and +27.4% respectively. In general, both models gave accurate flood peaks and satisfactory runoff volumes. The number of parameters in both models was large and therefore considerable effort had to be taken for model calibration.

Zech *et al.* (1994) applied the Digital Terrain Model - DTM (Grandjean and Zech, 1991) for a partly urbanised catchment. The results of the DTM model were compared with those of SWMM (U.S. Environmental Protection Agency, 1992) and WALLRUS (Hydraulics Research Ltd., 1991). It allows the use of Geographic Information System (GIS) to manipulate input data and output results, and display the data and results on catchment maps. SWMM was discussed briefly in Section 2.3.3.1. WALLRUS is an event model. Loss models used in WALLRUS are the runoff coefficient and SCS methods. Nonlinear reservoir routing is used to route overland flow, while pipe routing is done by the Muskingum method. WALLRUS can simulate the surcharging and pressurised flows. The calibration procedure for each model was not discussed details in Zech *et al.* (1994).

Visual comparison of simulated and observed hydrographs of these models for two verification storm events showed excellent agreement with observed hydrographs.

Naidu and Kearney (1995) applied ILSAX to a catchment in the Central Business District of Auckland (catchment area is 170 ha with 90% impervious) to identify the most effective drainage strategy to meet current and future development needs. Naidu and Kearney (1995) stated that the model was calibrated, but the details were not given. Three independent events were then considered and the results compared with observed hydrographs. The results showed that the ILSAX model produced accurate results for peak flows within the error of -7.3% to +9.0%. This study concluded that the ILSAX model were adequate for modelling of steep upper catchment.

Maheepala (1999) used ILSAX and SWMM models to study the flood mitigation measures of the Timbertop Estate catchment in City of Knox, Victoria. In this study, the rainfall and runoff data of three storm events were used to calibrate the parameters of the two models. The percentage impervious area of the catchment and the depression storage were considered as calibration parameters. Parameter sets were computed for each storm event and the average values from the three parameter sets were used as the catchment parameters. Three additional storm events were used for testing of calibrated model parameters. Maheepala (1999) found that both models were equally good in simulating runoff volume and time to peak discharge. However, some differences were found for peak discharges of two events (out of three) from both models.

2.5.2 Non-Calibrated Studies

Terstriep and Stall (1969) used the RRL model and the Chicago (Tholin and Keifer, 1960) method on three urban catchments. The simulated and observed hydrographs at the catchment outlet were compared for 39 storm events. Both models had simulated peak discharge, time to peak discharge and runoff volume satisfactorily.

Papadakis and Preul (1972) tested the Chicago method, UCUR and SWMM in three case studies. By using suitable parameter values from the available manuals, the runoff hydrographs were computed. The results showed that UCUR and SWMM produced almost the same results, but the Chicago method gave lower peak and lower recession limb than the other two. The times to peak discharge from the three methods were the same.

Aitken (1975) applied the Deterministic Rational method (DRM), the Statistical Rational method (SRM), RRL and the Laurenson Runoff Routing model (LRR) to several gauged urban catchments in Sydney, Melbourne, Brisbane, Adelaide and Canberra. The models were already discussed previously except the LRR model, which is almost similar to RORB. From the results of this study, it was found that the DRM and RRL methods were satisfactory to compute peak discharge for sewer systems of relatively small areas and that the error range for peak discharge was -19% to +21%.

Aitken (1975) reviewed each model and the following conclusions were made:

a) the SRM was shown to have considerable merit when applied to data for Melbourne, but results were inaccurate in the case of high rainfall intensity or when applied to large catchments,
b) the RRL was found to be a very convenient and accurate design tool where design rainfalls were of comparatively low intensities,
c) for large urban catchments where the areal variation of catchment rainfall was significant, it was shown that the LRR model reproduced the observed hydrographs satisfactorily, and
d) a mathematical model of the rainfall-runoff process was required for urban catchments in Australia.

Vale *et al.* (1986) applied the ILSAX and SWMM models for Fishers Ghost Creek and the Bunnerong stormwater channel gauge catchment at Morouba (Australia). The major objective of the study was to assess the accuracy of SWMM in modelling runoff and to determine suitability of SWMM as a general model for stormwater drainage system design and analysis. ILSAX was taken as the benchmark. The SWMM model was applied to 13 storm events of Fishers Ghost Creek catchment, while both SWMM and ILSAX models were applied to 12 storm events of Bunnerong stormwater channel. Both models were not calibrated. Depression storages were taken as 1 mm for impervious areas and 5 mm for pervious areas. The simpler Horton model was used to model infiltration loss, using infiltration curves developed by Terstriep and Stall (1974) for ILLUDAS (Terstriep and Stall, 1974). Appropriate infiltration curves were selected for the soil types of the catchments and the antecedent moisture conditions. Antecedent moisture conditions were estimated using the total rainfall for the 5 days preceding the storm as given in the ILSAX manual (O'Loughlin, 1993). The Bunnerong stormwater channel results showed marked differences between calculated and recorded runoff volumes and peak flow rates from both models. From the comparison of hydrographs of the Bunnerong stormwater channel, it was seen that the ILSAX results were better than those of SWMM.

The Pressurised ILLUDAS Backwater Simulator - PIBS (Chiang and Bedient, 1986) is a modified version of the original ILLUDAS model. PIBS can model both surcharged conditions in pipes and variable tailwater levels. A comparison was made by Chiang and Bedient (1986) between PIBS and EXTRAN (of SWMM). A 53-piped system, which drains the 0.77 km^2 area of Texas Medical Centre, was selected for the comparison. Simulated hydrographs from PIBS and EXTRAN showed that there was good agreement between the simulated outflow hydrographs, which meant that the capabilities of both PIBS and EXTRAN in term of modelling both surcharge and variable tail water conditions are equal.

Kemp (1994) applied RAFTS (WP Software, 1991), RORB (Laurenson and Mein, 1990) and ILSAX models to two urban catchments, namely Algate Creek and Frederick Street catchments in Australia to examine the performance of these three models. The results showed that all models showed marked differences between recorded and calculated runoff volumes. This suggests that there was either an error in rainfall and/or flow measurements, or an error in simulating the rainfall excess in models. No model was superior to the others.

Ball (1987a) compared the predicted hydrographs from ILSAX, SWMM (EXTRAN block) and PIPENET (Ball, 1987b) models, which use different techniques for modelling hydraulics of free surface and pressurised flow in pipe networks. PIPENET is similar to SWMM, but deals only with the hydraulic component of an urban drainage system. It was developed to determine the propagation of hydrograph through an urban drainage network consisting of pipes and manholes. Three hypothetical pipe networks were considered under conditions of both pressurised and free surface flow regimes. Discharge hydrographs predicted by each model at the downstream end of each pipe network were computed using assumed inflow hydrographs at the upstream inlets. Results of the study showed that a truncation of the discharge hydrograph occurred whenever ILSAX was used to route pressurised flows. This effect was not shown in SWMM or PIPENET, as they explicitly modelled the pressurised flow conditions. Additionally, the ILSAX model did not model flow reversals, while the other two models were capable of simulating this effect.

2.5.3 Quantification of Model Accuracy

As a part of this review, an attempt was made by the author to quantify the model accuracy by reviewing the comparative studies published in the literature in simulating the event runoff hydrographs of urban catchments. For most of these studies, the plots of simulated and observed hydrographs were given, but no quantification of model errors was done. For these studies, model simulation errors were quantified by measuring the difference between observed and simulated hydrographs. Results of all comparative studies were analysed with three error functions (i.e. Equations 2.23-2.25) relating the differences between observed and simulated hydrograph peak, volume and time to peak. These three attributes are important in design and analysis of urban drainage systems. Peak discharge is required in urban drainage design for sizing pipes, culvert and bridges. Runoff volume is required for design and operation of flood control structures such as retarding basins. Time to peak discharge is required for flood forecasting and operation of control structures during storm events.

Error in runoff volume (VOL),

$$VOL = \{(V_m - V_o) / V_o\} \times 100 \quad (2.23)$$

Error in peak discharge (PEAK),

$$PEAK = \{(P_m - P_o) / P_o\} \times 100 \quad (2.24)$$

Error in time to peak discharge (TTP),

$$TTP = \{(TP_m - TP_o) / TP_o\} \times 100 \qquad (2.25)$$

where P_o is the observed peak discharge,

P_m is the modelled peak discharge,

V_o is the observed runoff volume,

V_m is the modelled runoff volume,

TP_o is the observed time to peak, and

TP_m is the modelled time to peak.

Equations 2.23-2.25 were used to compute the model errors for both calibrated and non-calibrated studies reviewed in Sections 2.5.1 and 2.5.2 respectively. The results are shown in Tables 2.4 and 2.5 respectively. These tables show the catchment characteristics (i.e. area and percent imperviousness), number of events used in calibration and verification, and the error range arising from different events.

Table 2.4: Summary of Percentage Errors for Calibrated Studies

Model or Method	Cat. Area (ha.)	Imp. %	No. of Eve.	% Error			Reference
				Volume	Time to Peak	Peak Discharge	
ILSAX	94	24	10	1 to –143	0 to 100	-7 to 100	Dayaratne (1996)
	41	37	1	19	1	19	Maheepala (1999)
	9	50	1	29	1	36	Maheepala (1999)
	14	47	3	26 to 57	0 to 1	4 to 73	Maheepala (1999)
RRL	502	20	5	-53 to -21	N/A	-64 to +38	Heeps and Mein (1973a)
SWMM	502	20	5	-21 to +64	N/A	-53 to +128	Heeps and Mein (1973a)
	15	N/A	2	-20 to -16	0 to +5	-25 to -22	Zech *et al.* (1994)
	94	24	10	6 to 99	0 to -50	1 to -108	Dayaratne (1996)
	41	37	1	23	2	13	Maheepala (1999)
	9	50	1	29	2	15	Maheepala (1999)
	14	47	3	26 to 57	0 to 1	3 to 32	Maheepala (1999)
UCUR Model	502	20	5	+45.0 to +134	N/A	-48.0 to +66	Heeps and Mein (1973a)
ARBM	502	20	78	8 to 26	N/A	0 to -2	Black and Aitken (1977)
HSP	502	20	78	4 to 27	N/A	0 to 4	Black and Aitken (1977)
	94	24	10	N/A	N/A	0 to -120	Dayaratne (1996)
DTM	15	N/A	3	-2 to +6	0.0 to +2.0	-2 to +2	Zech *et al.* (1994)
WALLRUS	15	N/A	1	-28	+10	-30	Zech *et al.* (1994)

Key to Acronyms for Tables 2.4

Cat. Area (ha.)	Catchment area in hectares
Imp.	Impervious area as a percentage of total catchment area
No of Eve.	Number of storm events considered in study
N/A	Not available or not applicable

The results of the study on level of accuracy of computer models showed a large variability, which does not permit any conclusions to be drawn. However, in general the time to peak has been modelled better than the peak and runoff volume. The routing parameters such as pipe roughness, gutter factors and pit capacity parameters affect the time to peak discharge, which suggests that the routing parameters had been estimated fairly accurately in those studies; these parameters in general have the least variability. The accuracy of time to peak discharge depends on the computational time step used in the analysis. If short time step is used, the accuracy will be more. The results also showed that the runoff peak and volume had larger errors, which suggests that the rainfall excess may not have been calculated correctly. The impervious and pervious area parameters such as depression storages, antecedent moisture content and soil curve number, affect the rainfall excess. These parameters in general have the highest variability. Therefore, the study on level accuracy suggests that it is necessary to estimate these impervious and pervious parameters accurately.

Table 2.5: Summary of Percentage Errors for Non-Calibrated Studies

Model Method	Cat. Area (ha.)	Imp. %	No. of Eve.	% error			Reference
				Volume	Time to peak Discharge	Peak Discharge	
RRL	32	45	1	+18	+6	+21	Terstriep and Stall (1969)
	1	100	10	N/A	N/A	-17 to +20	Terstriep and Stall (1969)
	916	44	28	N/A	N/A	-27 to +17	Terstriep and Stall (1969)
	5	45	2	-27 to -22	+4 to +19	-4 to +24	Papadakis and Preul (1972)
	70	36	5	-51 to +103	N/A	-9 to +87	Heeps and Mein (1975a)
	70	36	16	N/A	N/A	-97 to +60	Aitken (1975)
	5 No.	N/A	271	N/A	N/A	+1 to +6	Aitken (1973)
ILSAX	57	55	12	+83 to +414	N/A	+33 to +156	Vale *et al.* (1986)
	57	55	12	61 to 320	N/A	17 to 159	Vale *et al.* (1986)
	5	45	1	-14	+25	-11	Papadakis and Preul (1972)
	960	55	3	-48 to -22	-40 to +118	-38 to 0	Papadakis and Preul (1972)
	70	36	5	+2 to +149	N/A	+1 to +107	Heeps and Mein (1973a)
Chicago	32	45	1	+36	-6	+21	Terstriep and Stall (1969)
Method	5	45	1	+18	-4	+4	Papadakis and Preul (1972)
UCUR	5	45	2	-27 to -11	-4 to +31	-7 to +4	Papadakis and Preul (1972)
Model	960	55	3	-32 to +1	-18 to +4	-20 to +4	Papadakis and Preul (1972)
	70	36	5	+39 to +249	N/A	+19 to +203	Heeps and Mein (1973a)
RFD	70	36	16	N/A	N/A	-131 to +56	Aitken (1975)
	5 No.	N/A	271	N/A	N/A	-21 to +19	Aitken (1975)

<u>Key to Acronyms for Tables 2.5</u>

Cat. No.	Catchment number
Cat. Area (ha.)	Catchment area in hectares
Imp.	Impervious area as a percentage of total catchment area
No of Eve.	Number of storm events considered in study
N/A	Not available or not applicable
(quantity) no.	Number of catchments considered

The inaccuracies in runoff peak and volume are higher for ungauged catchments compared to the gauged. Therefore, the review also highlights the necessity for guidelines or improved methods for the application of urban drainage models for ungauged catchments in simulating of peak discharge and runoff volume. Some studies in this review showed that the model error depends on storm characteristics and the land use conditions of catchments.

2.6 SUMMARY

The main parts of an urban drainage system consist of property drainage, street drainage, trunk drainage and major water receiving bodies. In most drainage systems, retention and detention basins are also used for flood control and water quality improvement. Several urban drainage models have been developed to simulate the rainfall-runoff process of urban drainage systems. The major components of these models include the modelling of rainfall excess, overland flow routing and pipe routing. Different models use different methods to model these components.

Most drainage models calculate rainfall excess using hydrologic methods and this rainfall excess is then routed through the pipe system and other system components using hydraulic methods. However, there are other models where hydrology and hydraulics of the system are lumped together in computing flood hydrographs and/or peak discharges. The choice between the two types of models depends on the type of the catchment to be modelled, the availability of catchment data, the level of complexity and sophistication required in the simulation of the catchment runoff response and time available for the analysis.

It is important to know the level of accuracy obtained from various urban drainage models, before embarking on an urban drainage study. However, no information is available on the level of accuracy of these models. Several individual comparative studies have been found in the literature, but quantification of the level of accuracy of these models has not been done in most of these studies. An attempt was made in this project to study the level of accuracy of these models and found a large variability in model errors.

Due to large variability of the errors found in this study, it was difficult to draw specific conclusions on superiority of one model over another. In general, the errors in time to peak discharge seemed to be less compared to those of peak discharge and runoff volume, which suggests that the routing processes have been modelled reasonably well in those studies. The routing parameters in general have the least variability. The rainfall excess may not have been modelled accurately and these parameters in general have the highest variability.

Some studies in this review also showed that the model error was dependent on storm characteristics and land use conditions of catchments. The inaccuracies in runoff peak and volume were higher for ungauged catchments compared to the gauged. Therefore, the review also highlighted the necessity for guidelines or improved methods for application of urban drainage models for ungauged catchments in simulating of peak discharge and runoff volume.

CHAPTER 3

CURRENT URBAN DRAINAGE DESIGN AND ANALYSIS PRACTICE IN VICTORIA

3.1 INTRODUCTION

The average annual expenditure in Australia for urban drainage works is approximately 52 million Australian dollars (Maidment, 1993) as at 1988. Design of drainage works affects the cost and therefore, it is important to estimate the flood magnitudes accurately for use in design. There are several methods available to determine these flood magnitudes. The development of these methods is continuously in progress. In 200 years of European settlement in Australia, the design methods for drainage systems have changed from rule-of-thumb methods to standardised and sophisticated methods (including computer models) based on experiments and research. The evolution of these design methods is as follows (O'Loughlin and Joliffe, 1987).

Up to 1845	rule-of-thumb methods
1845-1935	empirical equations
1935-1985	the Rational method
Post 1985	computer models

As stated in Section 2.1, recent emphasis is on total management of stormwater instead of the quick disposal method practised earlier. The total management of stormwater requires an innovative conceptual design of the system consisting of rainwater tanks, wetlands etc. However, the design of the drainage components requires the estimation of flows through above methods.

The Rational method and the widely used urban drainage computer models were discussed in Section 2.3. As stated in Section 2.3, the Rational method is still widely used as the preferred method both in Australia and overseas for pipe system design instead of the more advanced computer methods. In the last few decades, some of the important developments in urban drainage design are (O'Loughlin and Joliffe, 1987):

- pipe and channel friction equations,
- description of hydraulic behaviour of pits, culvert and other drainage system components, and
- modelling of flows in pipe networks by finite difference techniques.

Since there are several methods available for estimating design flows in urban drainage design and analysis, it is important to know the current methods that are being used. Therefore, a customer survey was conducted in May 1997 to investigate the current practice used by Government Authorities and consultants in Victoria (Australia) for design and analysis of urban stormwater drainage systems.

This chapter presents the details and the results of the customer survey. Then, the current methods used for model parameter estimation, design rainfall data, return periods used in design and analysis, and water quality issues are discussed based on the information obtained from the survey.

3.2 CUSTOMER SURVEY

The major objective of the customer survey was to investigate the methods that were used by practitioners (at the time of customer survey) for urban drainage design and analysis especially in relation to estimation of flows. The other objectives were to identify the problems faced by the practitioners in an attempt to address these problems in this thesis. Due to time and resource constraints, the survey was limited to Victoria.

3.2.1 Design of Questionnaire

The questionnaire consisted of two parts. The first part of the questionnaire was designed to

- identify the method used for estimating design flows,
- identify the methods used to obtain the model parameters and rainfall information,
- identify the return periods used for design and analysis, and
- understand the difficulties faced by practitioners in applying computer models.

The second part was designed to get information on

- the current practice in relation to water quality aspects of urban drainage, and
- other information/problems/comments on the methods applied in design and analysis of urban drainage systems.

A copy of the questionnaire is included in Appendix A.

3.2.2 Selected Organisations and Respondents

The questionnaire was sent to 78 city/shire councils (i.e. all city/shire councils in Victoria) and 38 consultants, who were engaged in urban stormwater drainage design and analysis. This group was considered to be a good cross-section of the organisations that deal with urban drainage design and analysis. Figure 3.1 shows the number of questionnaires sent, the number of responses received and the number of organisations engaged in urban stormwater drainage design and analysis from the group that responded to the questionnaire. In this survey, the replies from city/shire councils were higher (60%), compared to the consultants (31%). The 93% of the respondents were engaged in urban drainage design and analysis. This response is quite usual for this type of a survey in the sense that those who are engaged in activities described in the questionnaire tend to respond.

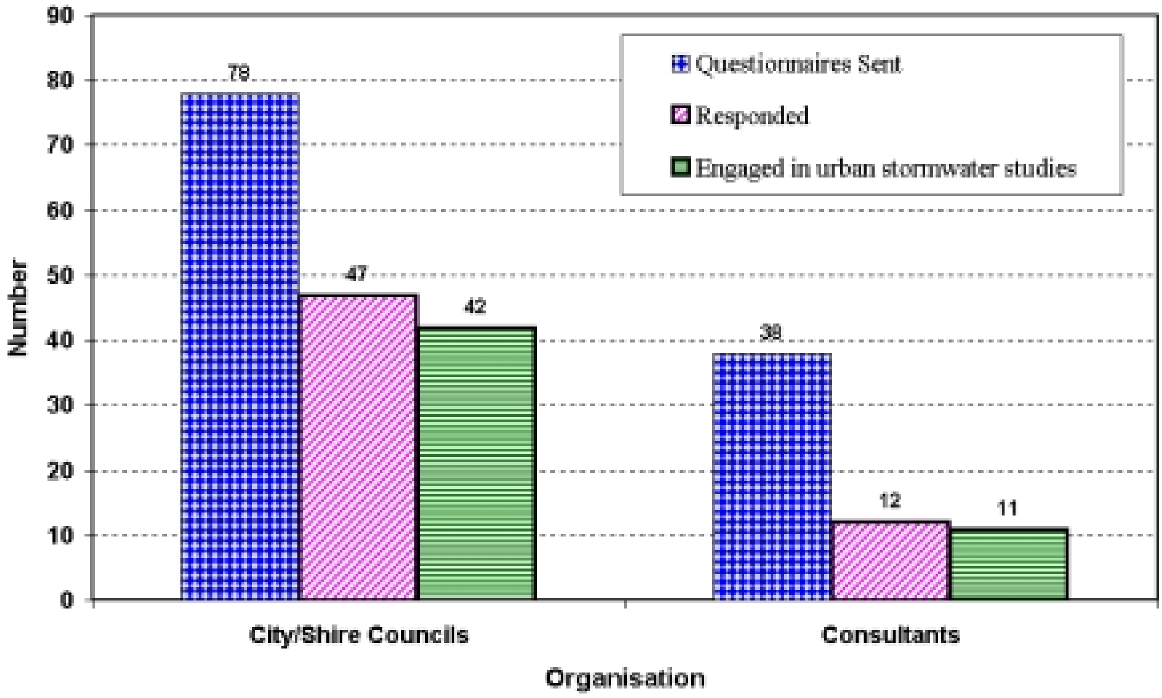


Figure 3.1: Number of Questionnaires Sent and Responses Received

3.3 RESULTS OF THE SURVEY

3.3.1 Methods and Computer Models

According to the survey results, the widely used methods and computer models used in Victoria for stormwater drainage design and analysis are Rational Formula method, ILSAX (O'Loughlin, 1993), RAFTS-XP (WP Software, 1991), RatHGL (WP Software, 1992), CIVILCAD (Surveying and Engineering Software, 1997), RORB (Laurenson and Mein, 1990), WBNM (Boyd *et al.*, 2000), Wallingford Procedure (Hall, 1984) and HEC-RAS (Hydrologic Engineering Centre, 2000). These methods and models were discussed in Section 2.3. The number of users of these models is shown in Figure 3.2. The Rational Formula method was the common method used for their design. Fifty one out of 59 respondents (86%) used the Rational Formula method. The ILSAX model was the most popular (17% of the respondents) among the respondents in relation to computer models. RAFTS, RatHGL and RORB were the other computer models used by the respondents. It should be noted, as discussed in Section 2.3, some of these models can model the pipe drainage networks, while the others are catchment models.

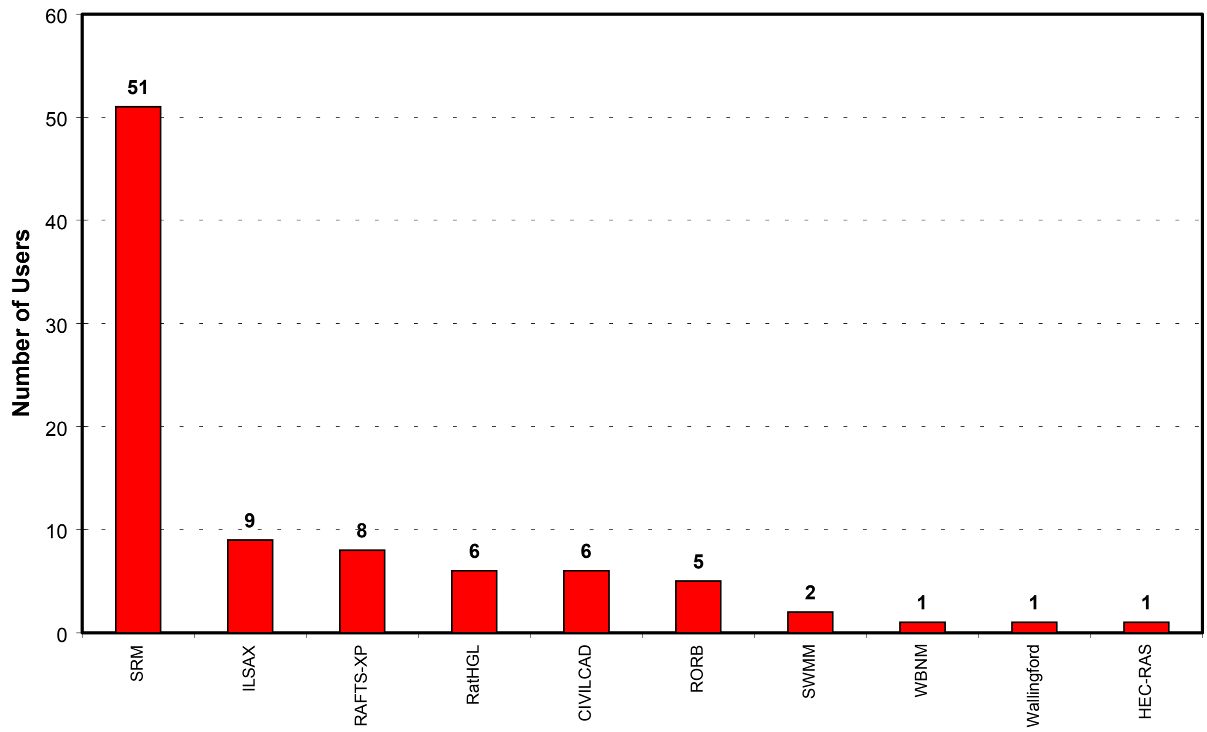


Figure 3.2: Number of Users of Urban Catchment Models

3.3.2 Rainfall Information

All councils and consultants responded in the survey deal with ungauged drainage systems (where rainfall and flow are not monitored), as it is the case with most urban catchments. They use design rainfalls as recommended in ARR87. Majority of them (78%) extracted rainfall data from the IFD curves of the site for a given storm duration and a return period, but consider an uniform temporal pattern. The reason for this is that they used the Statistical Rational method for the design and the method does not require the temporal pattern. Rest (22%) used rainfall data from the IFD curves with the temporal pattern as recommended in ARR87. This group uses computer models.

One of the important factors, which describe the catchment response, is the temporal pattern of the storm event. Ball (1992) carried out an investigation of the influence of the storm temporal patterns and made the following observations.

- The temporal pattern of rainfall excess influences the catchment response,
- The design temporal patterns presented in ARR87 result in early peaking of the runoff hydrograph compared to that obtained using a constant rate of rainfall excess,
- Of the design patterns, the 25 minutes pattern consistently predicted the highest peak flow rate and flow depth irrespective of the hydrological zone, and
- Rectangular and triangular storm patterns provide reasonable bounds to potential hydrographs arising from alternative patterns.

The analysis of (Ball, 1992) shows the importance of temporal patterns in urban drainage analysis, especially when computer models are used and also highlights the weakness of the current (ARR87) temporal patterns.

3.3.3 Model Parameters of Statistical Rational Method

The runoff coefficient and the time of concentration are the parameters of the widely used Statistical Rational method. Many relationships have been proposed relating runoff coefficients and time of concentration to factors such as land-use, surface type, slope and rainfall intensity (The Institution of Engineers, 1977, 1987; VicRoads, 1995; ACT Department of Urban Services, 1996). When runoff coefficient of a catchment increases due to urbanisation, the runoff volume increases. Furthermore, the time of concentration reduces as a result of artificial pipes and channels, which reduces the time to peak discharge causing an increase in peak discharge.

Three sources had been used by respondents to obtain the runoff coefficients for urban drainage systems in Victoria. They were ARR87, ARR77 and VicRoads charts. The ARR87 method is based on the statistical interpretation of runoff coefficients, using data from a few urban gauged catchments (The Institution of Engineers, 1987). The method relates the coefficient to the impervious fraction of the catchment, and to its rainfall climate, expressed as the 10 year ARI, 1-hour intensity. An equation is given to compute the 10 year ARI runoff coefficient (C_{10}). For return periods other than the 10 year, the runoff coefficient is computed using C_{10} and a frequency factor given in ARR87.

The ARR77 provides several curves to obtain the runoff coefficient for different land uses and storm duration. VicRoads charts are graphical representation of the ARR87 method. Table 3.1 shows the different methods used by the respondents in estimating the runoff coefficient. The majority (80%) uses ARR87, with 11% using ARR77. The rest uses VicRoads charts. Since VicRoads charts use the ARR87 method, almost 90% use the ARR87 method.

Table 3.1: Source of Information Estimating Runoff Coefficient

Method	% Use of Respondents
ARR87	80
ARR77	11
VicRoads charts	9

The ARR87 method does not consider the slope of the catchment in computing the runoff coefficient. It can be considered that catchments with lesser slope will have higher losses (or a lower runoff coefficient), since water is in contact with soil for a longer time. Although, the ARR87 method does not allow for the slope explicitly in computing the runoff coefficient, the slope is included implicitly through the time of concentration. Lower slope produces higher t_c, effectively reducing rainfall intensity, which produces lesser runoff.

The Rational Formula method requires some estimate of time of travel of runoff (i.e. time of concentration of the catchment). Flows can reach drains via roof-to-gutter conduits, overland flow paths or along gutters. In many cases flows travel along two or three paths. For flows from roofs and other surfaces which drain quickly through down pipes and underground drains, a single response time (i.e. property time) can be nominated. The ARR87 suggests 5 minutes as the property time for single dwellings. Similarly, the ARR87 recommends the kinematic wave equation to compute the overland flow time and Manning's formula for flow time in channels and pipes. Gutter flow time can be estimated from design aids for relationships derived from hydraulic models (e.g. VicRoads, 1995) or field measurements. If such information is not available for the gutter cross-section being used, the equation given in Section 4.2.2.3 can be used.

3.3.4 Model Parameters of Computer Models

Five methods were used by those respondents who used computer models, to obtain the model parameters of urban drainage computer models that were used. These methods were:

- use of regional parameters,
- use of default values given in user's manual,
- model parameters by calibration, and
- use of the calibrated parameter values from nearby catchments.

Figure 3.3 shows the above methods used by respondents. Six percent of those respondents obtained the parameter values from the regional equations. However, the respondents did not mention the regional equations that were used. These respondents were contacted again to find out the regional equations used. Most of those respondents used RORB with parameters determined from regional expressions given in ARR87. Some respondents had used Statistical Rational method with model parameters from the regional equations given by ACT Department of Urban Services (1996). The ACT Department of Urban Services provided few regional equations to determine time of concentration (t_c) and runoff coefficient of pervious area (C_p) of urban catchments for the use in Statistical Rational method.

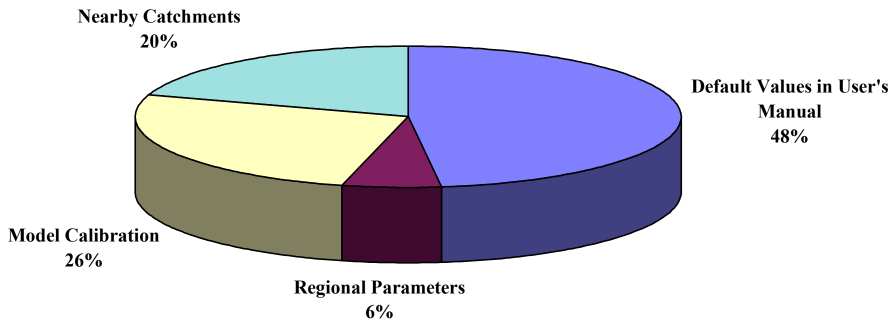


Figure 3.3: Different Methods used by Respondents to Estimate Model Parameters of Computer Models

Most of those respondents who used computer models (48%) obtain the model parameter values from the defaults given in respective user's manuals. Generally, the user's manuals provide a range for the parameters and therefore, the user needs some engineering experience to choose a suitable value. Few respondents calibrated models (i.e. ILSAX and RORB) using observed rainfall-runoff events. However, most of them used one or two events for calibration and the model parameters were not verified using independent data sets. The main reason might be inadequate recorded storm events for calibration and verification. Twenty percent of respondents from the model users stated that they determined the model parameters from the values of nearby catchments. However, in most cases, these catchments were not nearby and the parameter values were obtained from studies for closest catchments found in the literature. Most of the respondents stressed that they had not enough guidelines or regional equations to derive the model parameters for ungauged urban catchments, where rainfall and runoff data are not available.

3.3.5 Return Period

The average recurrence interval (ARI) or return period is defined as the average interval in years between the occurrence of a specified discharge or larger. This has been expressed as a probability of exceedance in exceeding a discharge of certain magnitude (The Institution of Engineers, 1987). This is a convenient way to describe the degree of protection offered by the design. When selecting the average recurrence interval for a design, the following factors should be considered:

- The consequences of flooding, such as damage to property, road and structures,
- Traffic delays or extra travel distance due to road closure during floods,
- Maintenance costs of structures, and
- The additional cost of providing for a longer average recurrence interval.

The size or scale of an urban drainage system can be expressed in terms of the return period of the design flows, which can be carried by the system. General practice of urban drainage design is to design the system for low ARI storm events (i.e. design) and test the system for higher ARI storm events (i.e. analysis). The return periods used by the respondents of the survey are given in Table 3.2 for design and analysis separately. Results of this survey show that there is no single return period that had been used for components of the drainage system. For example, some respondents used 5 year return period for the design of easement, while others used 10 year return period. The location of the structure and the structural component determine the return period to be used in the design. The main difficulty in compiling this information was that different respondents defined the system components differently. There is no clear definition for some drainage system components. For example, there is no clear definition of minor and major works.

Table 3.2: Return Periods Used by the Respondents for Design and Analysis

Structure/Component	Return Period	
	Design	Analysis
Minor works (gutter and pipes for minor storms)	5	10
Major works (drainage routes for major storms)	50, 100	100
Road reserves	2, 5	10
Easement	5, 10	10
Private property	2, 5, 20	5, 20
Industrial	10, 20	10, 20
Street drain	5, 10, 20	20, 50, 100
Road culvert	50	100

Recommended Return Periods in Literature

The decision as to the return period to be adopted in design is essentially a problem of balancing average annual benefits against average annual cost, with regards to the standard of protection from flooding which the community demands. It is difficult to conduct this sort of investigation for each system component of an urban drainage system, during the design stage. Therefore, it is a common practice to specify the return period for these structures by experience or through some form of guidelines.

The recommended return periods by different authorities and experts for design and analysis of different components of urban drainage system in Australia are summarised in Table 3.3.

Table 3.3: Recommended Return period

Components	Return period		Reference
	Design (Years)	Analysis (Years)	
Road surface (Hydroplanning)	0.5-2		NAASRA, 1986
Major roads- Gutters Inlets Table Drains Catch drains Major roads	5-10 10-20 10-20 10-20	10-25 25-50 25-50 25-50 100	NAASRA, 1986 ARR, 1987
Minor roads- Gutters Inlets Table Drains Catch drains	5-10 5-10 10-20 10-20	10-25 10-25 25-50 25-50	NAASRA, 1986
Intensely developed business, commercial and industrial areas	20 20-50		ARR, 1958 ARR, 1987
Business, commercial and industrial areas, closely but not intensely developed	10 25-100 5-40		ARR, 1958, 1987 ARR, 1977 O'Loughlin and Avery, 1980
Intensely developed residential areas	10 10-25 5-20		ARR, 1958, 1987 ARR, 1977 O'Loughlin and Avery, 1980
Sparsely developed residential areas	5 1-10		ARR, 1958, 1987 ARR, 1977
Sparsely built-up areas	3 1-10		ARR, 1958 ARR, 1977

3.3.6 Other Important Issues Raised by Respondents

The following important points were raised by the respondents of this survey, as additional information.

- No regional relationships available to compute the model parameters for use in ungauged catchments, where there are no rainfall and flow data are available.
- No drainage package to work in conjunction with road design package.
- Lack of user friendly methods to enter input data into the models.
- Lack of good information on coefficient of runoff and travel time for small urban catchments.
- Lack of information or methods to compute pit capacity parameters.
- No accurate methods to calculate pipe flows when surcharge occurred.

These issues are discussed below.

Most urban catchments are ungauged. As seen from comments from the respondents, urban drainage designers are reluctant to use the computer models since suitable model parameters are not available for ungauged catchments. Therefore, it is necessary to develop regional equations to estimate model parameters of widely used urban drainage models.

Some respondents highlighted the necessity of a drainage package to work in conjunction with a road design package. CIVILCAD computer software has this functionality, although its drainage analysis capabilities are limited.

Some respondents were concerned about the difficulty in entering data into urban drainage models. Several attempts have been made recently on this point. For example, the ILSIN program had been introduced to ease the data preparation procedure of the ILSAX model. Currently, the user friendliness of ILSAX has been further increased by the introduction of DRAINS (O'Loughlin and Stack, 1998), which runs on Windows. Similar user-friendly data entry capabilities exist in current urban drainage models such as XP-UDD2000 (XP Software, 2000a) and XP-SWMM (XP Software, 2000b).

Designers require relationships to determine inflow rates into pits and possible bypass flows. However, there is some limited information or methods available to compute pit capacities, which determine inflow rates to the pits and bypass flows. The ILSAX user's manual (O'Loughlin, 1993) provided the mathematical expressions to estimate pit capacities. These expressions are discussed in detail in Chapter 4. O'Loughlin *et al.* (1992a) provided parameter values for the expressions dealing with few standard pits. The ARR87 provided a plot representing the flow rate captured against approaching inflow for two pit inlet sizes. The ACT Department of Urban Services (1996) provided several curves for different pit sizes and cross fall slopes. Pezzaniti *et al.* (1999) developed a full scale laboratory model, which can be used to test different types of pit. However, this document did not provide the results of the laboratory tests, but can be requested. Therefore, it is necessary to identify the different types of pits in sites and derive pit capacity parameters with different site conditions (e.g. cross fall slopes) and pit sizes.

Several urban drainage models such as SWMM, WALLRUS, MOUSE, RatHGL, DRAINS, XP-UDD2000 etc., can model surcharge flows. However, ILSAX and RORB models cannot handle surcharge flows.

3.3.7 Water Quality Considerations

Water quality problems in urban drainage systems are not so severe in Australia compared in many places in North America and Europe (O'Loughlin *et al.,* 1992b). However, there are locations where pollutants carried in urban stormwaters have contributed to problems in receiving waters. Therefore, control measures are very important for the total management of stormwater. For the completeness of this survey and to get a feel on water quality improvement methods used by practitioners, a question regarding water quality measures was included in the questionnaire. According to respondents, the following measures have been implemented by some councils in Victoria to improve water quality in urban drainage networks and receiving water bodies.

- Design of wetlands for removal of nutrient, sediment, trash and oil.
- Construction of litter and silt traps.
- Street sweeping.
- Routine maintenance.
- Collection of chemical wastes separately.
- Holistic (or integrated) catchment management approach.
- Implementation of education programs to reduce litter entering receiving water bodies.

3.4 COMPARISON WITH PREVIOUS SIMILAR STUDIES

As stated in Mein and Goyen (1988), the USA Federal Emergency Management Agency (USA FEMA) conducted a survey on urban flood estimation methods in USA in 1986 and found that more than 90% practitioners used manual methods such as Rational Formula method and TR-55 method (US Soil Conservation Service, 1986) for stormwater drainage design and analysis. A market analysis survey for Australian practice in urban stormwater drainage design and analysis conducted by WP Software in 1989, found that 78% of practitioners use manual methods (O'Loughlin and Goyen, 1990). Only 22% use computer models. The present survey conducted in Victoria found that 79% use manual methods (in this case the Rational Formula method) and remaining 21% use computer models. Comparing the present study (which was restricted to Victoria) with the WP Software study in 1989 (dealing with whole of Australia), the use of computer models in Victoria for drainage analysis is almost same as the national figure. Details of these surveys are given in Figure 3.4. Moreover, within the 9 years from 1989 to 1997, the percentage users of computer models for urban drainage design and analysis have not increased. However, the general perception is that there are more users of computer models in urban drainage design and analysis now compared to 1989. Perhaps what this mean is that there are more consultants and government authorities conducting urban drainage studies in 1997 compared to 1989.

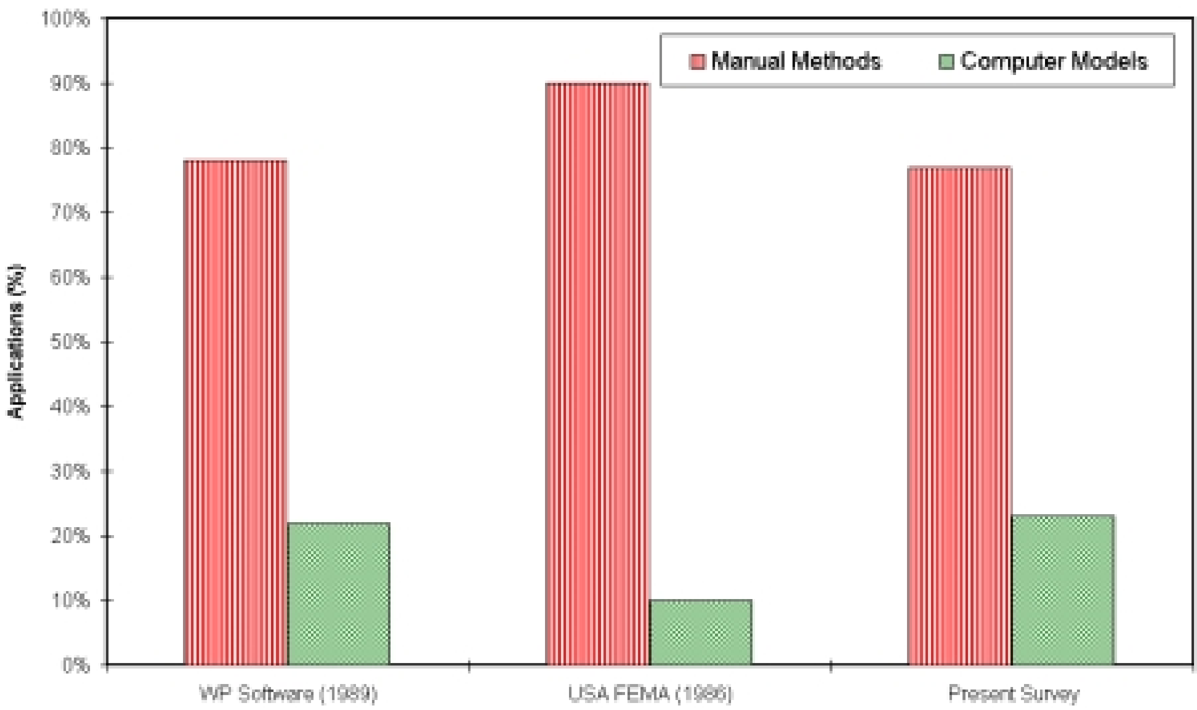


Figure 3.4: Results of Surveys on Methods Used in Practice

3.5 SUMMARY

A Victoria wide survey was conducted in May 1997 to investigate the current practice used by government authorities and consultants in design and analysis of urban stormwater drainage systems. A questionnaire was prepared and sent to 78 city/shire councils and 38 consultants in Victoria. These are the organisations that deal with urban drainage system design and analysis in Victoria.

The results of the survey showed that a large number of respondents still use manual (and approximate) methods such as Rational Formula method (which involve many assumptions) for urban drainage design studies. Based on the survey conducted in this study, only 21% use computer models in Victoria for urban drainage design and analysis. This is an agreement with the findings of a similar but national survey conducted in 1989. The ILSAX model was the widely used computer model in Victoria.

Based on the survey results, most designers were reluctant to use computer models since most of them were not user-friendly at the time of the survey. In addition, adequate guidelines were not available to use of these models. Users find difficulty in selecting the model parameters for application of these models to ungauged urban drainage systems.

CHAPTER 4

THE ILSAX MODEL

4.1 INTRODUCTION

As stated in Section 2.3, the ILSAX model was used in this study. The major reason for this was that it was the widely used computer model for stormwater drainage design and analysis in Victoria, as found from the customer survey conducted in May 1997 (Chapter 3). The name ILSAX stands for "ILLUDAS-SA with something extra (**IL**LUDAS-**SA**+e**X**tra). Since its development in 1986 as an improved version of ILLUDAS-SA (Watson, 1981), the ILSAX model has been used widely in Australia and New Zealand for many large projects such as the analysis of the Auckland drainage system (O'Loughlin and Stack, 1998).

The ILSAX model has a long history of development. It started as the RRL (Road Research Laboratory) model in 1962 (Terstriep and Stall, 1969). Table 4.1 shows the historical development of the ILSAX model and its future development. Details of different stages of the development are given in O'Loughlin and Stack (1998).

A detailed analysis was conducted in Chapter 6 to select the appropriate modelling options to be used in this thesis from several options available for modelling various hydrologic and hydraulic processes. These modelling options are related to pervious area loss subtraction, time of entry for overland flow routing, pipe routing and pit inlet capacity restrictions. Only a description of these modelling options is presented in this chapter. For full details of the ILSAX model, the reader is referred to O'Loughlin (1993) and O'Loughlin and Stack (1998).

Table 4.1: Different Development Stages of the ILSAX Model

Model	Year of Development	Important Features, Enhancements to Previous Version
RRL (Terstriep and Stall, 1969)	1962	Included only impervious areas
ILLUDAS (Wenzel and Voorhees, 1980)	1974	Incorporated modelling of pervious areas
ILLUDAS-SA (Watson, 1981)	1981	No major changes to modelling philosophy except that additional pervious area loss modelling options added. The other main changes were: • The program was converted to metric units, • The input and output facilities were improved, • Rainfall hyetographs or patterns can be entered in four ways, • Provision was made to carry out sensitivity analyses, and • An arbitrary runoff hydrograph can be inserted at any point in the design system.
ILSAX (O'Loughlin, 1993)	1986	Included more detailed methods for overland flow routing and pit entry modelling.
DRAINS (O'Loughlin & Stack, 1998)	1998	Included improved methods for supplementary area and pipe hydraulic (Hydraulic Grade Line) modelling. Converted to Windows.

4.2 ALTERNATIVE MODELLING OPTIONS OF HYDROLOGIC AND HYDRAULIC PROCESSES

The ILSAX computer model is capable of describing the behaviour of a catchment and a pipe system for real storm events, as well as statistically based design storms. In order to use the ILSAX model, the catchment is divided into several subcatchments according to land use or other physiographic conditions. It uses storm rainfall as input, subtracts infiltration and other losses, and routes the resultant rainfall excess through the subcatchment and the pipe system. It models dendritic or tree-like networks, but cannot handle looped systems.

A schematic diagram is shown in Figure 4.1, which illustrates the ILSAX modelling representation of an urban or semi-urban catchment. It also shows various components of drainage system such as inlets, pipes and detention storage, and the flow paths. At the upper ends of reaches, an inlet receives stormwater from its subcatchment and discharges to the rest of the system via outlet pipe of the subcatchment. Therefore, a basic modelling element in ILSAX can be considered as the pit with its subcatchment and the outlet pipe, as shown in Figure 4.2. The pits other than those at the upper ends receive stormwater from their subcatchments as well as from upstream inlet pipes.

As seen in Figures 4.1 and 4.2, each subcatchment can be divided into three significant parts, namely directly connected impervious area, supplementary area and pervious area. In addition, there can be a fourth catchment surface, which does not contribute to runoff (e.g. swimming pool). This area is excluded from the catchment area in modelling. The directly connected impervious area includes road surfaces, driveways, roofs and other elements that are directly connected to the drainage system. The supplementary area considers the impervious area which is not directly connected to the drainage system, but the runoff from these areas flows over the pervious surfaces before reaching the drainage system. The pervious area includes bare surfaces, porous pavements, lawns and other elements that are directly connected to the drainage system.

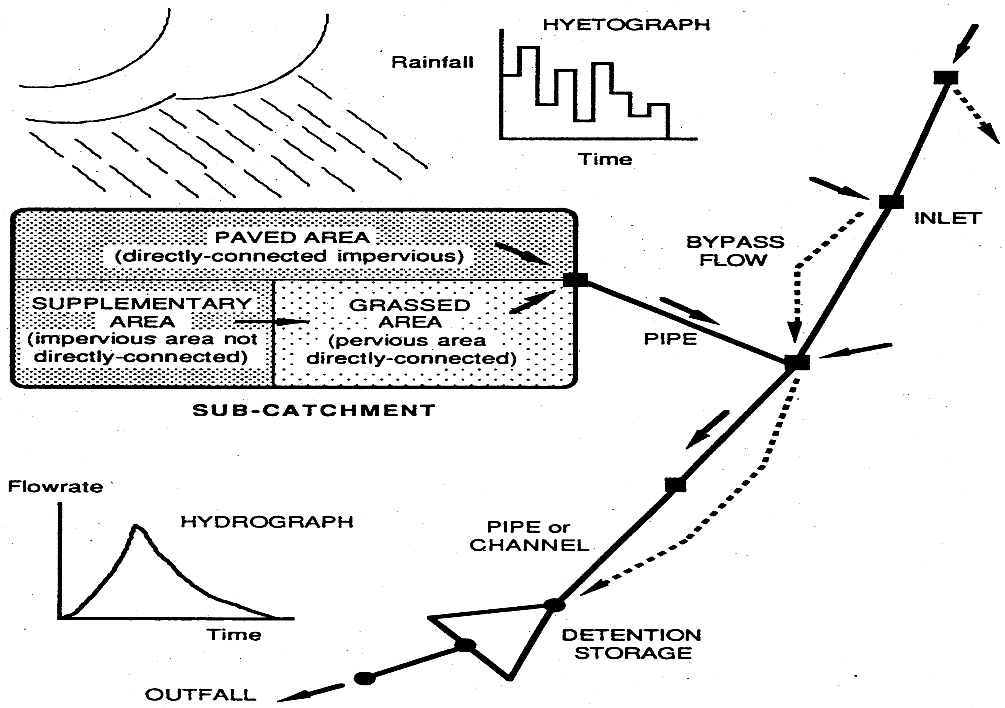


Figure 4.1: ILSAX Representation of a Catchment

(O'Loughlin, 1993)

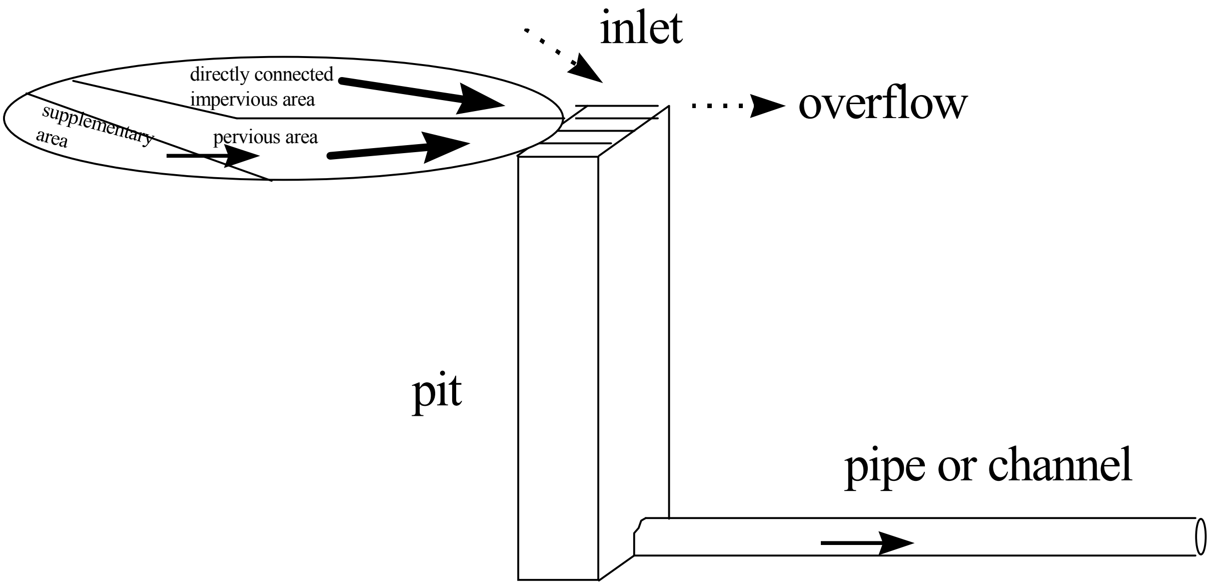


Figure 4.2: Basic ILSAX Modelling Element

Generally, the directly connected impervious area responds first to the rainfall. Then, supplementary areas and pervious areas respond. In most cases, for 'small' storm events, the runoff is generated only from the directly connected impervious area, while for 'large' storm events, the runoff is also generated from supplementary and previous areas.

The rainfall excess from different types of land surfaces (i.e. directly connected impervious areas, supplementary areas and pervious areas) should be estimated first to obtain the input hydrograph into a pit inlet. The rainfall excess is computed by subtracting evaporation, infiltration and other losses from rainfall. In storm event modelling, the evaporation loss is insignificant since duration of storm event is small and evaporation itself is less during a storm event. The rainfall excess is then routed through different overland surfaces (i.e. over property, road and gutter system). After overland flow routing, the surface runoff enters the pipe system through pit inlets. The runoff is then routed through the conveyance system, which includes pipes and channels, in some cases detention or retention basins.

4.2.1 Options for Modelling of Rainfall Excess from Pervious Areas

The rainfall excess corresponding to different surfaces (i.e. directly connected impervious areas, supplementary areas and pervious areas) are modelled differently in ILSAX because of their different processes. Since there are two methods available for modelling rainfall excess from pervious areas, they are discussed in this section. These methods are studied in detail in Section 6.5. The details of modelling rainfall excess for directly connected impervious areas and supplementary area are described in O'Loughlin (1993).

The pervious area losses are modelled in the ILSAX model through an initial and continuous loss model. The initial loss model is represented by pervious area depression storage, which accounts for the processes of interception, depression storage and evaporation. The continuing loss allows for infiltration and estimated from the Horton infiltration equation. Since the ILSAX model is an event model, the conditions at the start of the event must be established by defining a value of the antecedent moisture conditions (AMC) for the soil underlying the pervious area. The ILSAX model defines four soil classifications designated as A, B, C and D, as shown in Figure 4.3. These soil classifications are also called the Curve Numbers (CN) in ILSAX, and given values 1, 2, 3 and 4 respectively. Other soil types can be generated by interpolation between these four classifications. These soil types are used in conjunction with antecedent moisture conditions (AMC's), which fix the points on the infiltration curves at which calculations commence (Figure 4.3).

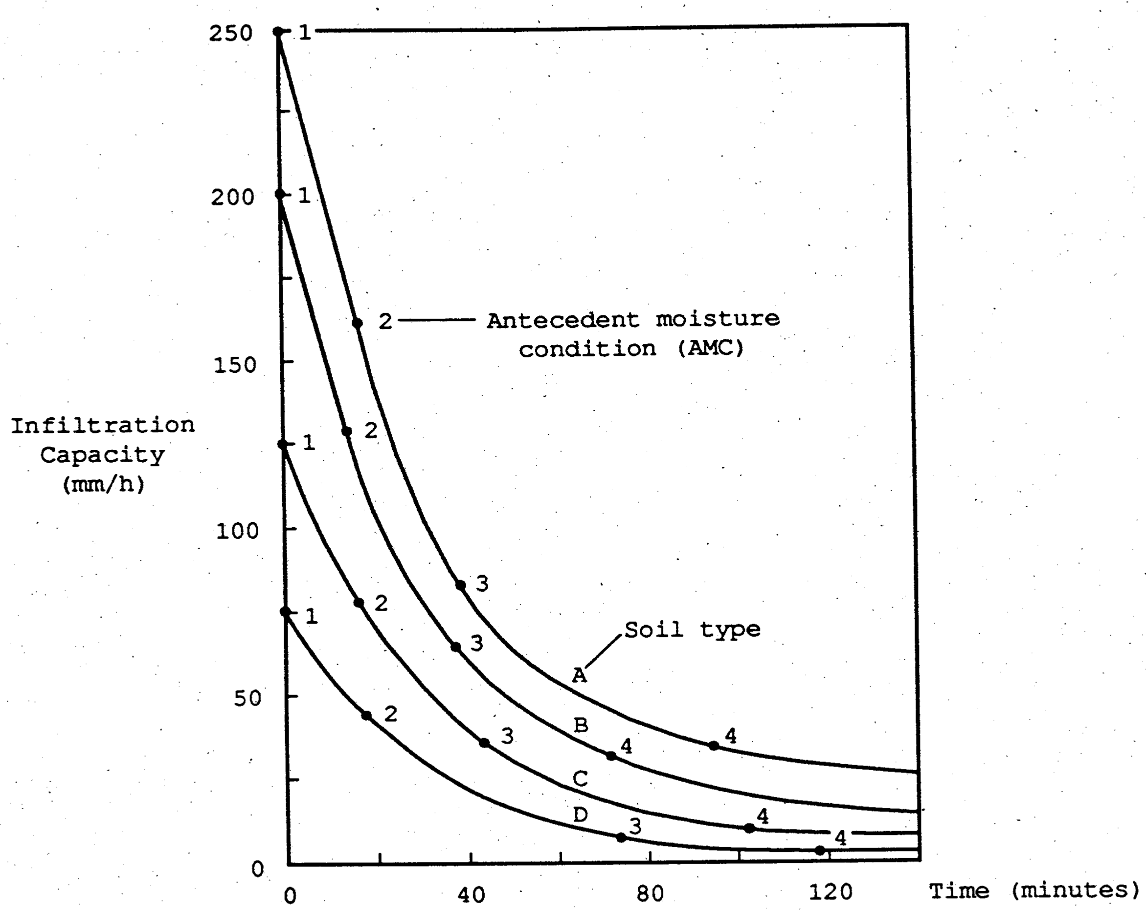


Figure 4.3: Infiltration Curves for Soil Types Used in ILSAX (O'Loughlin, 1993)

The soil type CN in ILSAX is described by the Horton infiltration equation, as follows.

$$f_p = f_c + (f_0 - f_c)e^{-kt} \tag{4.1}$$

where f_p is the infiltration capacity of soil (m/s),

f_c	is the minimum or ultimate value of f_p (m/s),
f_0	is the maximum or initial value of f_p (m/s),
t	is the time from beginning of storm (s), and
k	is the decay coefficient (s^{-1}).

To define the soil type of a catchment, the user can specify CN in the range 1 to 4 (non-integer numbers are possible) according to soil type of the catchment and AMC value considering the rainfall occurred prior to the storm event. Then the four soil curves (or intermediate curve if CN is non-integer value) are used in conjunction with AMC which fix the points on the infiltration curves at which calculations commence. The second option is to define a curve with characteristics provided by the user. In this option, the user has to enter f_0, f_c, k and four AMC values (altogether 7 parameters) to define the curve for catchment soil type.

AMCs immediately prior to the storm event obviously play a very significant role in determining the actual pervious area runoff contribution. The effect is more significant, as the magnitude of the storm event increases because of the potential for increased pervious area runoff. Table 4.2, which is reproduced from the ILSAX manual (O'Loghlin, 1993), provides some guidance for selecting the AMC immediately prior to the storm event by considering the total 5-day rainfall depths prior to the event.

As can be seen from Table 4.2, the AMC does not consider the temporal distribution of the rainfall during the previous five days. In reality, the AMC condition of a catchment that receives 25 mm of rainfall on the first day of the 5-day period prior to the storm event is not the same as the catchment receiving 25 mm of rainfall on the fifth day. However, Table 4.2 is the best information currently available to compute the initial AMC.

As stated earlier, there are two options in ILSAX to compute the rainfall excess from pervious areas. In the first method, the losses are subtracted from rainfall, while in the second method, losses are subtracted from supply rate (i.e. after rainfall is routed using the time-area method). These methods are explained below.

Table 4.2: Selection of Antecedent Moisture Condition

(O'Loughlin, 1993)

AMC condition	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 25.0
4	Saturated	Over 25.0

Losses subtracted from rainfall

In this method, the losses (i.e. both initial and continuous) are subtracted from the rainfall to compute rainfall excess. The rainfall excess is then routed using the time-area method. During the hydrograph event, although rainfall stops at the end of storm, flows may still travel across pervious surfaces to the catchment outlet, which will have extra continuing losses. Therefore, this method does not allow for the possible continuing losses after rainfall stops.

Losses subtracted from supply rate

In this method, the hyetograph is convolved with the time-area diagram without considering losses to obtain the supply rate. Then, both initial and continuing losses are subtracted from this supply rate. This method is preferred since it allows for possible losses even after rainfall stops.

Time-Area Method

Both methods above use the time-area method to generate hydrographs for each subcatchment area. In this method, the rainfall hyetograph (whether the original rainfall or the rainfall excess) is combined with the time-area diagram in a similar manner to unit hydrograph calculations. Figure 4.4 illustrates the time-area procedure. The following paragraphs explain the basic theory of this procedure, as described in O'Loughlin (1993).

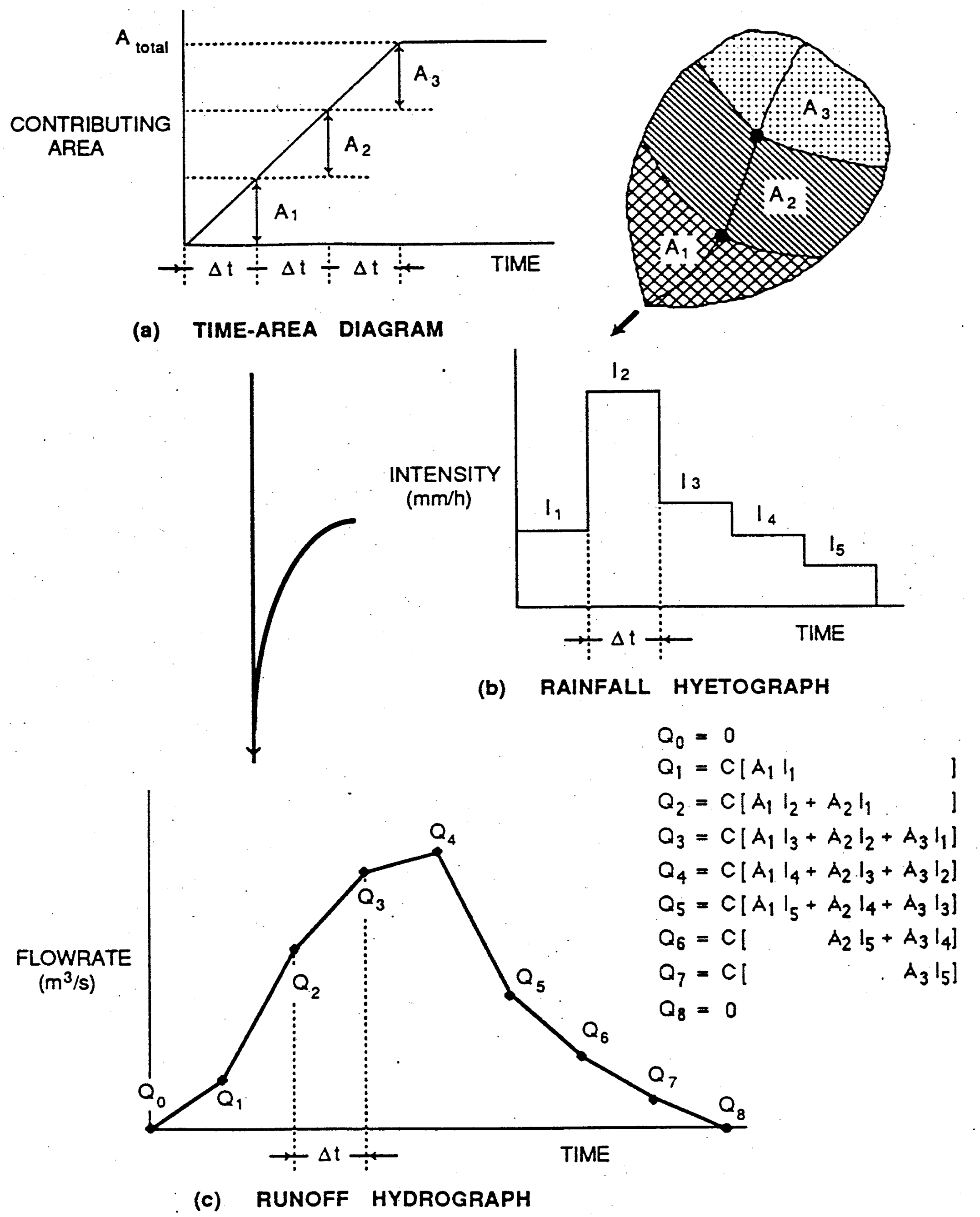


Figure 4.4: Construction of Hydrograph by the Time-Area Method (O'Loughlin, 1993)

The rainfall hyetograph is divided into computational time steps of Δt. Δt is a user-defined value and is a fraction of the time of concentration (or the time of entry) for a subcatchment area. The time of concentration is determined from the methods described in Section 4.2.2.

The time-area diagram (i.e. a plot of the catchment area contributing to runoff at the inlet pit versus time from the start of the storm) is also divided into time steps of Δt. This diagram can be visualised by drawing isochrones, or lines of equal time of travel to the catchment outlet. For times greater than the time of concentration, the area contributing equals the total area of the catchment. ILSAX assumes a linear relationship between contributing area and time of concentration, as shown in Figure 4.4(a).

When a storm commences in a catchment which has a time of entry of $3\Delta t$ (Figure 4.4), the initial flow Q_0 is zero. After one time step Δt, only sub-area A_1 contributes to the flow at the outlet. Any runoff from other sub-areas is still in transit to the outlet. Thus the flow rate at the end of the first time step can be approximated using the formula of $Q_1 = C.A_1.I_1$, where C represents the conversion factor from mm/h to m^3/s units (If A is in ha, C = 1/360), and I_1 is the average rainfall intensity (mm/h) during the first time step.

At the end of the second time step, there are two contributions to the outlet flow, Q_2, due to the first and second blocks of rainfall. First contribution is from subarea A_1 due to second block of rainfall ($=C.A_1.I_2$) and the second contribution is from subarea A_2 due to first block of rainfall ($=C.A_2.I_1$). Therefore, $Q_2 = C.(A_1.I_2 + A_2.I_1)$. At the end of the third time step, there are three contributions, $Q_3 = C.(A_1.I_3 + A_2.I_2 + A_3.I_1)$, and so on, as shown in Figure 4.4. The hydrograph builds up to a peak and then recedes once rainfall stops and catchment drains.

4.2.2 Options for Modelling of Times of Entry for Overland Flow Routing

Times of entry must be specified or calculated within ILSAX for modelling overland flow from impervious and pervious areas. They are effectively the same as the times of concentration (or times of travel) used in the Rational Formula method. They are used to set the base lengths of the time-area diagram in computing hydrographs of subcatchments from different surfaces.

Three options are available in ILSAX to compute the time of entry. Each option considers the pervious and impervious area separately. The ILSAX model does not allow the overland hydrograph computed from an impervious or pervious area to be routed over another area. The three options to compute the time of entry are listed below.

- User-defined time of entry,
- ILLUDAS-SA method and
- ARR87 method.

4.2.2.1 User-defined times of entry

With this option, the times of entry for both pervious and impervious areas can be calculated by the user beforehand and enter them directly into ILSAX. However, this method can be time consuming if the system consists of a large number of pits and if it is required to calculate the time of entry for each pit.

4.2.2.2 ILLUDAS-SA method

The second option for computing the time of entry is the method used in the ILLUDAS-SA model (Watson, 1981). Under this option, the times of entry for impervious and pervious areas are computed differently. These methods are described below.

Time of Entry for Impervious Area

For this option, the time of entry is computed as the sum of the travel time from houses to the gutter (i.e. property time) and the gutter time. The user should specify the lengths and slopes of the gutters to calculate the time in street gutters (i.e. gutter time). Manning's equation with a hydraulic radius of 60 mm and a roughness coefficient of 0.020 are assumed in computing the gutter time and then two minutes added to allow for the property time. It is assumed that the depth of flow is approximately 60 mm in computing the gutter time. These assumptions are not valid for every storm event and every impervious surface. For larger events, the flow depth may exceed 60 mm. Surface roughness varies from one impervious area to another. The property time may not always be 2 minutes.

Time of Entry for Pervious area

The time of entry for pervious area is computed based on the kinematics wave equation (O'Loughlin, 1993) for overland flow and is given by Equation 4.2.

$$t_{overland} = 6.94\ (L\ n)^{0.6}\ /\ i^{0.4}\ s^{0.3} \tag{4.2}$$

where $t_{overland}$ is the overland flow time (min),
L is the flow path length (m),
i is the rainfall intensity (mm/h),
s is the slope of pervious area (m/m) and
n is the surface roughness or retardance coefficient (similar, but not identical to the coefficient 'n' in Manning's formula. Typical values are given in O'Loughlin, 1993).

In ILLUDAS-SA, n is set at 0.05 and I is at 25 mm/h. Therefore, if rainfall intensity of an event is different from 25 mm/h or retardence coefficient is different from 0.05, this method may not produce correct pervious area time of entry.

4.2.2.3 ARR87 Method

This method is a modification of the ILLUDAS-SA method and is recommended in Chapter 14 of ARR87 (The Institution of Engineers, 1987). The ARR87 relationship is more complex compared to the other two methods. Flow path lengths, slopes, cross-sectional details of gutters and surface roughness coefficients are required as input for this method.

Time of entry for impervious areas

As for the ILLUDAS-SA method, the time of entry in the ARR87 method consists of gutter and property time. The gutter time can be considered for two configurations for this option, namely the gutter profile with vertical and sloping kerbs as shown in Figure 4.5. For gutters with a vertical kerb, the discharge equation is obtained by applying the Izzard equation for a triangular channel. The Izzard equation is given by:

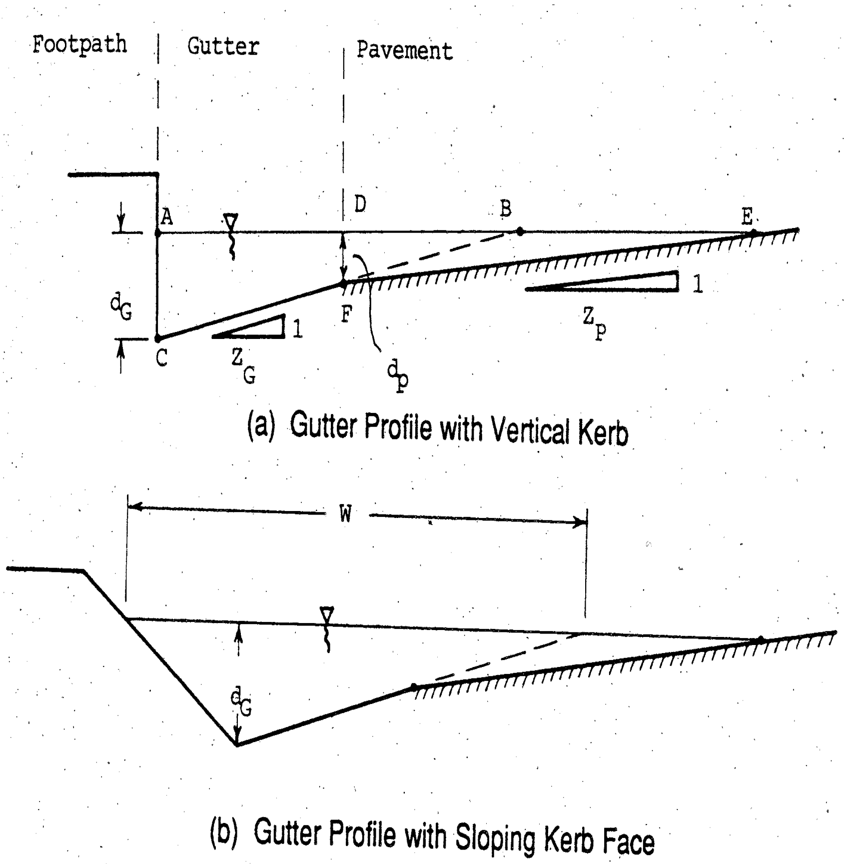


Figure 4.5: Gutter Flow Characteristics
(O'Loughlin, 1993)

$$Q = 0.375\ F\ (Z/n)\ d^{8/3}\ S_0^{1/2} \tag{4.3}$$

where
- Q is the flow rate (m^3/s),
- F is the flow correction factor,
- Z is the gutter cross slopes (m/m),
- n is the Manning roughness coefficient,
- d is the greatest gutter depths (m), and
- S_0 is the longitudinal slope (m/m).

The gutter flow for this configuration is then given by:

$$Q = 0.375\ F\ [(Z_G/n_G)\ (d_G^{8/3} - d_P^{8/3}) + (Z_P/n_P)\ d_P^{8/3}]\ S_0^{1/2}$$
(4.4)

Cross sectional area of the gutter (A) is given by:

$$A = [Z_G\ (d_G^2 - d_P^2) + Z_P\ d_P^2]/2 \quad (4.5)$$

where Z_G is the gutter cross slopes (m/m),
Z_P is the pavement cross slopes (m/m),
d_G is the greatest gutter depths (m), and
d_P is the greatest pavement depths (m).

From continuity equation, the average flow velocity in the gutter (V) can be expressed as:

$$V = Q/A = 0.375\ F\ [(Z_G/n_G)\ (d_G^{8/3} - d_P^{8/3}) + (Z_P/n_P)\ d_P^{8/3}]\ S_0^{1/2}/A \quad (4.6)$$

where Q is the flow rate (m^3/s),
F is the flow correction factor,
Z_G is the gutter cross slopes (m/m),
Z_P is the pavement cross slope (m/m),
n_G is the Manning roughness coefficient for gutter,
n_P is the Manning roughness coefficient for pavement,
d_G is the greatest gutter depths (m),
d_P is the greatest pavement depths (m), and
S_0 is the longitudinal slope (m/m).

The general equation for calculation of gutter time is:

$$\text{Time} = \frac{(Gutter\_Length)}{V * 60} \quad (4.7)$$

where *Gutter_Length* is gutter length.

The travel time from roof to pit (i.e. sum of gutter time and property time) given in ILSAX as:

$$\text{Time} = \frac{(Gutter\_Length)}{GUT * \sqrt{S_0} * 60} + t_o \qquad (4.8)$$

where t_o is travel time allowed for roof to gutter (5 min constant time as default or a user-defined time).

Therefore, combining these equations (i.e. Equations 4.6, 4.7 and 4.8), it can be shown that GUT equals:

$$GUT = 0.75\ F\ [(Z_G/n_G)(d_G^{8/3} - d_P^{8/3}) + (Z_P/n_P)\ d_P^{8/3}]\ /\ [Z_G.(d_G^{2} - d_P^{2}) + Z_P\ d_P^{2}] \qquad (4.9)$$

where
- F is the flow correction factor,
- Z_G is the gutter cross slopes (m/m), (Figure 4.5a)
- Z_P is the pavement cross slopes (m/m),
- n_G is the Manning roughness coefficient of gutter,
- n_p is the Manning roughness coefficient of pavement,
- d_G is the greatest gutter depths (m), and
- d_P is the greatest pavement depths (m).

As can be seen from Equation 4.9, GUT factor depends on F and geometry of the gutter. In the absence of more precise information, ILSAX assumes F as 0.8 for vertical kerb gutters.

Where the face of a kerb is relatively steep, it can be considered to be vertical. For gutter profile with sloping kerb (Figure 4.5b), the same method used in vertical kerb can be used to compute GUT approximating Z_G to be equal to w/d_G.

The ILLUDAS-SA procedure assumes two minutes as the property time and is in-built into ILSAX. Therefore, the user cannot change the property time with the ILLUDAS-SA procedure. The ARR87 method, on the other hand, suggests five minutes as the property time and also provides facility to use a different value, if required.

Time of Entry for Pervious area

Equation 4.2 is used to compute the time of entry for pervious area. However, this option allows the user to enter a retardance coefficient n, and uses I as the mean intensity of the rainfall pattern provided for the storm under consideration, as opposed to the ILLUDAS-SA method. Therefore, the ARR87 method offers more flexibility than the ILLUDAS-SA in computing the pervious area time of entry.

4.2.3 Options for Pipe and Channel Routing

Pipe and channel routing procedures model the passage of flows through a pipe/channel reach. Due to the travel time and storage effects in pipes and channels, the downstream hydrograph shape and peak discharge are different from those of the upstream hydrograph. Two pipe/channel flow routing procedures namely time-shift and implicit hydrological methods are available in ILSAX.

4.2.3.1 Time-shift method

The time-shift method lags the upstream hydrograph by the time of travel with respect to peak flow to produce the downstream hydrograph. In this method, the storage effects are neglected. Therefore, the downstream hydrograph shape and peak discharges are the same for both upstream and downstream hydrographs.

4.2.3.2 Implicit hydrological method

The implicit hydrological method considers routing of the upstream hydrograph through the storage occurring in the reach (pipe and channel) to produce the downstream hydrograph. Therefore, the peak discharge of the downstream hydrograph is lower than that of the upstream hydrograph. Since this method considers the storage effects in routing, it is obviously better than the time-shift method, although the data requirements are heavier. The reach cross section is an input for this option, and the following reach cross sections can be modelled in ILSAX.

a) circular section,
b) closed rectangular section,
c) open trapezoidal section (including rectangular and triangular sections), and
d) irregular open or closed sections.

Pipe friction relationships are important in determining reach flow capacities. In the ILSAX, the model users have a choice of using the Manning's formula or the Colebrook-White equation for all types of reach cross-sections.

4.2.4 Modelling of Pit Inlets

In piped stormwater drainage systems, pits serve several purposes. They act as inlets for stormwater, as points where pipes can conveniently change their size, slope or direction, and as inspection and maintenance openings. Stormwater entry into these pits depends on their inlet capacities.

The hydraulic capacity of an inlet depends on its geometry and the characteristics of the gutter flow. Inadequate inlet capacity or poor inlet location may cause:

- under utilisation of the underground system; and
- flooding on traffic lanes resulting in hazard to moving vehicles, or overflow into adjacent properties.

Before discussing the pit inlets, it is important to understand the difference between bypass flow and overflow definition in ILSAX. These two types of flows are shown in Figure 4.6. If overland (surface) flows approaching to a pit is higher than the its inlet capacity, then bypass flow at the pit occurs. Similarly, at the pit if the sum of approaching pipe flow and inflow from the pit is higher than the reach capacity of the immediately downstream pipe, then overflow occurs from this pit.

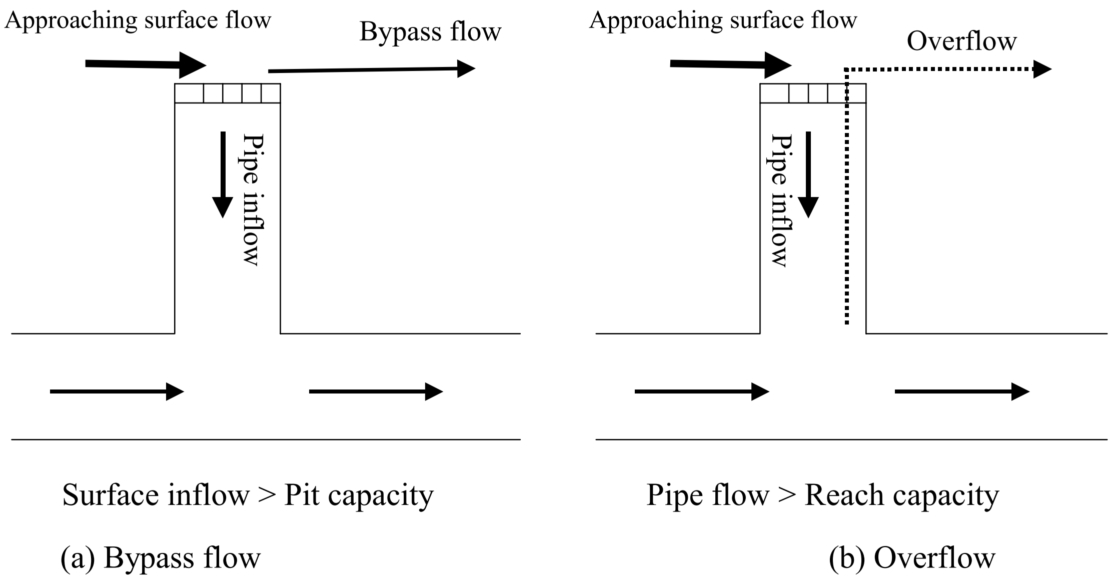


Figure 4.6: Bypass Flow and Overflow in a Pipe Reach

Two types of pits are commonly used. They are on-grade and sag pits. On-grade pits are generally located on a slope. Bypass flows from an on-grade pits move away from the pit and travel into the next pit. Sag pits, on the other hand, are located in a depression (or sag), so that water cannot readily escape. If the capacity of the sag pit is not sufficient to accept all flows arriving at the pit, stormwater ponds near the pit until it becomes high enough to cross some barriers such as the crown of a road. Ponded water is released to the pit, when the inlet capacity becomes available. There are several variations of these two common pit types (NAASRA, 1986, O'Loughlin, 1993). For both on-grade and sag pits, there are three types of inlets namely side entry (or kerb-opening), grade inlets and combine inlets. For details of these different variations of these pits, the reader is referred to McIllawraith (1959), NAASRA (1986), O'Loughlin *et al.* (1992a) and O'Loughlin (1993).

The ILSAX model has two ways of modelling the pit inlet capacity. They are:

- No inlet restriction (Infinite capacity) - In this case, unlimited inlet capacity is assumed. However, if reach capacity is not sufficient to cater for the incoming flow, then the overflows at the pit are stored at the upstream end of the reach and released back into the reach when capacity becomes available. This option is available for both on-grade and sag pits.

- Inlet capacity determined by relationships obtained through hydraulic model studies (Finite capacity) - This option is also available for both on-grade and sag pits. When modelling on-grade pits, bypass and overflows can be directed to a pit downstream or directed out of the drainage system. With sag pits, water will pond, up to an user-defined limit. Once water level exceeds the limit, bypass flows can be directed out of the system or directed to a downstream pit. Generally, these relationships have to be determined through hydraulic model studies, since there is no comprehensive theory available to determine them. The inlet capacity can be changed significantly by small differences in dimensions and by features such as depressions and types of grate. For details of these pits and estimation of their capacity parameters, the reader is referred to The Institution of Engineers (1987), O'Loughlin *et al.* (1992a), O'Loughlin (1993) and Pezzaniti *et al.* (1999). This method is better than the no inlet restriction method, since it is more closer to the reality.

4.2.4.1 On-grade pits

The relationship available in the ILSAX model to describe on-grade pit inlet capacity has the form of Equation 4.10.

$$\text{CAPACITY} = \text{CAP1} + \text{CAP2}\ Q + \text{CAP3}\ Q^{\text{CAP4}} \qquad (4.10)$$

where	CAPACITY	is the inlet capacity (m^3/s),
	Q	is the flow discharge arriving at the Inlet (m^3/s) and
	CAP1, CAP2, CAP3 and CAP4	are the factors supplied by users.

The relationship given in Equation 4.10 can be used as a linear equation, polynomial or power function, depending on the values of user-defined parameters. The main practical difficulty is to get the correct values for these four parameters. The ARR87 provides a curve for on-grade pits to estimate these parameters, for pits of 1 m and 2 m sizes.

4.2.4.2 Sag pits

For a sag pit, the following relationship is available in the ILSAX model to compute the inlet capacity:

$$CAPACITY = VCAP1 + VCAP2.V^{VCAP3} \tag{4.11}$$

where	CAPACITY	is the inlet capacity (m^3/s),
	Q	is the flow rate arriving at the inlet (m^3/s),
	VCAP1, VCAP2, VCAP3	are the factors supplied by users, and
	V	is the ponded volume (m^3).

The user has to develop a relationship for ponded volume (V) and inlet flow velocity (Q) from VCAP1, VCAP2 and VCAP3 can be estimated.

4.2.4.3 Choke factor

When the mathematical relationships are derived from hydraulic model testing, the tests are generally conducted with water, free from debris. However, stormwater flows carry debris loads during storm events. Because of these debris loads, the inlet capacity will be reduced. The choke factor allows for this effect in ILSAX.

For both on-grade and sag pits, the choke factor (CF) simulates the blockage of the pit. If CF is 0, there is no blockage at the pit and the inlet capacity is determined from either Equation 4.10 or 4.11. If it is 1, then there will be complete blockage at the pit and stormwater does not enter the pit at all. Typical values recommended in the ILSAX user manual are 0.2 for an on-grade pit and 0.5 for a sag pit. However, it is understood that CF depends on conditions of the catchment prior to the storm event. These conditions depend on the cleaning frequency of road-gutter system, the season of year (i.e. more blockage during Autumn due to fallen leaves), prior rainfall etc. The choke factor is a dynamic parameter for a catchment like AMC. However, the ILSAX model treats this factor as a static factor. Although, an average value of choke factor can be selected for design of urban drainage systems, the most suitable value of the choke factor at the time of analysis should be taken in the analysis (i.e. evaluation of adequacy of the system for different rainfall conditions) of these systems.

4.3 ILSAX MODEL PARAMETERS AND THEIR ESTIMATION

As any other computer model, the ILSAX model has its model parameters. The ILSAX model is conceptualised as shown in Figure 4.7 showing its model parameters. They can be divided into two main groups. The first group deals with the parameters responsible for the rainfall excess. The second group accounts for routing parameters of pervious and impervious areas, and drainage pipes and channels. These two groups are loosely termed in this thesis as hydrological and routing parameters respectively. The hydrological parameters are the pervious area depression storage (DS_p), the impervious area depression storage (DS_i), the soil curve number (CN) and the antecedent moisture condition (AMC). The parameters CN and AMC define the infiltration process of pervious areas. The routing parameters are the Manning's friction coefficient of pipes (N_p), the retardance coefficient of pervious areas (N_r) and the choke factor (CF). Additionally, the gutter flow factor (GUT) and two pit parameters (CAP3 and CAP4) for grade pit inlets were also considered. Although the sag pits can be modelled with ILSAX, they are not shown in Figure 4.7, since they were not present in this study catchments described in this thesis. Therefore, altogether six ILSAX routing parameters were considered in this study.

GUT can be estimated from hydraulic data of the gutters and the pit capacity parameters from published literature based on physical hydraulic modelling and hence, both these parameters are dependent on the physical characteristics of gutters and pits. Therefore, GUT and pit capacity parameters can be considered as data, but they should be carefully selected since they affect the output response.

The hydrological parameters are responsible for the runoff volume of the catchment but the routing parameters do not affect the runoff volume. Therefore, the runoff volume depends only on the hydrological parameters, while the peak discharge depends on both hydrological and routing parameters.

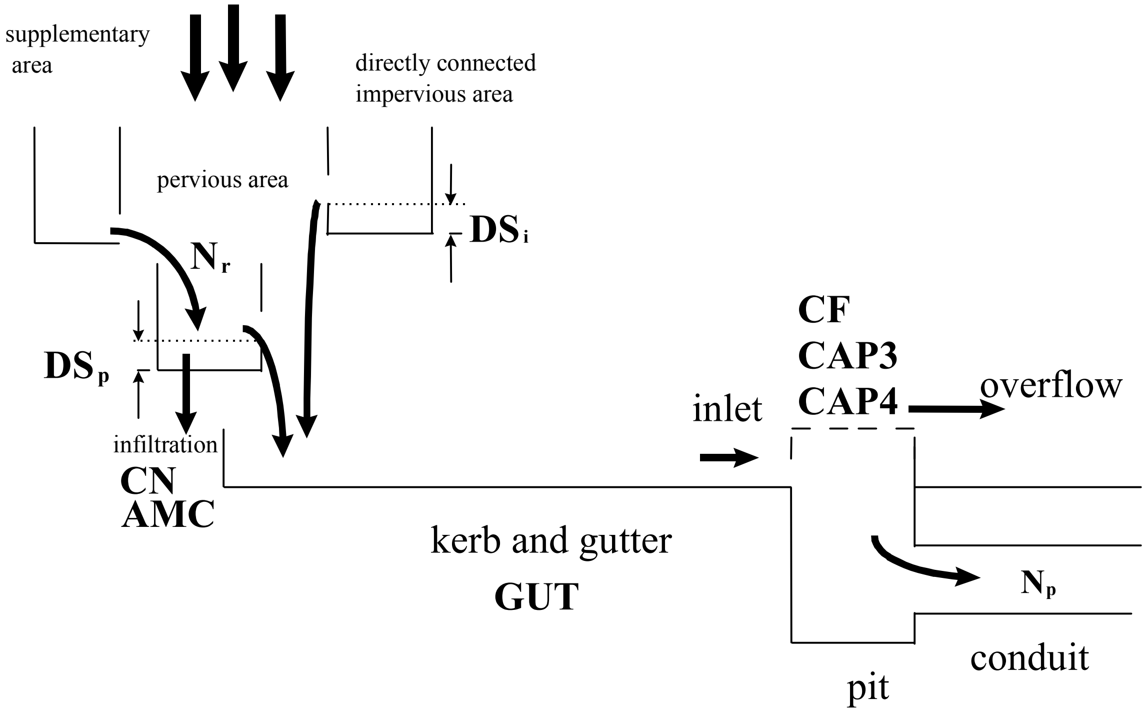


Figure 4.7: ILSAX Model Representation and Its Parameters

The hydrologic parameters define the rainfall excess and depend on specific catchment characteristics (e.g. soil type, percent imperviousness, and depression storage) and in some cases on rainfall characteristics. These parameters are sensitive to output responses such as runoff volume and peak of the hydrographs. Therefore, the hydrologic parameters should be calibrated for gauged catchments or estimated by some reliable method for ungauged catchments. The routing parameters describe flow routing in the catchment and the pipe/channel systems. They are often fairly constant or, at least, can be estimated or extracted from literature with less variability. These parameters are less sensitive to output responses compared to the hydrologic parameters. The sensitivity of both sets of parameters was carried out and discussed in detail in Chapter 6. The estimation of the ILSAX model parameters is discussed in Sections 4.3.1 to 4.3.3.

4.3.1 Pervious and Impervious Area Depression Storage

Depression storage is a volume that must be filled prior to the occurrence of runoff on both pervious and impervious areas. It represents a loss or an initial abstraction caused by such phenomena as surface ponding, surface wetting, interception and evaporation. Depression storage of directly connected impervious areas may be derived from rainfall-runoff data by plotting runoff depth versus rainfall depth for storm events (Alley and Veenhuis, 1983, U.S. Environmental Protection Agency, 1992, and Boyd *et al.*, 1993). This method will be discussed in detail and used in Chapter 7 for study catchments. However, there is no such method available to compute the pervious area depression storage and therefore, it is necessary to treat the pervious area depression storage as a calibration parameter if rainfall/runoff data are available for the catchment.

As will be discussed in Section 8.2, Kidd (1978a) developed a regional regression equation to compute the directly connected impervious area depression storage using data from European catchments. In this equation, the directly connected impervious area depression storage is expressed as a function of the catchment slope. However, the directly connected impervious area depression storage does not depend only on the catchment slope, but also on land-use type, physiographic condition of surfaces, etc. Due to these reasons, the applicability of the equation for Australian catchments is questionable. However, the U.S. Environmental Protection Agency (1992) suggested the use of this equation to compute the directly connected impervious area depression storage, in the absence of better information.

4.3.2 Infiltration Parameters

As explained in Section 4.2.1, the ILSAX model uses the Horton's infiltration equation to compute the infiltration losses from pervious areas. Although it is one of the well-known infiltration equations available, there is little guidance to determine parameters f_o, f_c and k for a particular catchment. Some guidance is available for estimating f_c based on the soil group (U.S. Environmental Protection Agency, 1992). The parameters f_c and k depend on the soil and vegetation. Ideally, these parameters should be estimated using results from field infiltrometer tests for several sites of the catchment. They can be estimated without any reference to a particular storm. However, the results from such infiltrometer tests are not generally available for use in urban catchments. The parameter f_o, on the other hand, is storm dependent or should be known prior to the storm event. Hence, it is not practical to determine f_o through field infiltrometer tests prior to the storm event. In the absence of such infiltrometer measurements, these parameters have to be calibrated using rainfall-runoff data of storm events for gauged catchments. As stated in Section 4.2.1, it is possible to estimate f_o from Table 4.2 using 5-day prior rainfall depth.

As stated in Section 4.2.1, in the ILSAX model, the user can input either the Horton infiltration parameters (f_o, f_c and k), and four AMC values or an infiltration curve from four pre-defined curves (identified by integer or non-integer CN between 1-4) according to soil type. The latter method is preferred since it involved one curve number and one AMC value for calibration compared to four parameters in the former.

4.3.3 Other Parameters

N_p and N_r can be obtained from the ILSAX user's manual or other literature since they are fairly standard values. Pit capacity parameters are available for few configurations of grade pits in the literature (The Institution of Engineers, 1987; O'Loughlin *et al.,* 1992a; O'Loughlin, 1993, Pezzaniti *et al.,* 1999). Pit capacity parameters for sag pits are difficult to find from the literature except for one sag pit configuration given in the ILSAX user manual. There is not much information about the values for CF in the literature except one value each has been suggested for grade and sag pits in the ILSAX user manual. Therefore, further hydraulic model studies should be conducted to define these parameters for different common configurations of all types of pits.

GUT can be computed from geometry of kerb and surface roughness of gutter using Equation 4.9. In ILSAX, GUT should be entered into the Pipe file as data. However, there is no guidance given in the ILSAX user manual to choose the value for GUT. After discussions with engineers in City/Shire councils in Victoria on typical gutters and their dimensions, the GUT factor was calculated by the author for these typical sections. These GUT factors are given in Table 4.3.

Table 4.3: GUT Factor for Typical Gutter Section in Victoria

Gutter Section	d_G (mm)	Z_G	Z_p	n_G	n_p	GUT
1	150	8.0	40	0.012	0.014	10
2	150	8.0	30	0.012	0.014	11
3	150	7.5	40	0.012	0.025	4
4	150	33.0	40	0.012	0.025	12

Notes:

Z_G is the gutter cross slopes (m/m), (Figure 4.5a),

Z_P is the pavement cross slopes (m/m), (Figure 4.5a),

n_G is the Manning roughness coefficient of gutter, (Figure 4.5a),

n_p is the Manning roughness coefficient of pavement,

d_G is the greatest gutter depths (m), and

d_P is the greatest pavement depths (m).

4.4 DRAINS MODEL

DRAINS (O'Loughlin and Stack, 1998) is the successor to ILSAX. It runs on PCs with Microsoft Windows 95 and NT systems. It is a standard windows package. The DRAINS model provides a Windows graphical interface. Users can define the drainage system components such as subcatchments, pits, pipes and overflow routes. Right clicking on a component will display a pop-up menu from which user can choose to enter data or view results in various formats. The ARR87 design rainfall patterns can be entered separately. Results such as runoff hydrographs are displayed graphically and can be pasted into other Windows program such as spreadsheets and word processors. It produces summary graphs and tables, and longitudinal sections of pipes.

The DRAINS model has almost all features of the ILSAX model, including the modelling of pit bypass flows and detention storages. The major modelling differences in DRAINS compared to ILSAX are:

- Modelling of depression storage in supplementary areas and
- Employing hydraulic grade line (HGL) method for modelling of pipe system hydraulics.

The other enhancements are:

- A main window where drainage system components can be added and viewed,
- A variety of outputs providing results in formats easily transferred to reports and drawings,
- Help and checking systems, and
- Preparation of design drawings.

According to O'Loughlin and Stack (1998) the next enhancement of this model will include rural hydrology, full hydrodynamic modelling, sanitary sewer and property drainage design, stormwater quality modelling and many other design and investigation tasks enabling it to model small-scale property drainage systems, rural runoff and sanitary sewers.

Although the DRAINS model is a more recent version of ILSAX, it was not used in this study, since the project was started prior to the release of the DRAINS model. However, the procedures that were used in this thesis can be extended to the DRAINS model, since the hydrology and the most of hydraulics of the two models are similar.

CHAPTER 5

DATA COLLECTION AND ANALYSIS

5.1 INTRODUCTION

Good quality hydrologic and physical data are required to calibrate and test the parameters of hydrologic models. It is understood that good quality hydrologic data are not readily available for most urban drainage catchments. The Victoria University of Technology (VU), in collaboration with ten widely dispersed local government city/shire councils in Victoria, conducted a major data acquisition program for 26 major urban catchments. Up to three major catchments were selected from each city/shire council. These 26 catchments were selected to suit the specific needs of each collaborating city/shire council and to ensure that different catchment characteristics and hydraulic systems were represented. The catchments were located within the collaborating city/shire council boundaries, and these boundaries are shown in Figure 5.1. Although 26 major catchments were monitored for rainfall/runoff data, only the catchments in the Melbourne metropolitan area were considered in this thesis, since one of the aims is to regionalise the ILSAX model parameters for use in Melbourne metropolitan area. Since 'large' rainfall events from the Melbourne metropolitan catchments were not recorded during the monitoring period (i.e. 1996 to 1999), the Giralang catchment from Canberra, which had rainfall/runoff data for a long period including 'large' storm events, was also selected.

Catchment and hydraulic data were compiled from several sources including drainage, contour, land use and soil maps, areal photographs, site visits, and drainage design and asset management information. These catchments were monitored continuously for rainfall and runoff during storm events since July 1996. The magnitude and the temporal variation of rainfall were measured with automatic pluviometers installed close to the centroid of each catchment.

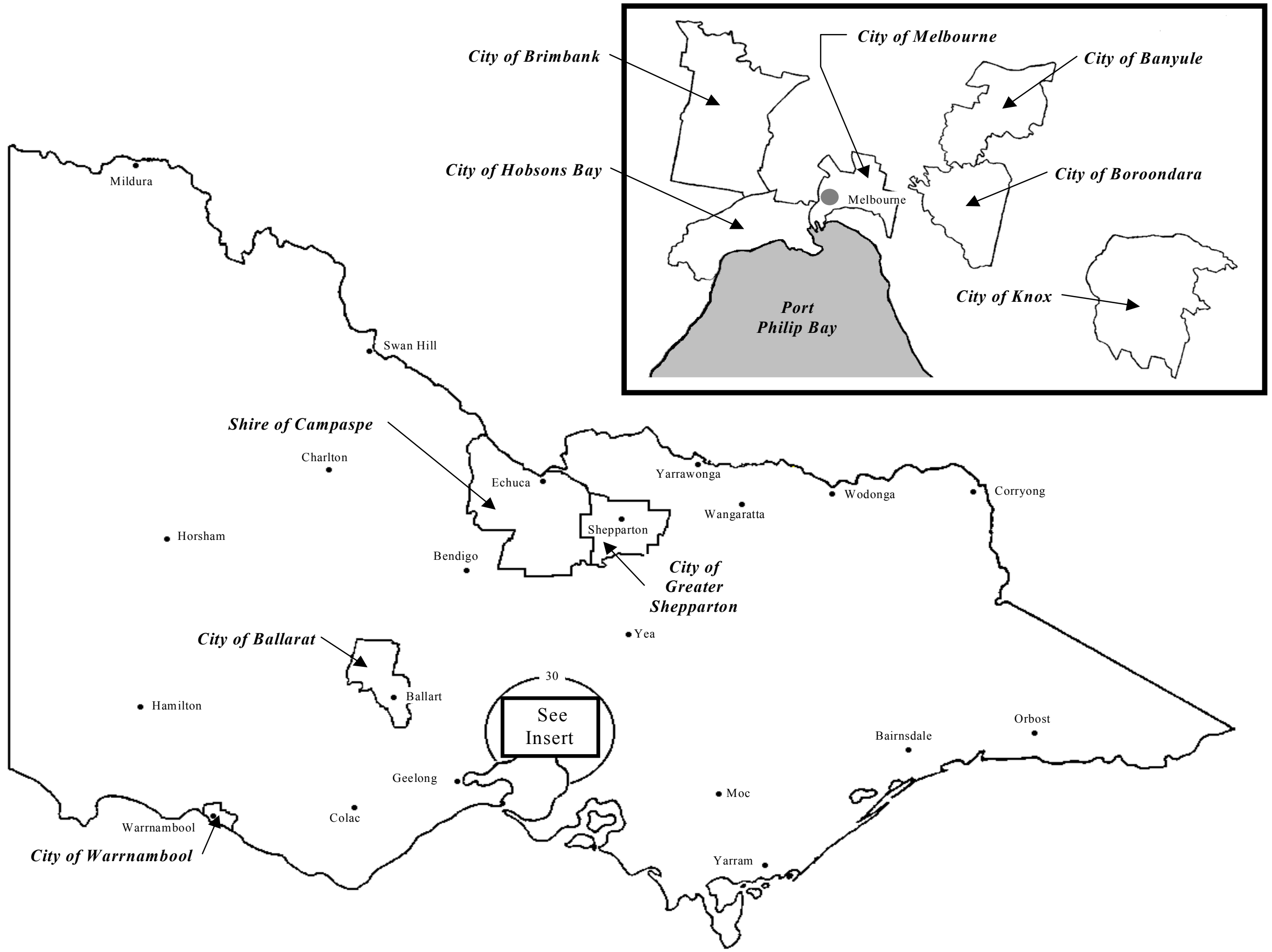


Figure 5.1: Locations of Study Catchments

Stormwater runoff in each catchment was continuously monitored at the outlet of the catchment and on average at two upstream locations, using Ultrasonic Doppler flowmeters. The rainfall/runoff data for storm events were collected until the end of 1999.

The accuracy of the results of any modelling exercise largely depends on the accuracy of the data used. For this reason to calibrate the rainfall-runoff models, accurate rainfall-runoff data and catchment data are required. Therefore, the rainfall and runoff data acquired in the data acquisition project were checked for accuracy and consistency. This chapter describes the study catchments used in this thesis and the collection of hydrologic (i.e. rainfall/runoff) and other data required for modelling.

5.2 STUDY CATCHMENTS

Eleven major urban drainage catchments from 5 city councils located within a 30-km radius of Melbourne Central Business District (MCBD) were selected for this study. These major catchments were monitored for rainfall and runoff data. The five councils were Banyule, Borrondara, Brimbank, Hobsons Bay and Knox (Figure 5.1). Catchments from Melbourne City council were not selected, although three catchments were monitored for rainfall and runoff data. This is because the drainage systems of these monitored catchments had looped pipes, which cannot be modelled by ILSAX and also the pipe drainage system were changed during the monitoring period. These 11 major catchments had 11 subcatchments, which were also monitored. Therefore, a total of 22 catchments were considered as study catchments and all these catchments had rainfall data at a representative location and flow data at the catchment outlets. In addition to these 22 Melbourne metropolitan catchments, the Giralang catchment in Canberra was also selected as a study catchment. This is because the Giralang catchment had 'large'storms recorded, while the Melbourne metropolitan catchments did not have the 'large' events during the monitoring period. Large storms are required to calibrate the pervious area parameters. The key characteristics of these catchments are given in Table 5.1. These characteristics include catchment code, catchment name, council, catchment area, land-use and other details.

Table 5.1: Characteristics of Study Catchments

Catchment Code	Catchment	Council	Catchment Area (ha)	No. of Subcatchments	Major Pipe Sizes (mm)	Total Pipe Length (m)	Total Gutter Length (m)
BA2	Heidelberg	Banyule	45	24	300-1050	2131	3882
BA2A			14	14	300-675	1067	2322
BA3	Karingal Reserve		43	37	300-825	2776	4882
BA3A	Greenbourough		30	22	300-750	1589	3319
BA3B			11	5	300-450	799	1787
BO1A	Balwyn North		3	3	300-450	270	730
BO2A	Kew		5	3	300-450	200	550

Table 5.1 Continued ....

Catchment Code	Catchment	Council	Catchment Area (ha)	No. of Subcatchments	Major Pipe Sizes (mm)	Total Pipe Length (m)	Total Gutter Length (m)
BR1	Delahey	Brimbank	8	12	375-750	610	2032
BR1A			4	5	375-675	212	969
BR2	Kealba		39	16	375-900	1597	4082
BR2A			14	12	375-750	1037	2482
BR3	Sunshine		20	7	750-1200	962	2746
H2	Altona Meadows	Hobsons Bay	14	28	375-2@450	1125	2225
H2A			7	12	375-525	544	994
K1	Boronia	Knox	22	18	300-750	1611	3705
K1A			10	10	300-675	472	1024
K1B			5	4	300-450	455	988
K2	Ferntree Gully		30	19	450-675	2093	3980
K2A			16	11	450-600	1241	2730
K3	Rowville		41	76	300-1050	4293	8458
K3A			9	28	300-525	1871	3829
K3B			14	5	300-525	930	2144
GI	Giralang	Canberra	94	14	300-1950	2730	4380

The major catchment BO1 had undergone changes to land-use conditions during the monitoring period. This was because the subcatchment BO1B of the major catchment BO1, which was previously bare land was converted to a housing estate during the monitoring period. BO1B had limited data on rainfall/runoff and BO1 had non-homogeneous data. Therefore, only BO1A was considered in this study and listed in Table 5.1.

The catchment code is made of the council name (e.g. BA for Banyule, K for Knox etc.), major catchment number (e.g. 1, 2, 3 etc) and flow meter number of the subcatchment (e.g. A, B etc). For the major catchment, the flow meter number is not included in the catchment code. For example, the catchment code BA3 indicates the third major catchment in Banyule city council. The catchment tag BA3A indicates the first subcatchment of the third major catchment in Banyule city council, while BA3B indicates the second subcatchment.

The study catchments and their drainage layouts are shown in Appendix B. These catchment plans show the catchment and subcatchment boundaries, flow meter and pluviometer locations, and main and secondary drainage lines. The catchment areas of these drainage systems were measured from the catchment plans using planimeters and vary from 3 ha to 45 ha. The slopes of the catchments were measured from contour maps and construction drawings. The average land slopes in the catchments vary from 0.3% to 6%. The catchment imperviousness was computed from the information in areal photographs and land use maps and found to be 29 - 80%. The land use conditions of these catchments are residential, industrial, commercial or a mixture of these.

5.3 DATA REQUIREMENTS FOR ILSAX MODELLING

The ILSAX model requires hydrological data, physical properties of the catchments and stormwater drainage system details for modelling. Two data files namely, the Run and Rain file, and the Pipe file are prepared based on above data. The Run and Rainfall file provides information such as catchment soil type, depression storage values for different surfaces of the catchment, pipe roughness, antecedent moisture conditions, and rainfall and runoff data of storm events. The Pipe file contains the information on each reach of the stormwater network, such as diameter, length, slope and pipe roughness of pipe (i.e. only if it is different from the global value given in the Run and Rainfall file), pit inlet type and associated parameters in pit entry capacity calculations, subcatchment area details (i.e. total area of the subcatchment, and pervious, impervious and supplementary area percentages), times of entry for pervious and impervious areas, detention basin data, etc.

5.4 DATA COLLECTION AND PRELIMINARY ANALYSIS OF MELBOURNE METROPOLITAN CATCHMENTS

5.4.1 Rainfall/Runoff Data Collection

One of the major purposes of the data collection program conducted by VU was to develop and calibrate urban drainage models for the monitored drainage systems. Therefore, it was necessary to collect complete rainfall hyetographs and runoff hydrographs for storm events. Therefore, the catchments were continuously monitored for rainfall and runoff storm events. The details of the monitoring program are found in Maheepala *et al*. (1998), Maheepala *et al*. (1999), Maheepala and Perera (1999), Maheepala (1999). Some details are given below. The rainfall/runoff data collection program was conducted as a separate project, not as part of this PhD project.

Automatic electronic tipping bucket type pluviometers with 0.2 mm accuracy (i.e. one tip of the bucket is equivalent to 0.2 mm of rainfall over the catchment) were used to monitor the temporal pattern and the magnitude of storm events. These pluviometers were installed close to the centroid of each catchment. The rainfall data records stored in pluviometers were downloaded into a notebook computer at two-month intervals. The magnitude and the temporal variation of storms were then obtained from these records.

For each of the 22 catchments, the stormwater runoff was monitored at the catchment outlet. Ultrasonic Doppler type flowmeters were installed on the inverts of drainage conduits to measure the flow depth and velocity, continuously at two-minute intervals. The measured data were stored in the flowmeters and downloaded into a computer at monthly intervals.

5.4.2 Rainfall/Runoff Data Analysis

Raw hydrologic data (i.e. rainfall/runoff data) acquired from the data acquisition program were carefully checked for accuracy and consistency as part of the program. Whenever possible, the collected rainfall data were checked against independent rainfall data obtained from other nearby measuring stations operated by Melbourne Water and Bureau of Meteorology, for selected 'large' storm events. After the data were downloaded from pluviometers and flowmeters, they were manually checked for apparent malfunctioning of instruments, and errors and inconsistencies in the raw data.

The storm events were selected from the runoff data series by removing the data recorded during dry periods. The rainfall data for storm events and the corresponding runoff data were then checked for consistency in terms of temporal trends. Graphical time series plots of recorded flow depth and velocity were used in this preliminary data checking process. Figure 5.2 shows a time series plot showing temporal trends of velocity and depth of flow for a selected storm event together with rainfall of the event. The flow velocity measured at a given monitoring point should increase as the flow depth increases and vice versa, as shown in Figure 5.2.

The quality of rainfall/runoff data can be best observed by plotting rainfall and runoff in the same chart. When a catchment has several monitoring stations as the case in this data acquisition program (i.e. subcatchments within the major catchment), the data obtained at the outlet and at other upstream monitoring points can be plotted as shown in Figures 5.3, together with the rainfall of the event. These plots can be used to detect timing errors of rainfall/runoff data of storm events. As shown in Figure 5.3, the runoff peak should occur some time after the rainfall peak. Furthermore, the time of concentration (i.e. approximately estimated as the time between peak rainfall and peak runoff) of internal subcatchments should be smaller than that of the whole catchment.

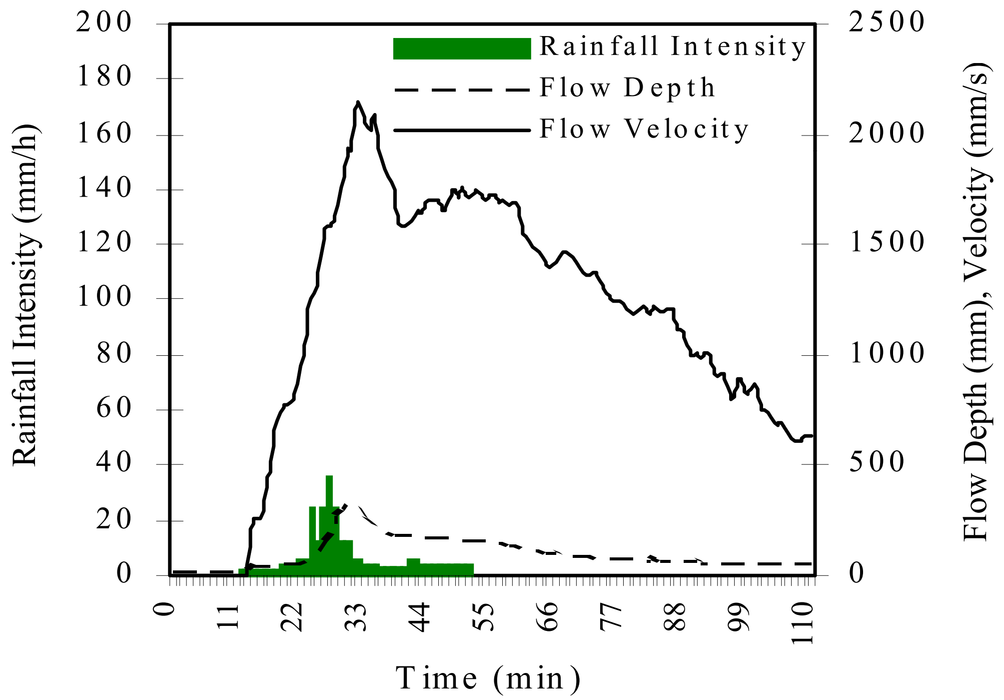


Figure 5.2: Time Series Plots of Measured Flow Depth and Velocity

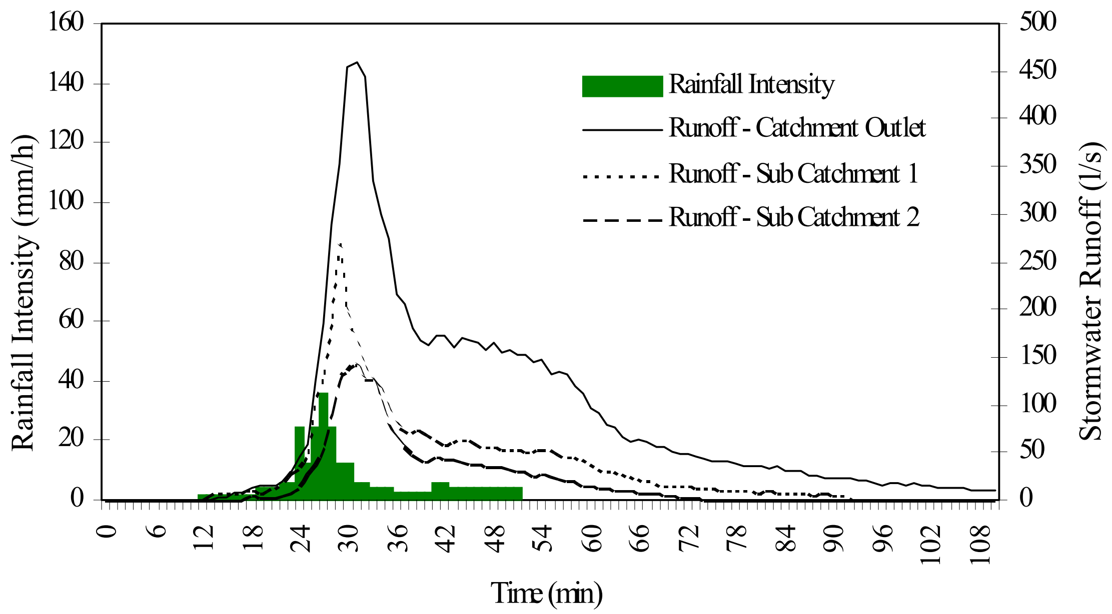


Figure 5.3: Rainfall Hyetograph and Runoff Hydrograph Plots for a Storm Event

These plots are good indicators of the accuracy and consistency of rainfall and runoff data. They were prepared for all storm events of the study catchments, as part of the monitoring program.

After each data download, the rainfall data for storm events and the corresponding runoff data were checked for consistency in terms of rainfall and runoff volumes. For each storm event, rainfall and stormwater runoff depths of each catchment were computed and compared. The runoff depth should be always less than the rainfall depth.

Further data analysis was conducted under this PhD project to select the storm events for calibration and verification of urban drainage models of the study catchments. These data analyses are explained in Section 7.2.3.2.

5.4.3 Catchment and Drainage System Data Collection

The physical data of the catchments and their stormwater drainage systems were also collected for modelling of study catchments. This was conducted as part of this PhD project. These data include pipe system and land-use layout, catchment areas, percentage impervious and pervious areas, soil type or infiltration characteristics of catchment soils, topography of the catchments, lengths of overland flow paths, and dimensions, slopes and roughness parameters of drainage conduits, etc. These data were compiled from several sources such as drainage, contour, land-use and soil maps, areal photographs, drainage design and asset management information, reports on previous studies, VicRoad road directory and site visits.

5.5 DATA OF GIRALANG CATCHMENT

As stated in Section 5.2, the Giralang catchment was used to demonstrate the model calibration process for 'large' storm events. Urbanisation of this catchment was fully completed in 1976. The areal maps used in this study had been photographed in 1976. Since then, there were not significant changes to the catchment (Laurenson *et al.,* 1985). The drainage plans used to extract data for modelling work in this study were prepared in

1976. Therefore, the rainfall-runoff data of the Giralang catchment after 1976 were considered for this study. Clearly defined ‘small’ and ‘large’ storm events were selected from the rainfall-runoff database.

5.6 SUMMARY

Good quality hydrologic data (i.e. rainfall and runoff data) should be used in developing computer models for urban drainage systems, in order to obtain accurate results and reliable predictions. Once the data are corrected, they should checked for their accuracy and consistency. Several methods had been employed to check the data consistency and accuracy of the study catchments. Therefore, good quality hydrologic data were available for both Melbourne metropolitan and Giralang catchments. In addition to hydrologic data, physical data of these catchments and their drainage systems were collected. These data were compiled from several sources such as drainage, contour, land-use and soil maps, areal photographs, drainage design and asset management information, reports on previous studies, VicRoad road directory and site visits.

CHAPTER 6

INVESTIGATION OF ILSAX MODELLING OPTIONS AND MODEL PARAMETER SENSITIVITY

6.1 INTRODUCTION

As stated in early chapters, the ILSAX model was used in this study. In modelling certain hydrologic and hydraulic processes, the ILSAX model allows the user to select one of the available modelling options. These options were described in detail in Section 4.2. They were studied in this chapter with the aim of selecting one option each for each of those processes with several modelling options. Once the preferred modelling options were selected, they were used in Chapter 7 for estimating the model parameters of the study catchments. In order to use the ILSAX model, like any other mathematical model, the model parameters should be known. The model parameters are generally estimated by calibration, if the catchments are gauged for rainfall and runoff. Otherwise, they are estimated from the information in the published literature. To calibrate the models effectively, it is important to know the sensitivity of model output to each model parameter. Similarly, when a model is applied to an ungauged catchment, care should be taken to select the "accurate" values for all sensitive model parameters. This information can be obtained from a sensitivity analysis of the model parameters.

Sensitivity analysis of model parameters requires the investigation of changes in the model output to the changes in model parameters. Such an analysis is required as part of an effort to increase the understanding of a modeller's knowledge of the processes considered in the model. It may also be the first step of a model calibration exercise, whereby the key model parameters are identified. This chapter also dealt with a detailed study to identify the most sensitive model parameters of the ILSAX model. These sensitivity parameters were given careful attention in Chapter 7 in model calibration.

The level of catchment subdivision is important in the analysis of urban drainage systems using rainfall-runoff models. The level of catchment subdivision indicates the level of information to be considered in modelling. The suitable catchment subdivisions found in the literature were also tested in this chapter in order to select the best catchment subdivision for use in Chapter 7.

In this chapter, the methodology used for these three investigations (i.e. modelling options, parameter sensitivity and catchment subdivision) is described first, followed by catchment and storm event selections for the study. Detailed studies conducted on different modelling options, the sensitivity of model parameters and the effect of catchment subdivision on storm hydrograph are presented then. Finally, conclusions drawn from this detailed investigation are outlined for use in Chapter 7.

6.2 METHODOLOGY USED

The methodology used for all three investigations (i.e. ILSAX modelling options, sensitivity of ILSAX model parameters and effect of catchment subdivision) is similar. A base run was first defined considering a suitable option for those hydrologic and hydraulic processes which have more than one modelling option (available in ILSAX), some suitable base values for model parameters and a suitable catchment subdivision method. Different ILSAX modelling options, different model parameters and different catchment subdivisions were then carried out and their results were compared with those obtained from the base run.

The base run was defined using the following modelling options. The selection of these modelling options was qualitatively justified in Section 4.2.

- Loss subtraction from supply rate, for pervious area loss modelling.
- The ARR87 method for overland flow routing,
- The implicit method for pipe routing, and
- The finite capacity for inlets.

The base run model parameter values for CN, AMC, DS_p and DS_i were selected as the middle value of the recommended range in the ILSAX manual (O'Loughlin, 1993). The N_p, N_r and CF values were selected as 'reasonable' best values for the catchments from the recommended range in the ILSAX manual. A reasonable GUT value was selected as the base value from the calculated GUT range (Section 4.3.3). CAP3 and CAP4 values were taken from O'Loughlin *et al.* (1992a). These base run parameters are shown in Table 6.1. Fine subdivision was used in the base run.

Table 6.1: Base Run Model Parameter Values

Parameter	Value
DS_p (mm)	5.0
DS_i (mm)	1.0
CN	2.5
AMC	2.5
N_p	0.012
N_r	0.2
CF	0.2
GUT	10
CAP3	0.345
CAP4	0.738

One 'small' and one 'large' catchment, selected from the major Melbourne metropolitan catchments were considered in this detailed investigation. At the time of this investigation, 'large' storm events producing runoff from pervious areas had not been collected in the stormwater drainage monitoring program (Section 5.2). Therefore, four design storms of different magnitudes were considered in this study.

At the catchment outlet, there are two hydrographs that need to be considered in this investigation to model the total catchment runoff. First is the hydrograph representing pipe flow (i.e. catchment outlet is considered as this pipe) and the other representing the bypass flow from the last pit (which is just upstream of the catchment outlet). The ILSAX model produces the complete pipe flow hydrograph, but does not give the full hydrograph for bypass flow. For bypass flow, only the peak discharge and runoff volume corresponding to a storm event are given. For the sensitivity analysis of the hydrological parameters, the total catchment runoff volume (both bypass and pipe flow volume) and pipe flow hydrograph were considered. However, for sensitivity analysis of the routing parameters, only the pipe flow hydrograph was considered. In the remainder of the chapter, peak discharge refers to peak of the pipe flow hydrograph, and runoff volume refers to both pipe flow and bypass flow volume.

6.3 CATCHMENT SELECTION

The Altona Meadows catchment (H2) in City of Hobsons Bay and the Therry Street catchment in City of Melbourne were selected for this detailed investigation. The Altona Meadows catchment was also considered in Chapter 7 in model parameter optimisation and regionalisation. However, the Therry Street catchment was not considered in Chapter 7, due to lack of adequate good quality rainfall/runoff data. The catchment areas of the Altona Meadows and the Therry Street catchments are 14 ha and 56 ha respectively with 80% and 91% imperviousness. The Therry Street catchment is the largest catchment of the monitored drainage catchments (Section 5.1), while the Altona Meadows catchment is the smallest major catchment. These two catchments represent typical ‘small’ and ‘large’ urban drainage catchments.

For both catchments, detail investigation was carried out by comparing the hydrographs at the outlet (i.e. pipe just downstream of the last pit). For the Altona Meadows catchment, the outlet pipe was not full due to different storm events conducted in the investigation. Therefore, the actual pipe diameters were considered in the analysis of this catchment. However, the initial ILSAX runs for the Therry Street catchment showed that the outlet pipe was full for storm events with ARI (i.e. Average Recurrence Interval) greater than 2 years and produced the same peak discharge at the outlet for these storms. This restricts the peak discharge comparisons in this study. Therefore, the underground pipe drainage system of the Therry Street catchment was first designed for the 100 year ARI and then used for the purpose of this study. The drainage system with new diameters produces different hydrograph peaks for different magnitude storms up to 100 year ARI.

6.4 STORM EVENT SELECTION

As stated in Section 5.2, the selected two catchments were monitored for rainfall and runoff since 1996, as part of the stormwater data acquisition program of VU. However, at the time of this investigation, 'large' storm events producing runoff from pervious areas had not been recorded during the monitoring period. Therefore, four design storms of different magnitudes were considered in the study.

By analysing the significant storm events of the stormwater program, it was found that the time difference between rainfall and runoff peaks at the outlet of each of these two catchments was of the order of 25 minutes. Similarly, considering various design storms of different durations with ARR87 temporal patterns, it was also found that the critical storm duration for the two catchments was also 25 minutes. This critical storm duration was computed as the storm duration that produced the highest peak discharge by comparing the runoff peaks due to different storm durations. This is the method recommended in ARR87 to select the critical storm duration. Therefore, four design storms of 25 minutes duration were selected for 1 year, 10 year, 100 year and very large (> 100 year ARI) return periods for each catchment. The Intensity-Frequency-Duration (IFD) curves were developed for the catchments using ARR87. The IFD curves are the same for the two catchments, since they are only 10 km apart. These curves are shown in Figure 6.1.

The rainfall intensities were read from the IFD curves for ARIs of 1, 10 and 100 years, and computed the rainfall depth. However, 70 mm of depth of storm was considered for the very large event (> 100 year ARI) for both catchments. The storm temporal pattern of ARR87 for 25 minutes duration was used to obtain the rainfall distribution (i.e. hyetograph) of these design storms. The details of the selected storm events used in this study are given in Table 6.2. The hydrographs were obtained from running the ILSAX model.

INTENSITY-FREQUENCY-DURATION (IFD) CURVES
FOR
CITY OF HOBSONS BAY

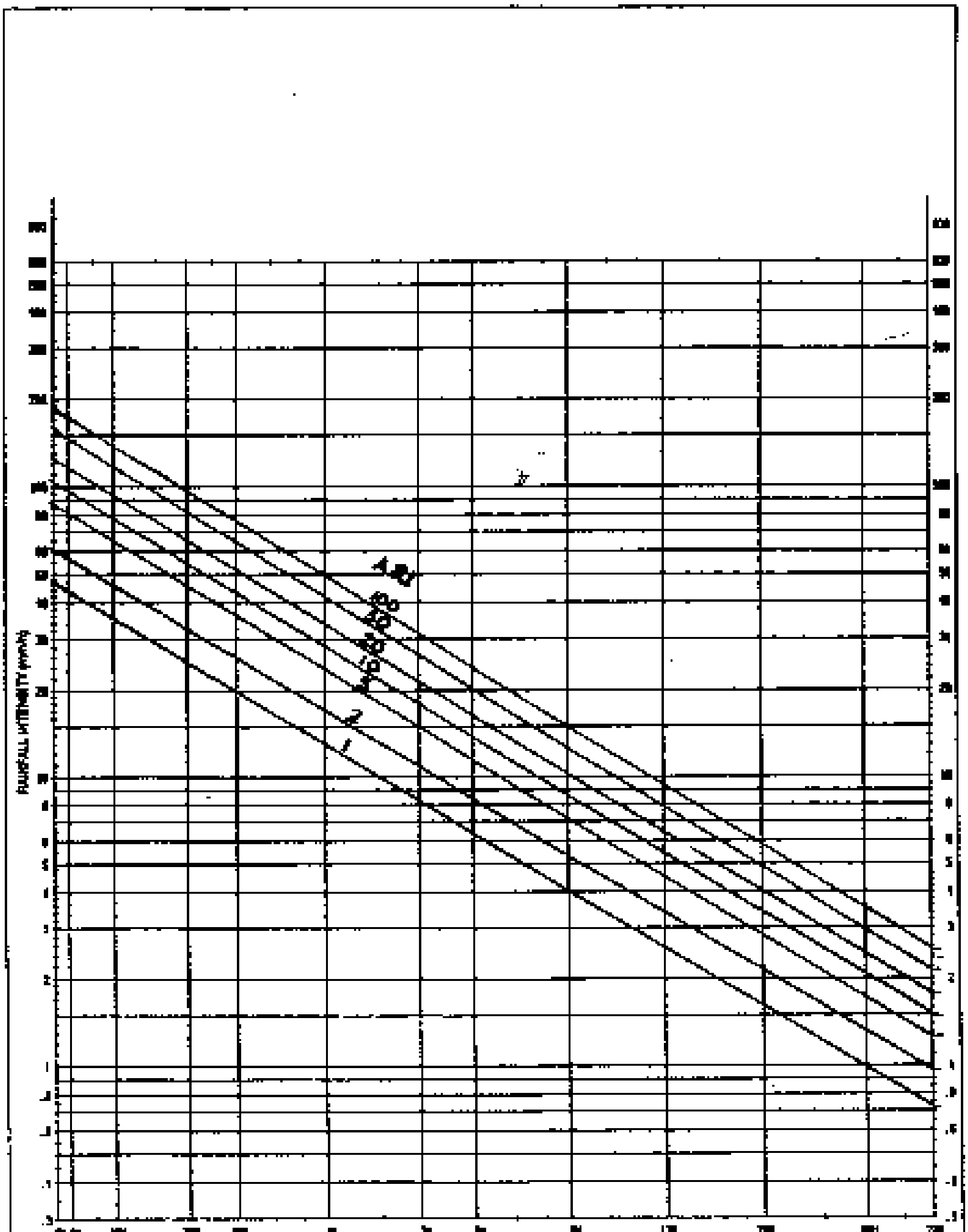


Figure 6.1: IFD Curve for Altona Meadows and Therry Street Catchments

Table 6.2: Selected Storm Events

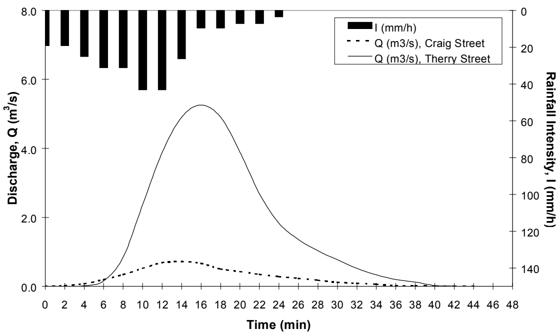
Storm Event No.	ARI (years)	Duration (min)	Rainfall Depth (mm)
E1	1	25	9
E2	10	25	20
E3	100	25	35
E4	> 100	25	70

The hyetographs and hydrographs corresponding to base run parameters for four design storms are shown in Figure 6.2. As seen in Figure 6.2, the outlet pipe of the Altona Meadows catchment is not full for four design storms. The outlet pipe of Therry Street catchment is also not full for first two design storms. However, the outlet pipe of the Therry Street catchment is full for the two 'large' storm events, as expected.

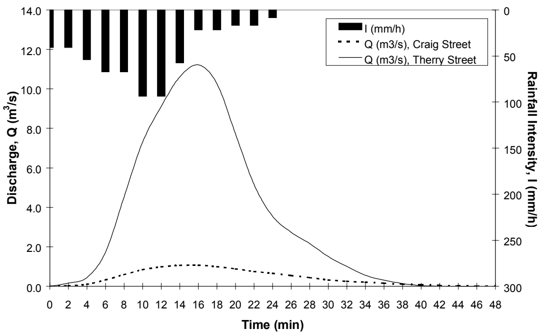
6.5 ILSAX MODELLING OPTIONS

As stated in Section 4.2, different options are available in the ILSAX model for subtraction of pervious area runoff losses, overland and pipe routing, and modelling of inlet pits. However, there are no adequate guidelines to select the most appropriate option for a particular study. Therefore, these modelling options were investigated using the selected design storm events of the two catchments. Once the modelling options were selected, they were used in Chapter 7 for all study catchments.

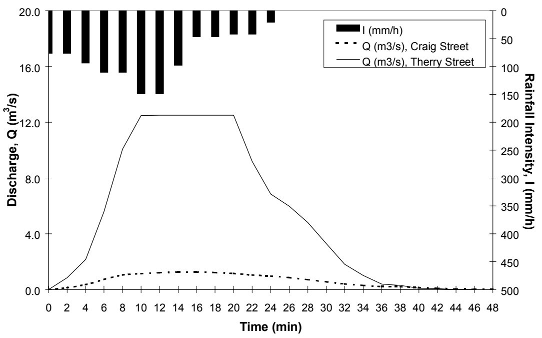
In order to investigate different ILSAX modelling options, the base run options and parameters (Section 6.2) were considered except for the modelling option under investigation. The results from this run were then compared with the base run results (Figure 6.2).



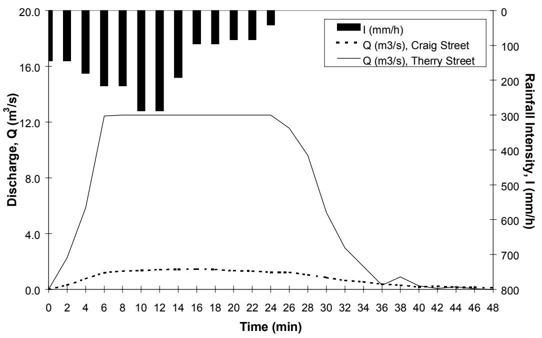
(a) 1 year ARI



(b) 10 year ARI



(c) 100 year ARI



(d) > 100 year ARI

Figure 6.2: Hyetographs and Base Run Hydrographs for Four Design Storms

6.5.1 Pervious Area Runoff Loss Subtraction

As discussed in Section 4.2, in the ILSAX model, the rainfall losses from pervious areas can be subtracted from either the rainfall hyetograph before it is convolved with the time-area diagram or can be subtracted from a "supply rate" determined by first convolving the total hyetograph with the time-area diagram. These options were investigated in this study using the selected design storms. The results in terms of runoff volume, peak discharge and time to peak discharge for both catchments are given in Table 6.3. Figure 6.3 shows the effect of these methods on runoff volume for both catchments, since only the runoff volume changes with the loss subtraction option used. This figure gives the percentage difference of runoff volume for the loss subtraction from rainfall compared to loss subtraction from supply rate. This form of percentage difference was selected since the base run considered the pervious area runoff loss subtraction from supply rate. There is no difference in runoff volume produced by the two loss subtraction methods for the first two storm events (E1 and E2) of both catchments, which suggests that there is no pervious area runoff for these two storm events. This was confirmed by examining the pervious area runoff produced in the model output.

For the two largest storm events (E3 and E4), the first method (losses subtracted from rainfall) gives higher runoff volumes compared to the second method (1.7% and 1.2% increase for the Altona Meadows catchment and 0.01% and 0.04% increase for the Therry Street catchment respectively). Although these catchments are typical urban catchments in the Melbourne metropolitan area in terms of their area, they have fairly small pervious areas (20% and 9% respectively for Altona Meadows and Therry Street catchments). If pervious areas are larger, these differences can become larger.

For each design storm, the results of the investigation showed that the peak discharge and the time to peak discharge from the two methods were the same. The simulated runoff discharge ordinates up to 25 min (which is the storm duration for all storms) should be the same from both methods. Peak of the hydrograph generally occurs during the storm duration and therefore the peak discharge and the time to peak discharge should be the same from both methods. After 25 min, the ordinates of the hydrograph corresponding to loss subtraction from rainfall should produce higher values than the subtraction from supply rate since the latter method considers further losses. This explains the higher runoff volumes in the former method.

Table 6.3: Effect of Different Loss Subtraction Methods

Storm Event Number	Losses Subtraction Method	Altona Meadows Catchment			Therry Street Catchment		
		V (m^3)	P (m^3/s)	TTP (min)	V (m^3)	P (m^3/s)	TTP (min)
E1	From rainfall	939	0.718	18	4261	5.253	18
E2		2186	1.068	18	9932	11.210	18
E3		4137	1.247	18	18408	12.495	14
E4		8762	1.443	18	38492	12.495	10
E1	From supply rate	939	0.718	18	4261	5.253	18
E2		2186	1.068	18	9932	11.210	18
E3		4067	1.247	18	18405	12.495	14
E4		8656	1.443	18	37477	12.495	10

V= Runoff volume, P= Peak discharge, TTP= Time to peak discharge

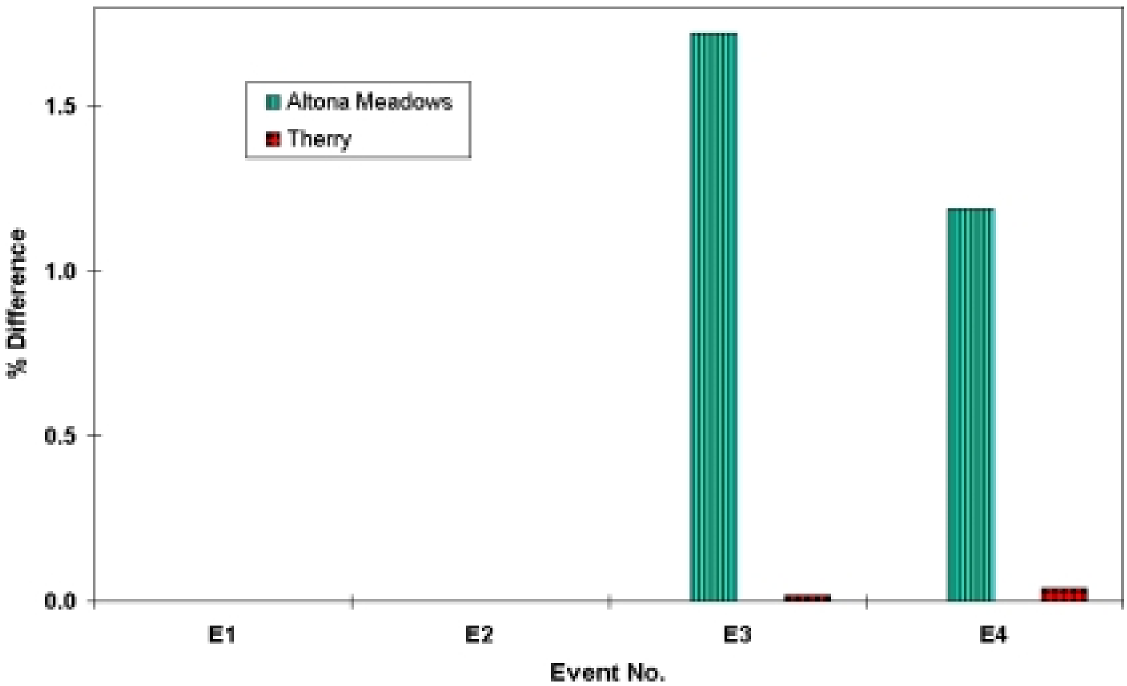


Figure 6.3: Effect of Loss Subtraction Methods on Runoff Volume

The peak discharge of the Therry catchment for storms E3 and E4 were the same. This was to be expected since the pipe was full during these storms. This was because the Therry catchment pipe system was designed for a 100 year ARI storm event for the purpose of this study.

Data requirements for the two methods are the same. However, in reality, stormwater travels over the pervious surfaces even after rainfall stops, and thus gives a better representation of the physical processes. Therefore, the method that subtracts losses from the supply rate is recommended and was used in Chapter 7.

6.5.2 Overland Flow and Pipe Routing

There are two options (i.e. ILLUDAS-SA procedure and ARR87 method) available in the ILSAX model to compute the time of entry to a pit for overland flow routing and two options (i.e. time-shift and implicit methods) for pipe routing.

6.5.2.1 Time of entry for overland flow routing

The time of entry is required to set the base lengths of pervious and impervious area time-area diagrams (O'Loughlin, 1993). The reader is referred to Section 4.2 for details of time of entry for overland flow routing. The two modelling options for computing time of entry for overland flow routing were studied using the above design storms (Section 6.4) on the selected two catchments (Section 6.3). The peak discharge and time to peak discharge obtained from the ILSAX model are given in Table 6.4. The runoff volume due to these two methods should not be different, since the rainfall excess is the same for both methods. Figure 6.4 shows the effect of these methods on peak discharge for both catchments, since only the peak discharge changes with the overland flow routing method. This figure gives the percentage difference of peak discharge of the ILLUDAS-SA method compared with the ARR87 method. This form of percentage difference was selected since the base run considered the ARR87 method for overland flow routing.

Table 6.4: Values for Different Time of Entry Methods

Storm Event Number	Time of Entry Method	Altona Meadows Catchment		Therry Street Catchment	
		P (m^3/s)	TTP (min)	P (m^3/s)	TTP (min)
E1	ILLUDAS-SA	0.731	16	5.404	18
E2		1.086	16	11.654	18
E3		1.258	16	12.495	12
E4		1.437	16	12.495	10
E1	ARR87	0.718	18	5.253	18
E2		1.068	18	11.210	18
E3		1.247	18	12.495	14
E4		1.443	18	12.495	10

P= Peak discharge, TTP= Time to peak discharge

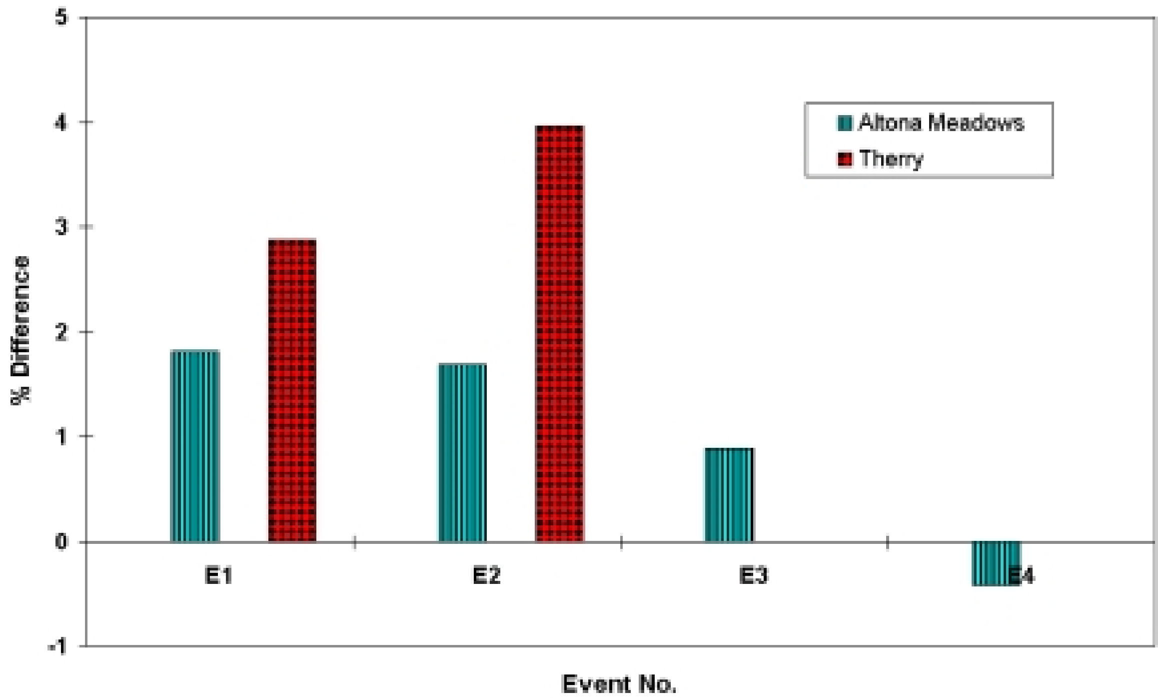


Figure 6.4: Effect of Time of Entry Methods for Overland Flow Routing on Peak Discharge

The results of the study showed that the ILLUDAS-SA procedure produced higher peak discharges and equal or lower times to peak discharge compared to the ARR87 method for all storms of both catchments, except the largest storm event of the Altona Meadows catchment. Two largest storm events of the Therry Street catchment gave the same peak discharge because the outlet pipe was full, as described earlier (Section 6.5.1).

From the information given in the ILSAX user's manual, the times of entry of impervious and pervious area from these two methods can be derived from Equations 6.1-6.4.

$$[TE_{impervious}]_{ILLUDAS\text{-}SA} = (0.002 * S^{-0.5} * L + 2) \quad (6.1)$$

$$[TE_{pervious}]_{ILLUDAS\text{-}SA} = (0.32 * S^{-0.3} * L^{0.6}) \quad (6.2)$$

$$[TE_{impervious}]_{ARR\ 87} = (0.02 * L * S^{-0.5} * GUT^{-1} + t_o) \quad (6.3)$$

$$[TE_{pervious}]_{ARR\ 87} = [6.94 * (L * N)^{0.6} * I^{-0.4} * S^{-0.3}] \quad (6.4)$$

where TE is the time of entry (min),
S is the slope of flow path (m/m),
L is the length of flow path (m),
GUT is the gutter factor (Section 4.3.3),
t_o is the property time,
N is the surface roughness (or retardance factor), and
I is the rainfall intensity (mm/h).

Equations 6.1 and 6.3 show that the times of entry of impervious areas from the ILLUDAS-SA and ARR87 methods produce the same results if GUT and property time in the ARR87 method are assumed at 10 and 2 min respectively. Equations 6.2 and 6.4 show that times of entry of pervious areas from the ILLUDAS-SA method is independent of rainfall intensity while the ARR87 method considers the rainfall intensity. Therefore, the ILLUDAS-SA procedure should give the same time of entry for all storms, while the ARR87 method produces the time of entry, which decreases as magnitude of the rainfall intensity increase. This fact is not clearly seen in results of either Table 6.4 or Figure 6.4, since the results are with respect to the outlet. The hydrograph attributes at the outlet depend on many processes other than the time of entry. They include pipe routing, bypass flow, overflow etc. Also, the computational time step used in modelling was 2 minutes and this time step does not show the difference of the two methods on time to peak discharge, because of rounding errors.

The data requirements of the ILLUDAS-SA method are low compared to the ARR87 method. Although, the percentage differences of peak discharge from these two methods are small, they increase with increase in catchment size (i.e. percentage difference of peak discharge for the Altona Meadows and Therry Street catchments for Event E1 was 1.7% and 2.8% respectively while these values for Event E2 were 2% and 4% respectively). Therefore, for accurate modelling of peak discharge, the ARR87 method is recommended and was used in Chapter 7.

6.5.2.2 Pipe routing

The ILSAX model provides two options for pipe routing, namely the time-shift method and the implicit method (Section 4.2.3). The time-shift option simply calculates the flow travel time through a reach and lags the hydrograph by this time. Therefore, there is no change in the hydrograph shape with time-shift routing. The implicit procedure performs a storage routing through the reach, calculating the reach storage from the depth of flow. However, it does not allow for dynamic effects such as pressurisation and backwater. These two options were studied using the four design storms (Section 6.4) on the selected two catchments (Section 6.3). The results are given in Table 6.5. Figure 6.5 shows the effect of these methods on peak discharge of both catchments, since only the peak discharge changes with routing options. This figure gives the percentage difference of peak discharges for the time-shift method compared to the implicit method. This form of percentage difference was selected, since the base run considered the implicit method for pipe routing.

Table 6.5: Values for Different Pipe Routing Methods

Storm Event Number	Time of Entry Method	Altona Meadows Catchment		Therry Street Catchment	
		P (m^3/s)	TTP (min)	P (m^3/s)	TTP (min)
E1	Time-shift	0.710	18	4.930	18
E2		1.067	18	10.806	18
E3		1.247	18	12.495	16
E4		1.443	18	12.495	10
E1	Implicit	0.718	18	5.253	18
E2		1.068	18	11.210	18
E3		1.247	18	12.495	14
E4		1.443	18	12.495	10

P= Peak discharge, TTP= Time to peak discharge

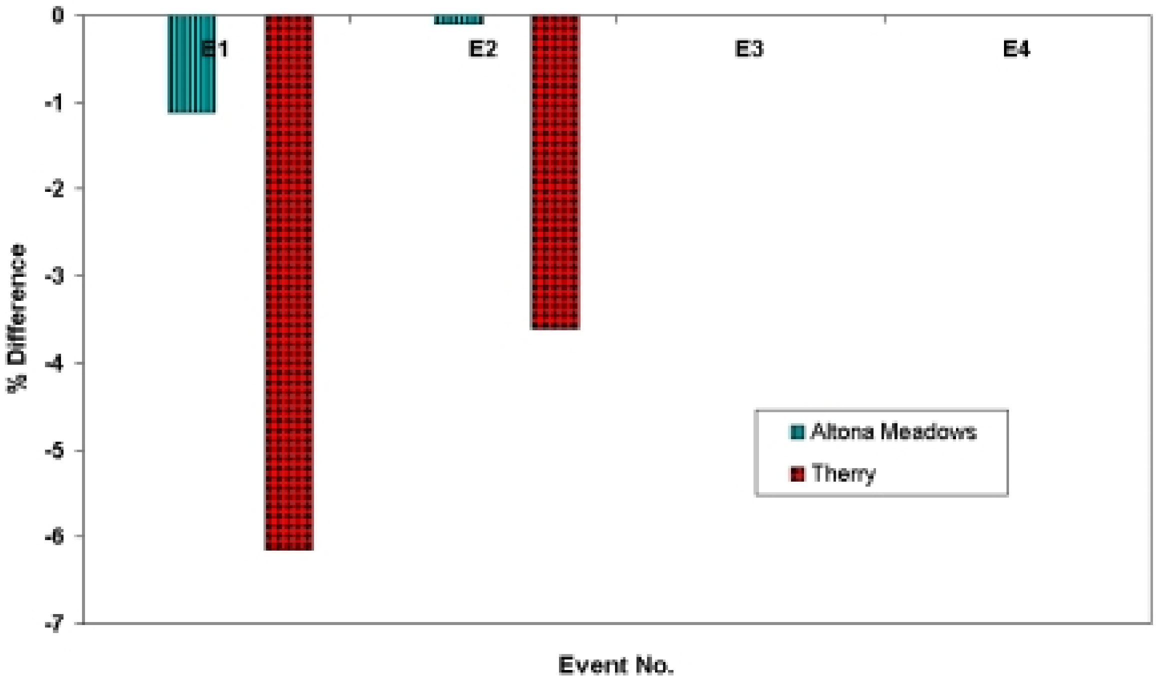


Figure 6.5: Effect of Pipe Routing Methods on Peak Discharge

The results of the study showed that there were no significant differences between the two methods in simulating the peak discharge and time to peak discharge for the Altona Meadows catchment. However, there was a significant difference in peak discharge from the two methods in the Therry Street catchment except the two largest storm events. The largest two events should produce the same peak discharge as discussed in Section 6.5.1. Time to peak discharge was almost the same from the two methods. The runoff volume from both methods should be the same, as discussed in Section 6.5.2.1.

Data requirements for the two methods are the same. The implicit method, which considers the storage effects, is recommended and was used in Chapter 7 for pipe routing, since it models the physical process more realistically than the time-shift method.

6.5.3 Pit Inlet Capacity Restrictions

The ILSAX model provides two options for modelling pit inlet capacities (Section 4.2.4). The first option considers no inlet restriction (i.e. infinite capacity). With this type of inlets, inflow of any magnitude can enter the pipe system through the pit inlet, provided the reach capacity of the downstream pipe is adequate. If the incoming flow is higher than the reach capacity, overflow occurs. In ILSAX, this overflow is conceptually stored at the upstream end of the reach and released back to the reach when capacity becomes available. The other option is the consideration of actual pit inlet capacities (i.e. finite capacity). In this case, if the incoming flow is higher than the inlet capacity, the excess flow travels to the next designated pit as bypass flow.

These two modelling options were studied using the four design storms (Section 6.4) on the selected two catchments (Section 6.3). The peak discharge and time to peak discharge at the outlet of the two catchments are given in Table 6.6. Figure 6.6 shows the effect of these methods on peak discharge for both catchments, since the peak discharge changes with pit inlet capacity restriction method. This figure gives the percentage difference of peak discharge for the infinite capacity method compared to the finite capacity method. This form of percentage difference was selected, since the base run considered the finite capacity method.

Table 6.6: Results of Two Methods for Pit Inlet Capacity Restrictions

Storm Event Number	Time of Entry Method	Altona Meadows Catchment		Therry Street Catchment	
		P (m^3/s)	TTP (min)	P (m^3/s)	TTP (min)
E1	No inlet restriction	0.585	18	5.343	18
E2		0.859	16	11.769	18
E3		1.188	18	12.495	14
E4		1.251	14	12.495	12
E1	Actual inlet capacity	0.718	18	5.253	18
E2		1.068	18	11.210	18
E3		1.247	18	12.495	14
E4		1.443	18	12.495	10

P= Peak discharge, TTP= Time to peak discharge

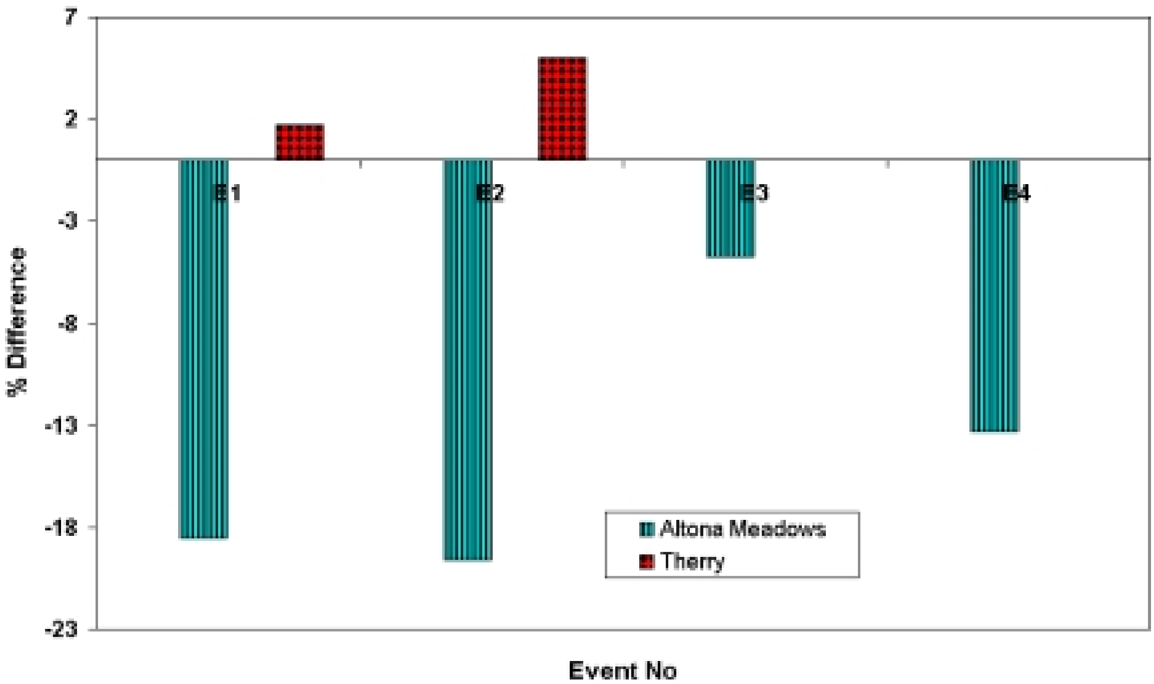


Figure 6.6: Effect of Pit Inlet Capacity on Peak Discharge

The results showed that the peak discharges were different with respect to inlet capacity option in two catchments. The Altona Meadows catchment gave lower peak discharges for all selected design storms for the infinite inlet capacity option. However, the Therry Street catchment gave higher or equal peak discharge for all selected design storms for the infinite inlet capacity option. There was no difference in the two largest storm events of the Therry Street catchment, since the outlet pipe was full, as explained in Section 6.5.1.

Therefore, the peak discharge depends on combine effects of catchment details such as pipe and the pit inlet capacity modelling method. Since the finite capacity method considers the actual pit inlet capacity, it is a more realistic representation of the physical drainage system. Therefore, this method is recommended and used in Chapter 7. However, if the modeller is interested only on total catchment runoff volume, then the infinite capacity option can be used, since the data requirements for this method are less.

6.6 SENSITIVITY OF ILSAX MODEL PARAMETERS

Generally, the purpose of a sensitivity analyses is to determine which input parameters apply the most influence on model results. Therefore, the parameter sensitivity analysis provides the information on:

- which parameters require more consideration in modelling, thereby reducing output uncertainty,
- which parameters are insignificant and should be given less consideration in modelling,
- which parameters are most highly correlated with the output, and
- what is the change to the output response, if the model parameters deviate significantly from the 'optimum' set.

In a model, there may be several sensitive parameters. Of these, some are more sensitive than the others. Therefore, it is important to know which parameters are the most sensitive among the sensitive parameters. They can be identified through parameter ranking which shows the amount of influence that each parameter has on the model output. Although there are several sensitivity techniques are available, each of which would result in a slightly different sensitivity ranking (Hamby, 1994). Since the main purpose of the sensitivity analysis of this chapter was to obtain a broader feeling of the sensitive parameters of ILSAX, a detailed sensitivity analysis was not conducted and therefore a simple sensitivity analysis method (i.e. one at a time sensitivity analysis) was used.

Four hydrological parameters (i.e. DS_i, DS_p, CN and AMC) and six routing parameters (i.e. N_p, N_r, CF, GUT, CAP3 and CAP4) were considered in the parameter ranking. Sag pits were not considered in the parameter sensitivity ranking, since they were not present in the study catchments. The output responses of runoff volume and peak discharge were considered in parameter ranking. Variation of parameter ranking with respect to storm magnitude was also considered. This sensitivity information was used for the ILSAX model calibration discussed in Chapter 7. Since the time to peak discharge did not change considerably based on 2 minutes simulation time step, it was not considered in the sensitivity analysis. The ranking of model parameters with respect to sensitivity of runoff volume and peak discharge will be valuable to the modeller in determining most sensitive parameters in a modelling exercise. For example, if the runoff volume is more important than the other hydrograph attributes in a particular modelling exercise (e.g. design of retarding basins and floodways), then the modeller can put more effort into the accurate determination of most sensitive parameters with respect to ranking of model parameters in terms of runoff volume. However, for design of small to medium hydraulic structures, the flood peak is more important and therefore the most sensitive parameters with respect to ranking based on peak discharge should be considered.

6.6.1 Ranking Index

In the past, different researchers have defined the model parameter sensitivity differently. McCuen (1972) defined it as the rate of change of the model output with respect to the change in the value of the parameter under consideration, while keeping the other parameters constant. Frankel and Hansen (1968) defined the parameter sensitivity in a general sense as the effect of parameter changes on the dynamics of a system (i.e. the time response, the state, the transfer function, or any other quantity characterising system dynamics).

A literature review was conducted to find a suitable technique to rank the model parameters. After searching through the literature, few formulae were found that could be applied for ranking of model parameters. However, they were not used for ranking model parameters in those studies.

One of the simple methods of determining parameter sensitivity is to calculate the output percentage difference from a range of output responses resulting from several values between minimum and maximum of one input parameter (Hoffman and Gardner, 1983; Bauer and Hamby, 1991). Hoffman and Gardner (1983) defined the sensitivity index (Si_a) in this case as:

$$Si_a = (D_{max} - D_{min}) / D_{max} \qquad (6.5)$$

where D_{min} and D_{max} represent the minimum and maximum output values respectively, resulting from varying the input parameter over its entire range. By computing SI_a for each input parameter, ranking can be obtained. Although, this method is very simple to apply, it is very subjective in obtaining the minimum and maximum value for a model parameter.

Maheshwari (1988) used the sensitivity index (SI_b) to study the sensitivity of parameters of a mathematical model used for border irrigation. SI_b is defined by Equation 6.6.

$$SI_b = \frac{\dfrac{100}{N}\displaystyle\sum_{i=1}^{N}\dfrac{(X_{ni} - X_{ci})}{X_{ci}}}{\Delta} \qquad (6.6)$$

where N is the number of points in an output,

X_{ci} is the value of output for the i[th] point due to base value of the input parameter,

X_{ni} is the value of output for the i[th] point due to new value of the input parameter, and

Δ is the absolute value of change in the input parameter, expressed as a percentage of its base value.

The sensitivity index (SI_b) represents the percentage change in the model output from that of the base run resulting from a one-percent change in the value of an input parameter from its base value. If several values are used for a parameter, then several SI_b values are computed from Equation 6.6, unlike in SI_a. Therefore, it is difficult to obtain a single ranking number for a parameter by applying this equation since it produces a number of SI_b values. Hence, this method is not suitable for parameter ranking.

In the PEST software (Watermark Numerical Computing, 1998), the parameter sensitivity (S) for a particular outcome is calculated as the ratio of the difference between the model outcomes due to current and base parameter set, to the difference between the values of current and base parameter sets. If n parameters are changed from their base values in a particular modelling exercise, then the parameter sensitivity is given by,

$$S = (O - O_b) / \{(P_1 - P_{1,b})^2 + (P_2 - P_{2,b})^2 + \ldots\ldots\ldots\ldots + (P_n - P_{n,b})^2\}^{1/2} \quad (6.7)$$

where O is the model output corresponding to the current parameter set,

O_b is the model output corresponding to the base parameter set,

P_n is the current value of the n^{th} parameter, and

$P_{n,b}$ is the base value of the n^{th} parameter.

Thus, if only a single parameter P differs from the base set, S then becomes:

$$S = (O - O_b) / (P - P_b) \quad (6.8)$$

S gives a dimensional quantity. McCuen (1972) also used S for sensitivity analysis. Since the ILSAX model parameters have different types of units (i.e. DS_p and DS_i in mm, while CN, AMC, N_p, N_r, CAP3 and CAP4 are non-dimensional), S is not suitable for this study to rank the model parameters.

Lei and Schilling (1994) and Lei (1996) carried out a parameter uncertainty propagation analysis in the runoff block of the HYSTEM-EXTRAN (Fuchs and Verworn, 1988) model. In their study, the parameter sensitivity was characterised by a sensitivity coefficient (SC), which was defined as the ratio of the coefficient of variance (CV) of a model output to the coefficient of variance of the model parameter itself. SC is a non-dimensional quantity. It can be derived from the model output values corresponding to different values of a parameter, keeping the other parameters constant. Although SC was not used for parameter ranking in their study, SC can be used for the present study, since SC is non-dimensional and also (to a certain extent) standardised. Therefore, it can be used to compare the parameters having different units. Also, SC produces a single value for a parameter combining the range of values of a parameter and corresponding outputs, unlike of SI_b of Maheswari (1988). SC is expressed in Equation 6.9.

$$SC = CV_o / CV_p \tag{6.9}$$

where subscripts o and p refer to the model output and the model parameter under consideration. Larger the number of points used in computing SC, the accuracy is higher. However, the number of points used to compute SC was not mentioned in Lei and Schilling (1994). In this study, model parameters were obtained on estimates and uniform distribution was considered for each parameter based on recommendations of 15 international urban hydrology experts. When computing CV in the Lei and Schilling (1994) study, the average value of parameters was taken as mean value.

If CV of output vary largely with CV of a particular input parameter, then parameter is a highly sensitive parameter giving high value for SC. If an input parameter is not highly sensitive, the output response will not change significantly with respect to different values of the parameter, producing a lower CV and lower SC of the output response.

6.6.2 Parameter Sensitivity Ranking

As stated in Section 4.3, the runoff volume depends only on the hydrological parameters, while the peak discharge depends on both hydrological and routing parameters. Therefore, the sensitivity of the hydrological parameters on both runoff volume and peak discharge was studied. Also, the sensitivity of routing parameters was considered on peak discharge.

Conceptually, the simplest method of sensitivity analysis is to repeatedly vary one parameter at a time while holding other parameters at fixed values (Gardner *et al.*, 1980; Breshears, 1987; Crick *et al.,* 1987). This method is commonly known as 'one-at-a-time' method and was used in this study. Therefore, for each parameter (except CAP3 and CAP4), several values between published minimum and maximum values were considered, as shown in Table 6.7. The CAP3 and CAP4 values between -20% to 20% of base run value were considered since the variation of CAP3 and CAP4 with different types of grade pits as given in O'Loughlin *et al.* (1992a) were within $\pm 20\%$. The CAP3 and CAP4 values were changed simultaneously to see the effect of both parameters since both parameters increase together when pit capacity increase. The first row of Table 6.7 contains the parameter values of the base run of sensitivity analysis.

For each model run, one parameter was changed while the other parameters were kept at their base values. Therefore, for one catchment and each design storm, 36 model runs were considered. Since two catchments and four design storms were considered in the study, the ILSAX model was run 288 (= 36 * 4 * 2) times. These results were used to rank the model parameters.

Table 6.7: Selected Parameter Values for the Sensitivity Analysis

DS_p (mm)	DS_i (mm)	CN	AMC	N_p	N_r	CF	GUT	CAP3	CAP4
4	1	2	2	0.012	0.2	0.2	10	0.345	0.738
0	1	2	2	0.012	0.2	0.2	10	0.345	0.738
2	1	2	2	0.012	0.2	0.2	10	0.345	0.738
6	1	2	2	0.012	0.2	0.2	10	0.345	0.738
8	1	2	2	0.012	0.2	0.2	10	0.345	0.738
10	1	2	2	0.012	0.2	0.2	10	0.345	0.738
4	0.0	2	2	0.012	0.2	0.2	10	0.345	0.738
4	0.5	2	2	0.012	0.2	0.2	10	0.345	0.738
4	1.5	2	2	0.012	0.2	0.2	10	0.345	0.738
4	2.0	2	2	0.012	0.2	0.2	10	0.345	0.738
4	1	1	2	0.012	0.2	0.2	10	0.345	0.738
4	1	3	2	0.012	0.2	0.2	10	0.345	0.738
4	1	4	2	0.012	0.2	0.2	10	0.345	0.738
4	1	2	1	0.012	0.2	0.2	10	0.345	0.738
4	1	2	3	0.012	0.2	0.2	10	0.345	0.738
4	1	2	4	0.012	0.2	0.2	10	0.345	0.738
4	1	2	2	0.011	0.2	0.2	10	0.345	0.738
4	1	2	2	0.013	0.2	0.2	10	0.345	0.738
4	1	2	2	0.014	0.2	0.2	10	0.345	0.738
4	1	2	2	0.015	0.2	0.2	10	0.345	0.738
4	1	2	2	0.016	0.2	0.2	10	0.345	0.738
4	1	2	2	0.012	0.1	0.2	10	0.345	0.738
4	1	2	2	0.012	0.3	0.2	10	0.345	0.738
4	1	2	2	0.012	0.4	0.2	10	0.345	0.738
4	1	2	2	0.012	0.2	0.0	10	0.345	0.738
4	1	2	2	0.012	0.2	0.4	10	0.345	0.738
4	1	2	2	0.012	0.2	0.6	10	0.345	0.738
4	1	2	2	0.012	0.2	0.8	10	0.345	0.738
4	1	2	2	0.012	0.2	0.2	4	0.345	0.738
4	1	2	2	0.012	0.2	0.2	6	0.345	0.738
4	1	2	2	0.012	0.2	0.2	8	0.345	0.738
4	1	2	2	0.012	0.2	0.2	12	0.345	0.738
4	1	2	2	0.012	0.2	0.2	10	0.276	0.590
4	1	2	2	0.012	0.2	0.2	10	0.310	0.664
4	1	2	2	0.012	0.2	0.2	10	0.380	0.812
4	1	2	2	0.012	0.2	0.2	10	0.414	0.886

For the sensitivity analysis, SENSAN module of PEST computer program (Watermark Numerical Computing, 1998) was used. SENSAN is model-independent and can communicate with a model through its own input and output files. SENSAN can conduct the sensitivity analysis automatically by adjusting model parameter inputs, running the model, reading the relevant outputs of interest, writing their values on a file and then commencing the whole cycle again. This process runs for each parameter and continues until the last set. SENSAN requires four types of input files namely the template files, the instruction files, the parameter variation file and the SENSAN control file. In this sensitivity analysis, two template files (i.e. for ILSAX rainfall and pipe files), one instruction file (i.e. for ILSAX output file), one parameter variation file and a SENSAN control file were prepared for each design storm. The details on the use of SENSAN for sensitivity analysis of model parameters are found in Watermark Numerical Computing (1998).

In this study, SC defined by Lei and Schilling (1994) and Lei (1996) was used to rank the model parameters based on their sensitivity to hydrograph attributes of runoff volume and peak discharge. Uniform distribution was assumed for ILSAX model parameters in this study, since there are no other information available in relation to probability distributions of these parameters. Standard theoretical equations were used to compute the mean and the standard deviation (thus CV) of input parameters using minimum and maximum values. CV of the output responses (i.e. runoff volume and peak discharge) was computed using several values of the input parameters within the range defined by their minimum and maximum values. The minimum and maximum values were obtained from O'Loughlin *et al.* (1992a) and O'Loughlin (1993). SC was computed for each model parameter using the results in 288 model runs as described above. SC was computed separately for each design storm.

6.6.3 Ranking of Model Parameters Based on Runoff Volume

As stated in Section 4.3, the model parameters of the pervious area depression storage (DS_p), impervious area depression storage (DS_i), soil curve number (CN) and antecedent moisture condition (AMC) are responsible for the runoff volume of the catchment. The other parameters (i.e. routing parameters) do not affect the runoff volume.

Figure 6.7 shows the sensitivity coefficient (SC) of runoff volume for each influencing model parameter considering the four design storms. SC was computed from Equation 6.9 considering 6 points for DS_p, 5 points for DS_i and 4 points each for CN and AMC. As can be seen from Figure 6.7, DS_p, CN and AMC are sensitive only for the two largest storm events, since pervious area runoff was not present for the smaller events. DS_i is sensitive for all four storm events.

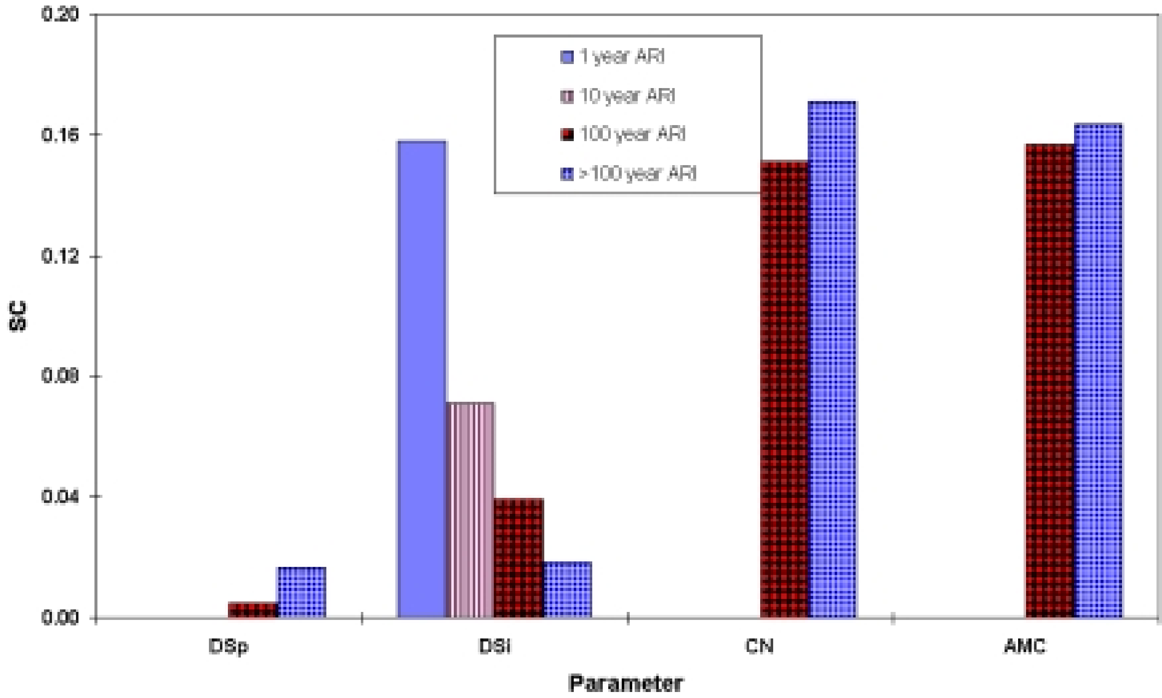
As the storm magnitude increases, SC of DS_i decreases. This is to be expected since variation of runoff depth due to different DS_i values at higher ARI storms is less compared to that of lower ARIs, since most runoff is originated from pervious areas. For largest storm events (i.e. 100 year ARI and > 100 year ARI), SC for parameters DS_p, CN and AMC increases slightly with the increase of storm magnitude, since the rate of pervious area runoff contribution increases with the storm magnitude. However, SC for DS_p is smaller than those of CN and AMC. These results are similar for both catchments.

In summary, the ranking of model parameters based on total runoff volume can be expressed in Equations 6.10 and 6.11 for 'small' (<= 10 year ARI) and 'large' (>= 100 year ARI) storm events respectively. These results are based on the ranking index SC (i.e. Equation 6.9), and based on study of two catchments and four design storms.

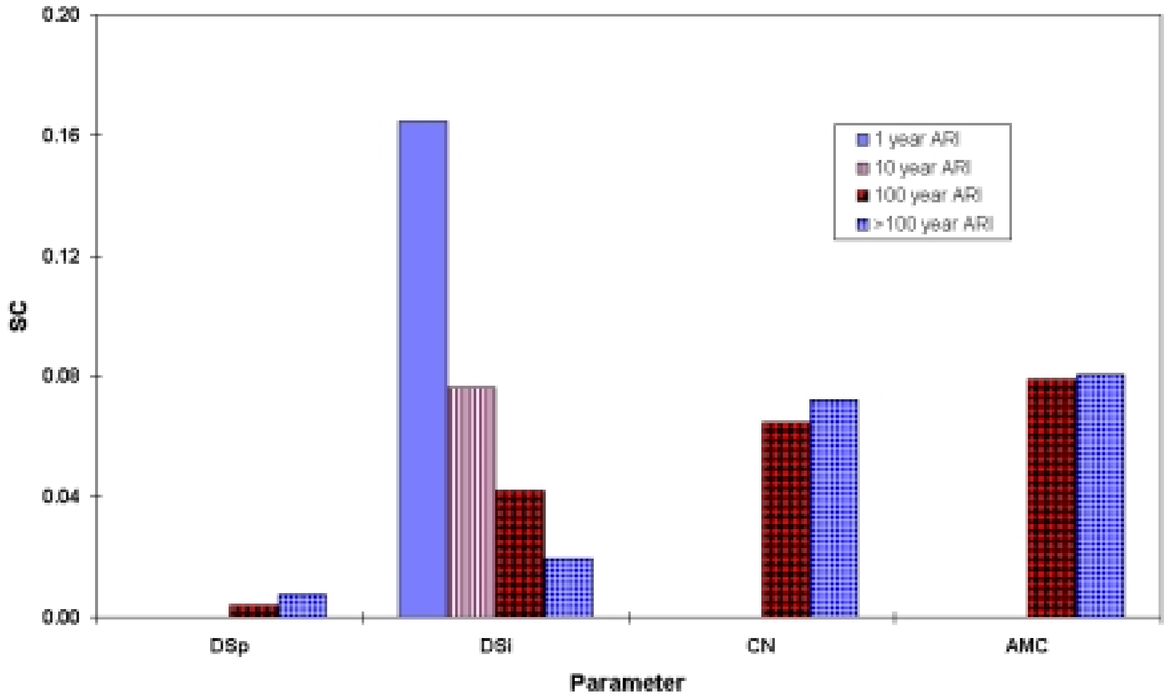
For 'small' storm events, only DS_i (6.10)

For 'large' storm events, $AMC = CN > DS_i > DS_p$ (6.11)

Therefore, DS_i is the only sensitive parameter in simulation of 'small' storm events for runoff volume and AMC and CN are for 'larger' events, irrespective to catchment size.



(a) Altona Meadows



(b) Therry Street

Figure 6.7: SC for Total Runoff Volume

6.6.4 Ranking of Model Parameters Based on Peak Discharge

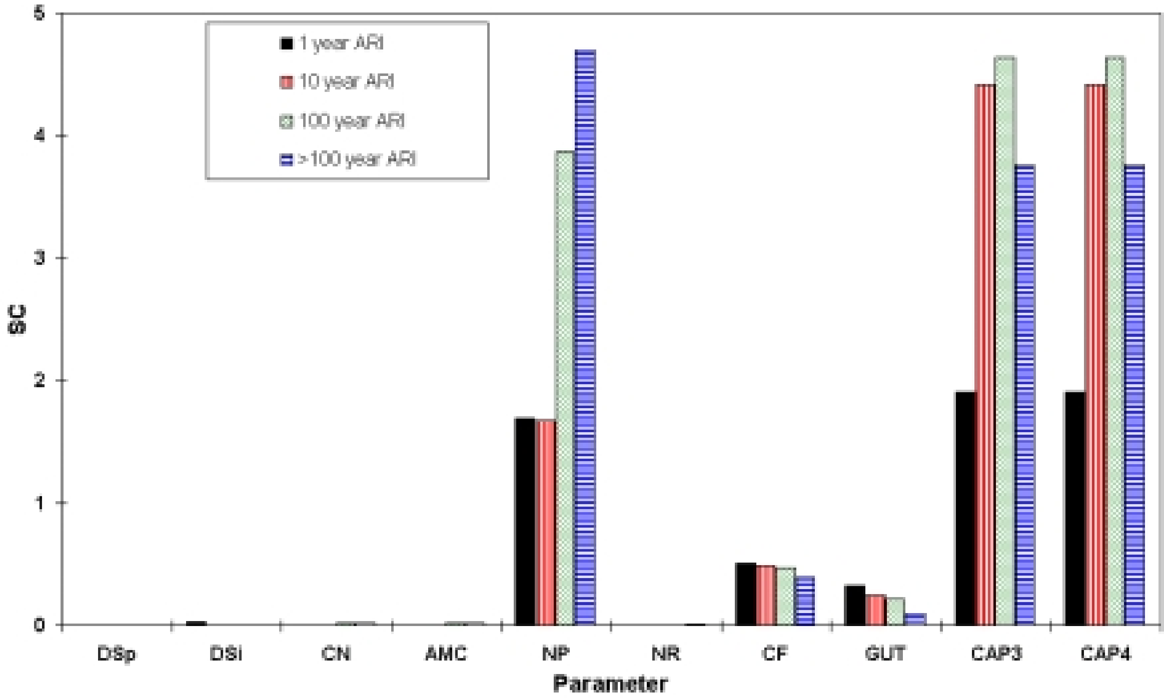
As stated in Section 4.3, both hydrological (i.e. DS_i, DS_p, CN and AMC) and routing parameters (i.e. CF, N_p, N_r, GUT, CAP3 and CAP4) affect the peak discharge of a catchment. The parameter sensitivity ranking based on the peak discharge of the pipe just downstream of the catchment outlet (ignoring the bypass flow for which hydrograph is not available from ILSAX) was analysed using four design storms of the two catchments (Section 6.3).

Figure 6.8 shows the sensitivity rankings for the two catchments. For both catchments, Figure 6.8 shows that the hydrological parameters are not important in terms of parameter ranking on peak discharge for four design storms. N_r was also not sensitive on peak discharge. N_p showed a high SC for both catchments. N_p, CAP3 and CAP4 were the most sensitive parameters for the four design storm events of the Altona Meadows catchment. CF and GUT were less sensitive for this catchment. N_p was the most sensitive in the Therry Street catchment. CF, GUT, CAP3 and CAP4 were less sensitive for the two 'small' storm events of the Therry Street catchment. The sensitivity could not be seen for the two largest events since the outlet pipe was full, giving the same discharge for most options. The detail rankings for both catchments are given in Equations 6.12 and 6.13.

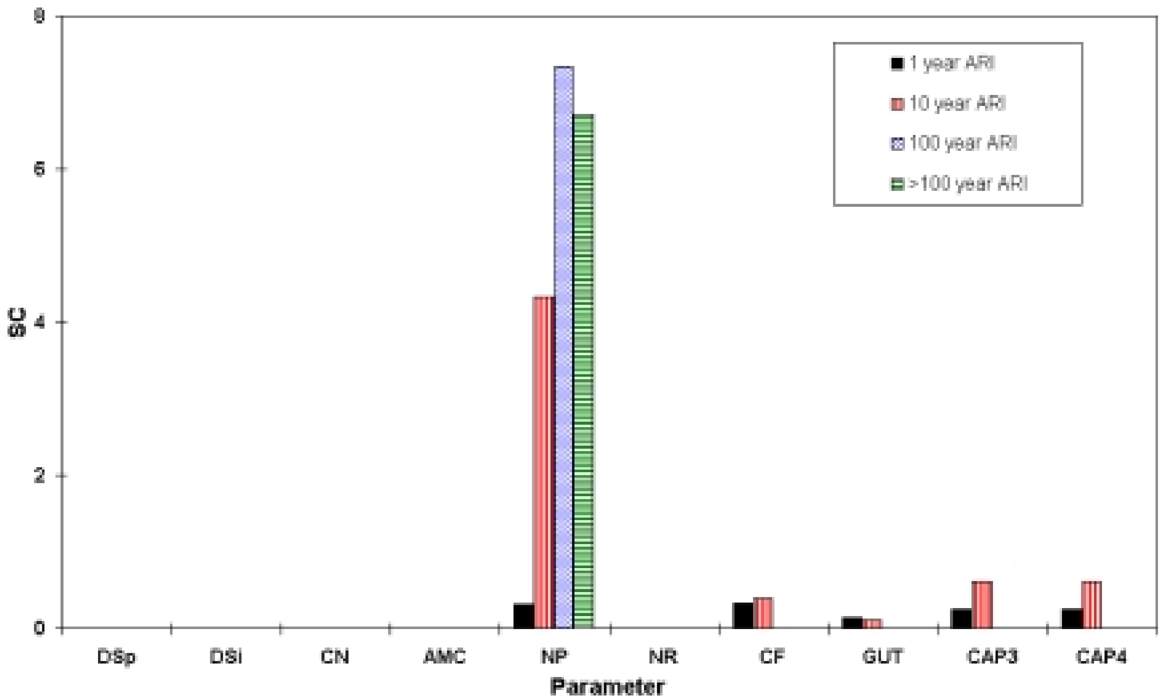
$$\text{CAP3} = \text{CAP4} > N_p > \text{CF} > \text{GUT for four events of the Altona Meadows catchment} \tag{6.12}$$

$$N_p > \text{CAP3} = \text{CAP4} > \text{CF} > \text{GUT for two 'small' storm events of the Therry Street catchment} \tag{6.13}$$

Therefore, it can said that the pipe roughness coefficient and the inlet capacity parameters are the most sensitive routing parameters for a catchment irrespective of the storm magnitude, based on limited experiments conducted on sensitivity analysis in this study.



(a) Altona Meadows



(b) Therry Street

Figure 6.8: SC for Peak Discharge of Pipe Flow

6.7 LEVEL OF CATCHMENT SUBDIVISION

The level of catchment subdivision is important in the analysis of urban drainage systems using any rainfall-runoff model. The typical questions asked by the modeller in this respect would be: does the model require every pipe and pit to be included, or can an acceptable result be achieved by considering only the major pipes and associated pits? However, the catchment subdivision in urban drainage models has received little attention in the literature.

Heeps and Mein (1973a,b) considered three categories of subdivisions in applying three computer models to an urban catchment in Australia. The subdivisions used were coarse (neglecting all lateral drains and considering each subcatchment to contribute directly to the inlets of the main drain), medium (considering the first-order lateral drains as well as the main drain) and fine (considering all drains). The results of the study were presented qualitatively and indicated that the degree of subdivision of a catchment had a marked influence on the simulated outflow hydrograph. However, they do not recommend the type of subdivision required for urban drainage modelling.

Zaghloul (1981) applied the SWMM (U.S. Environmental Protection Agency, 1992) model for several hypothetical and real urban catchments to illustrate the effect of subdivision on the accuracy of runoff simulation and to investigate the accepted level of subdivision. Fine and coarse subdivisions were used in the study. The definition for subdivision used in this study seems to be similar to that of Heeps and Mein (1973a,b). Zaghloul (1981) concluded in this study that for small catchments, conduit routing was insignificant and therefore coarse subdivision can be used. However, Zaghloul did not define the physical size of ‘small’ catchments. It was also emphasised that the flow simulation by coarse subdivision could be used for planning but not for design. For design purposes, detailed simulation using fine subdivision was recommended.

In the present study, the effect of subdivision was investigated using the three subdivision schemes used by Heeps and Mein (1973a,b). For fine subdivision, every pit in the drainage system was included. For medium subdivision, the pits of main and secondary (i.e. first-order) drains, which have flow-contributing subcatchments were considered. In addition, if the diameter and the slope of inlet and outlet pipes changed at pits, these pits were also considered. For coarse subdivision, only the pits of the main drainage line of the medium subdivision were considered. The same design storms used in Section 6.4 were used for the subdivision analysis. The base run in Section 6.5 was considered, which included fine subdivision for comparison purposes. The actual capacities of the pits were used for all subdivisions. Although this may not be realistic for medium and coarse subdivisions, there is no information to compute an equivalent pit capacity for medium and coarse subdivisions, at this stage.

The peak discharges of the outlet pipe are shown in Figure 6.9 for the Altona Meadows and Therry Street catchments for different subdivisions analysed. The results showed that the peak discharge was the highest for fine subdivision, while it was the lowest for coarse subdivision for both catchments. In the Therry Street catchment, the peak discharge at the outlet was the same for two largest storm events, since the outlet pipe was full for these two storm events. The absolute and percentage difference of peak discharges for the three subdivisions are given in Table 6.8. The percentage difference was computed with respect to fine subdivision, since fine subdivision was closer to the reality.

The time to peak discharge of each catchment was almost the same for all three subdivisions. The total runoff volume (pipe flow and bypass flow) for all subdivisions should be the same and it was seen from the results. As seen from Table 6.8, the percentage difference of peak discharge of the coarse subdivision is always higher (or at least equal) to that of the medium subdivision. Therefore, the coarse subdivision gives more errors in peak discharge compared to the fine and medium subdivisions.

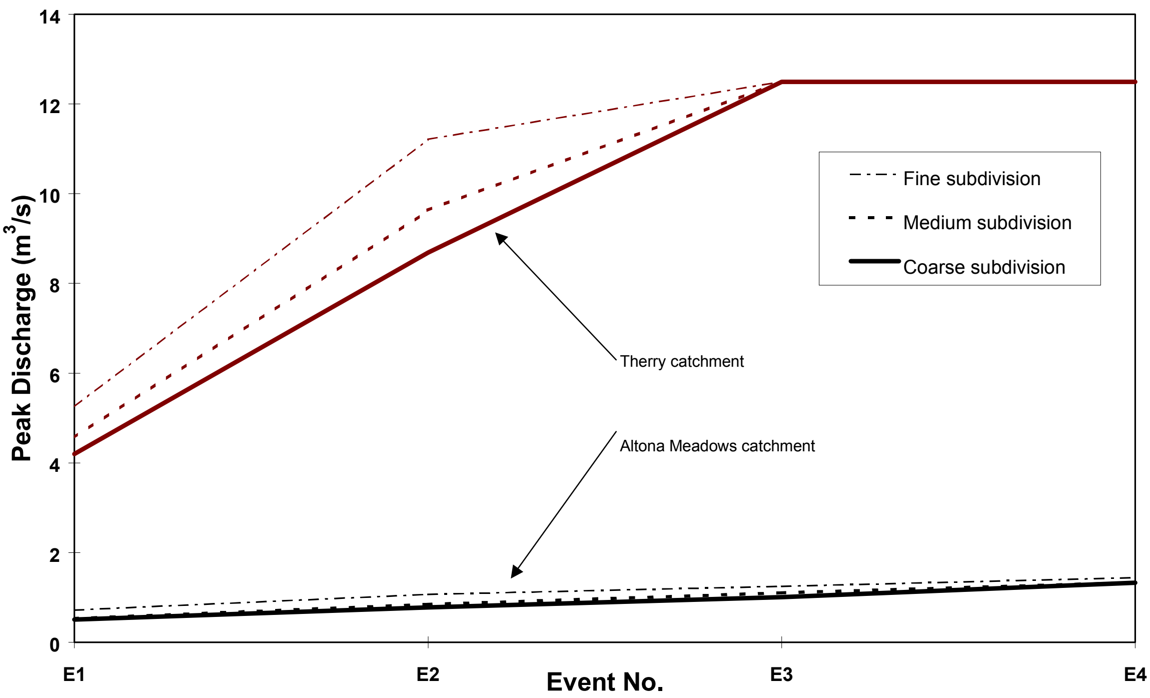


Figure 6.9: Effect of Catchment Subdivision on Peak Discharge

Table 6.8: Peak Discharges for Three Subdivisions

Catchment	Event	Peak discharge (m^3/s)					
		Fine subdivision		Medium subdivision		Coarse subdivision	
		Absolute	% difference	Absolute	% difference	Absolute	% difference
Altona Meadows	E1	0.718	N/A	0.531	26	0.505	30
	E2	1.068	N/A	0.845	21	0.780	27
	E3	1.247	N/A	1.102	12	1.010	19
	E4	1.443	N/A	1.334	8	1.326	8
Therry Street	E1	5.253	N/A	4.561	13	4.196	20
	E2	11.210	N/A	9.628	14	8.689	22
	E3	12.495	N/A	12.495	0	12.495	0
	E4	12.495	N/A	12.495	0	12.495	0

N/A Not applicable

The data requirements are less for coarse subdivision. Therefore, if only the runoff volume is important in an analysis, then the coarse subdivision can be used. Otherwise, the fine subdivision is recommended for the simulation of accurate peak discharge.

At least three factors are responsible for the differences in results due to the effect of subdivision. They are the overland and bypass travel time, the storage effect of the conveyance system (which affects pipe routing) and the capacity of modelled pits. For coarse subdivision, the bypass and overland flow times are the largest, while pipe routing is the minimum. For fine subdivision, it is the reverse. This is explained below by considering a hypothetical drainage network shown in Figure 6.10 using fine and coarse subdivisions.

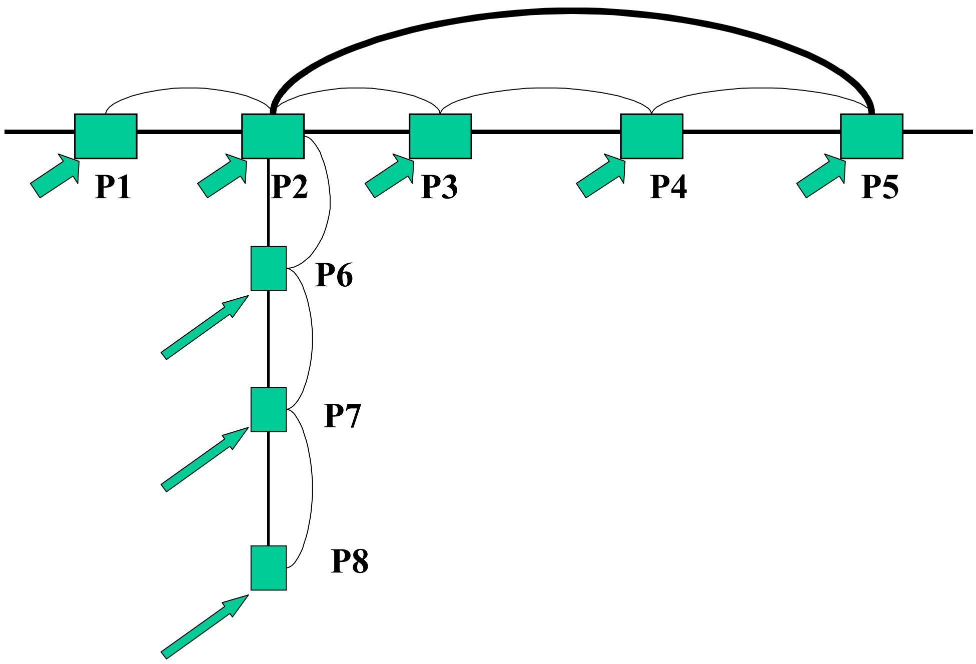


Figure 6.10: Flow Path for Different Subdivisions

For fine subdivision, all pits are modelled. Assume that only the pits P2 and P5 are modelled for the coarse subdivision. The overland flow paths for coarse subdivision at pits P2 and P5 are higher than those corresponding to the fine subdivision. Additionally, if flow at pit P2 is higher than its pit capacity, excess flow leaves the pit as bypass flow up to pit P5 in coarse subdivision compared to in fine subdivision.

The increase in overland bypass flow length in coarse subdivision increases the travel time compared to the fine subdivision and the base length of the hydrograph. This will give a lower peak discharge since the runoff volume is the same for both cases. However, the pipe routing (i.e. storage effect in pipe system) in coarse subdivision is less compared to the fine subdivision (e.g. in fine subdivision subcatchment flow at Pits 3 and 4 are pipe-routed to Pit 5, whereas these flows are treated as overland flow in coarse subdivision), which tends to give a higher peak discharge for coarse subdivision. This effect may not be as large as that for overland and bypass flow. There may also be a possibility that the effects of the overland and bypass flow time and pipe routing might compensate with each other in estimating the peak discharge. For fine subdivision, the actual capacity of each pit is considered, while the actual capacity of the pit at the location of the modelled pit is considered for medium and coarse subdivisions. In the medium and coarse subdivisions, some pits are not modelled and this may reduce the pipe flow in underground pipe system, which also tends to reduce the peak discharge for medium and coarse subdivisions.

6.8 SUMMARY AND CONCLUSIONS

The use of ILSAX modelling options, the sensitivity of ILSAX parameters on hydrograph attributes (i.e. runoff volume, peak discharge and time to peak discharge) and the effect of catchment subdivision were studied and discussed in this chapter. For all these studies, one typical 'small' (i.e. Altona Meadows) and one 'large' (i.e. Therry Street) catchment were considered with four design storms. The design storms covered the magnitudes of storms from ARI of 1 year to greater than 100 years. For sensitivity analysis, the SENSAN of PEST (Watermark Numerical Computing, 1998) optimisation module was used. Based on these results, the following conclusions were drawn.

Analysis of different modelling options of ILSAX revealed that the rainfall loss subtraction from supply rate, overland flow routing using the ARR87 procedure, pipe routing by the implicit scheme and consideration of actual pit capacities were the most preferable options in urban drainage studies. This recommendation was based on the accuracy of modelling of relevant processes and the accuracy of model output response (i.e. hydrograph attributes of runoff volume, peak discharge and time to peak discharge). The accuracy of modelling of relevant processes was conducted by equations and the procedures and for modelling of these processes. The accuracy of model output response was based on limited experiments conducted in this study using two catchments and four design storms.

The model parameter sensitivity on runoff volume and peak discharge was analysed in detail, again based on these two catchments and four design storms. Since time to peak discharge was almost the same with respect to model parameters, it was not considered. As far as sensitivity of hydrological parameters on runoff volume was concerned, DS_i was the only parameter sensitive to runoff volume for 'small' storm events. All hydrological parameters, AMC, CN, DS_i and DS_p were sensitive (in that order of sensitivity) to runoff volume for 'large' storm events. However, the sensitivity of DS_i and DS_p is less for larger events compared to CN and AMC. Sensitivity of DS_i decreased with increase of storm magnitude for both catchments. Sensitivity of AMC, CN and DS_p increased with increase of storm event magnitude of larger events for both catchments. When storm magnitude increases, the sensitivity of pervious parameters does not change after exceeds the certain storm magnitude. The runoff volume does not depend on routing parameters.

Only the on-grade pits were considered in the sensitivity analysis dealing with routing parameters, since sag pits were not present in those two study catchments. CAP3, CAP4, N_p, CF and GUT were the sensitive parameters for peak discharge in the order of sensitivity for all design storm events of the Altona Meadows catchment. The sensitivity of N_p, CAP3, CAP4, CF and GUT were the sensitive parameters in the order of sensitivity for 'small' storm events of the Therry Street catchment. The sensitivity of these parameters could not be analysed for the larger storm events since the outlet pipe of the Therry Street catchment was full for two larger events.

The results of this study also suggest that the catchment subdivision affects the peak discharge. This is because of the dependence of peak discharge on the overland and bypass flow behaviour, the storage effects of the drainage system and the pit capacity, which are modelled differently for different levels of subdivision. The main difficulty in medium and coarse subdivision is to estimate the pit capacity of the modelled pits through an equivalent pit capacity to account for the effects of the other pits that were not modelled. Previous studies on urban drainage analysis have not considered this effect and therefore further research is required to compute the equivalent pit capacity. This aspect is considered to be beyond the scope of this study. As general criteria, if only the runoff volume is important in an analysis, then the coarse subdivision can be used. Otherwise, the fine subdivision is recommended.

CHAPTER 7

ESTIMATION OF ILSAX MODEL PARAMETERS FOR STUDY CATCHMENTS

7.1 INTRODUCTION

To use mathematical models for design and analysis of urban drainage systems, it is necessary to estimate the model parameters relevant to these systems. The model parameters for gauged drainage systems are generally estimated from calibration of the models. However, this is not possible for ungauged systems due to the absence of rainfall and runoff data. If regional equations (correlating model parameters to the drainage system and other details) are available, they can be used to estimate the model parameters for ungauged drainage systems. To develop such regional equations, it is also necessary to estimate the model parameters for gauged catchments through calibration. Calibration can be performed by visual comparison of modelled and observed hydrographs, or through parameter optimisation.

As stated earlier, the ILSAX model was used in this study. A parameter optimisation method was used to calibrate the model parameters using data on observed rainfall-runoff events of the study catchments. Then, these model parameters were verified using rainfall-runoff events, which were not used in the calibration.

First, the ILSAX model parameters are briefly discussed in this chapter, followed by a description of the selection of rainfall/runoff events for calibration. The optimisation process used for calibration of model parameters, known as the two-stage inner/outer optimisation procedure is described then. This is followed by the results of the model calibration and verification and a discussion of the results. Finally, the conclusions drawn from the calibration study are presented.

Twenty two catchments from the Melbourne metropolitan area and one catchment from Canberra were used in model parameter estimation. Because of the volume of work involved in the analysis of all Melbourne catchments, the results of only one catchment from the Melbourne metropolitan area are shown in this chapter, while those of the remaining catchments are given in Appendices C-F. However, the results of the Giralang catchment in Canberra is given in this chapter, since it is the only catchment that produced pervious area runoff with respect to calibration and verification storm events.

7.2 CATCHMENT RAINFALL/RUNOFF DATA AND ANALYSIS

7.2.1 Study Catchments

Twenty two urban drainage catchments from five city councils located within a 30 km radius from Melbourne Central Business District (MCBD) were selected for the study for estimation of ILSAX model parameters. Of the 22 catchments, 11 were major catchments, while the remaining 11 were subcatchments of these major catchments. In addition, one catchment from Canberra (Giralang catchment) was selected for the demonstration of the calibration procedure for 'large' storm events, since the observed events in Melbourne catchments were not large enough to produce runoff from pervious areas. Details of these catchments are given in Section 5.2.

7.2.2 Rainfall/Runoff Data and Event Selection

As discussed in Sections 5.4.1 and 5.4.2, the MCBD study catchments were continuously monitored for rainfall/runoff during 1996-1999. The hyetograph and hydrograph were plotted for each storm event, to investigate whether they are accurate enough for use in model calibration. Clearly defined significant events were selected as the significant events, which show reasonably, correct hydrograph response to rainfall hyetograph, and used in further analysis, as described in Section 7.2.3.

A similar data analysis procedure was followed for the Giralang catchment (Dayaratne, 1996), except selection of events. Other data analysis in relation to this catchment is described in Section 7.5.4.

7.2.3 Use of Rainfall and Runoff Depth Plots of Significant Storm Events

The runoff depth versus rainfall depth plots (RR plots) of an urban drainage catchment can be used to determine the accuracy of rainfall-runoff data, to separate 'small' and 'large' storm events, and to estimate the directly connected impervious area percentage (DCIA) and its depression storage (DS_i). In these plots, the runoff depths are computed as the ratio of runoff volume at the catchment outlet to the total area of the catchment. These plots have been used by Kidd (1978a,b), Laurenson *et al.* (1985), Bufill and Boyd (1992), Boyd *et al.* (1993), Dayaratne (1996) and Maheepala (1999) in their studies to determine DCIA and DS_i. The theory of RR plots is briefly discussed below, followed by the use of these plots for data checking, separation of storm events for the calibration of 'small' and 'large' events, and estimation of catchment parameters.

7.2.3.1 Theory of rainfall-runoff depth plots

The catchment area of an urban catchment can be divided into three significant catchment surfaces such as directly connected impervious area (A_i), supplementary area (A_s) and pervious area (A_p). These surfaces are explained in Section 4.2. Even for 'small' rainfall depths, the directly connected impervious area responds immediately after fitting its depression storage. However, there could be cases where there are directly connected impervious areas with no depression storage, thus responding almost immediately as the rainfall commences. As rainfall depth increases, then both supplementary area and pervious area respond again after their respective depression storages. However, it is difficult to say which surface (from supplementary or pervious surfaces) responds first, since it depends on the location of these surfaces with respect to the catchment outlet. Bufill and Boyd (1992) and Boyd *et al.* (1993) conceptualised the RR plots, as shown in Figure 7.1.

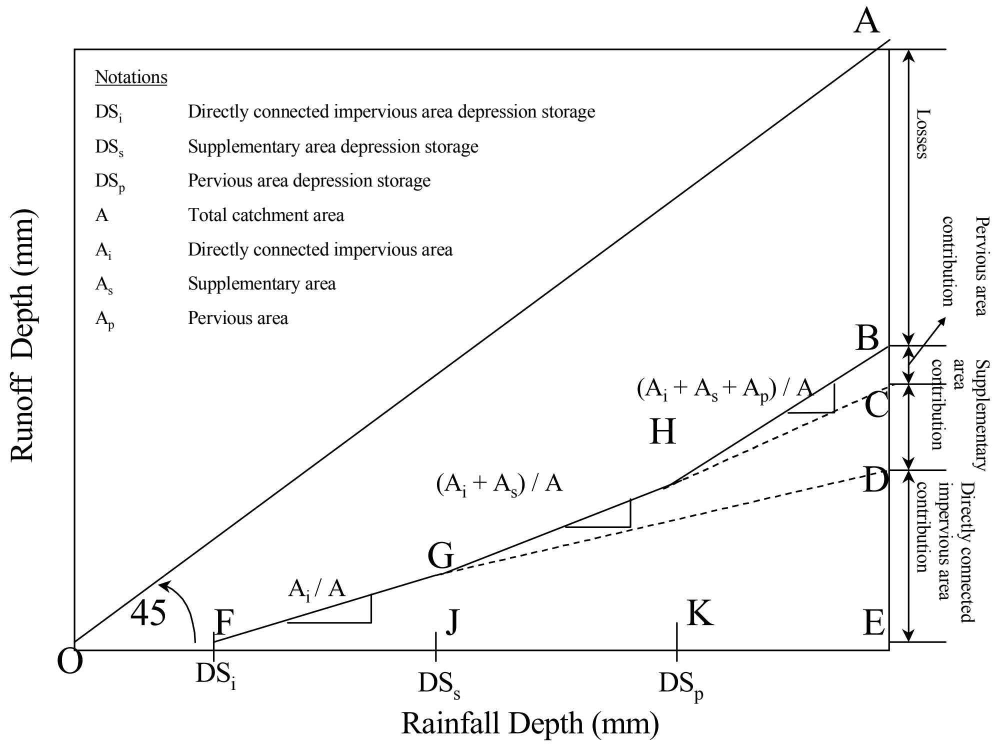


Figure 7.1: Rainfall-Runoff Depth Relationship from Different Catchment Surfaces

In Figure 7.1, it is assumed that the directly connected impervious area responds first, followed by the supplementary area and finally by the pervious area. However, as explained earlier, this is not the general case. For this reason, Kidd (1978a,b), Laurenson *et al.* (1985), Bufill and Boyd (1992), Boyd *et al.* (1993), Dayaratne (1996) and Maheepala (1999) used RR plots only to estimate parameters related to directly connected impervious areas.

In Figure 7.1, the segment FG represents runoff contributing from the directly connected impervious area and the slope of FG ($=A_i/A$) gives the directly connected impervious area percentage (DCIA). The depression storage (DS_i) of directly connected impervious area is given by OF. The segments GH represent runoff contribution from directly connected impervious and supplementary areas and the segment HB represents runoff from all three surfaces. The gradients of GH and HB give $[(A_i+A_s)/A]$ and $[(A_i+A_s+A_p)/A]$ respectively. Theoretically, GH and HB should be curvilinear because of the non-linearity of runoff response due to non-linear soil infiltration. However, in Figure 7.1, they are approximated as straight lines. OJ and OK give the depression storage of supplementary and pervious areas respectively.

Based on the above discussion, DCIA and DS_i can be estimated as a lumped catchment parameter with reasonable accuracy provided that the rainfall-runoff data are measured accurately and the catchment area estimated correctly.

7.2.3.2 Accuracy of rainfall-runoff events

For an event, the runoff depth should be always less than the corresponding rainfall depth. Due to the errors in measurements, there may be instances where the runoff depth is greater than the rainfall depth for some events. If runoff depth is plotted against rainfall depth, these events can be seen above the 45^0 line that passes through the origin. Therefore, all available significant storm events were plotted on the RR plot and checked whether there were events above the 45^0 line. This was done for all study catchments. If there were events where the runoff depths were greater than the rainfall depths, those events were removed from the data set. A sample RR plot for catchment BA2A is shown in Figure 7.2. In this plot, events indicated by 'triangle' symbol were removed from the data.

7.2.3.3 Estimation of directly connected impervious area parameters from RR plots

After removing erroneous data (as in Section 7.3.3.2), the RR plots were constructed again. These RR plots were used to estimate DCIA and DS_i of each catchment. This estimation was based on the theory presented in Section 7.2.3.1. A sample RR plot is shown in Figure 7.3 for the catchment BA2A. This plot gives DCIA and DS_i values 35% and 0.38 mm (= 0.13/0.35) respectively with a correlation coefficient of 0.92. From this plot, five 'small' storm events and another five 'large' storm events were selected for model calibration using hydrograph modelling. However, the ILSAX hydrograph modelling of these five 'large' storm events showed that they did not produce pervious area runoff. Therefore, of these ten events, eight events were selected for calibration and verification (5 for calibration and 3 for verification). These selected events are shown in Figure 7.3 as C1 to C5 for calibration events and V1 to V3 for verification events.

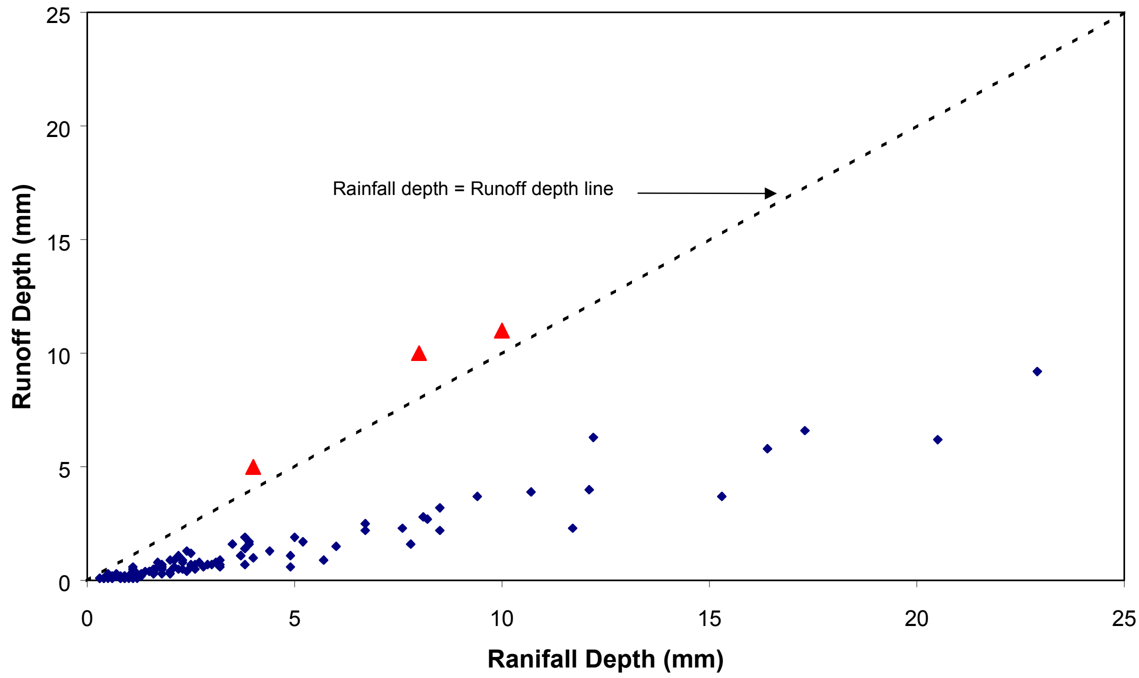


Figure 7.2: Sample RR Plot used for Data Checking (Catchment BA2A)

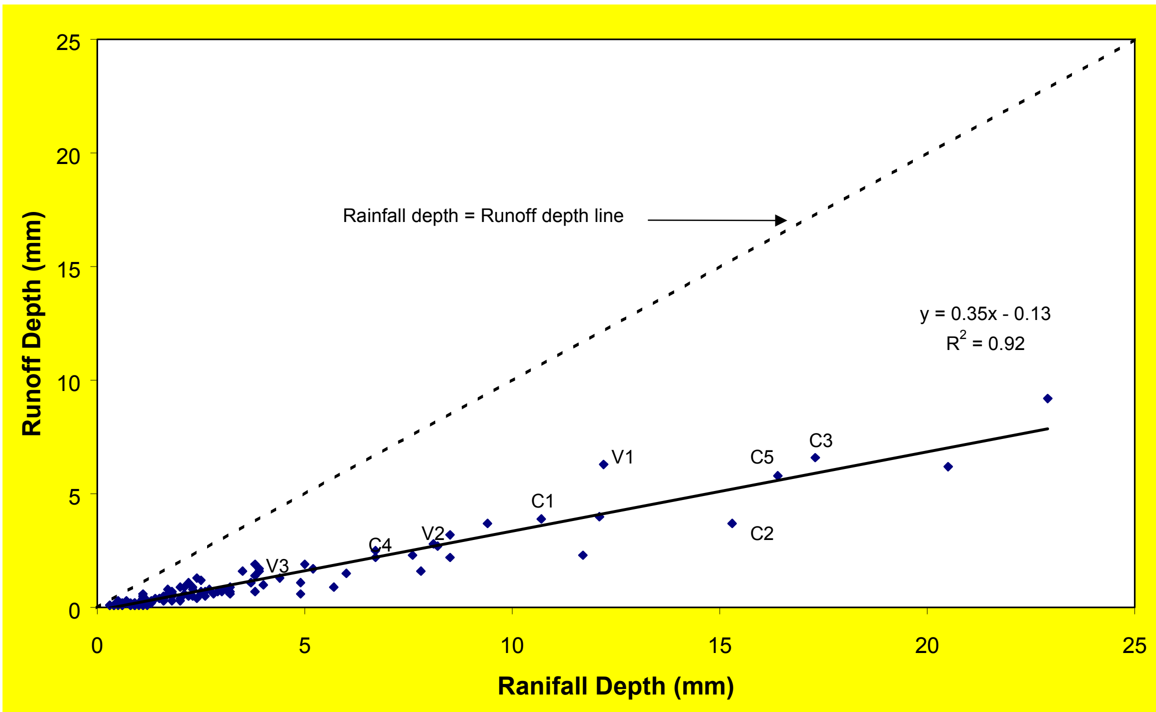


Figure 7.3: RR Plot for Catchment BA2A after Removing Erroneous Data

The RR plots were obtained for all study catchments and Giralang catchment, and DCIA and DS_i values for these catchments were estimated from these plots. DCIA and DS_i

obtained from the RR plots and correlation coefficient (R^2) are given in Table 7.1. The RR plots for the study catchments are given in Appendix C. Except for very few catchments, the RR plots showed very good correlation without significant scatter. Ninety one percent of the catchments showed greater than R^2 of 0.8.

Table 7.1: Directly Connected Impervious Area Parameters from RR Plots

Catchment Code	DCIA (%)	DS_i (mm)	R^2
BA2	47	0.57	0.83
BA2A	35	0.37	0.92
BA3	42	0.69	0.93
BA3A	35	0.69	0.90
BA3B	32	0.59	0.89
BO1A	50	0.14	0.72
BO2A	40	0.50	0.84
BR1	53	0.19	0.96
BR1A	58	0.29	0.98
BR2	52	0.31	0.95
BR2A	54	0.22	0.96
BR3	42	0.50	0.98
H2	77	0.90	0.95
H2A	67	1.03	0.96
K1	34	0.41	0.79
K1A	44	0.34	0.83
K1B	49	1.29	0.87
K2	35	1.17	0.87
K2A	26	1.77	0.92
K3	37	0.08	0.83
K3A	52	0.38	0.96
K3B	28	0.29	0.80
GI	19	0.26	0.93

The parameter estimates obtained from the RR plots were checked against the values obtained for the total impervious area of catchments (which includes the supplementary area) from areal photographs. This comparison was done only for 40% of the catchments, since areal photographs were not available for the other catchments. The DCIA values obtained from RR plots and estimated from areal photograph (for the study catchments for which areal photographs were available) are given in Table 7.2. The DCIA values estimated from areal photographs were always higher than the values obtained from RR plots, with the exception of one catchment (i.e. K1). This is to be expected, since the total impervious area includes both directly connected impervious area and supplementary area. The difference between two methods was within 10% limit except one catchment (i.e. K3B). The catchment K3B gave 17% higher value from areal photograph compared to value from the RR plot. However, estimating the total impervious area from areal photograph is a very approximate method and should be used only to check total impervious area obtained by other means.

Table 7.2: DCIA from RR Plots and Areal Photographs

Catchment Code	DCIA (%)	
	From RR plots	From areal photographs
H2	77	86
H2A	67	70
K1	34	40
K1A	44	45
K1B	49	51
K2	35	40
K2A	26	35
K3	37	40
K3A	52	54
K3B	28	45

7.2.4 Rainfall and Runoff Events for Calibration and Verification

Several storm events were selected from the RR plot described in Section 7.2.3.3 for calibration and verification of model parameters using hydrograph modelling. Although the procedure described in Section 7.2.3.3 can be considered as a calibration procedure to estimate the directly connected impervious area parameters (i.e. DCIA and DS_i), it considers only the runoff volume and no consideration was given to other attributes of the hydrographs such as peak discharge and time to peak discharge, which are equally important in urban drainage design and analysis. Therefore, the hydrograph modelling using selected storm events can be considered as another method to estimate model parameters of the study catchments. Since this method considers all attributes of hydrographs (i.e. runoff volume, peak discharge, time to peak discharge and shape), this can be considered to be a better method than the RR plots. The details of the selected eight events (i.e. five for calibration and three for verification) of catchment BA2A are given in

Table 7.3. The same information of events selected from other catchments for calibration and verification is given in Appendix D.

The total rainfall duration, the maximum intensity and the total rainfall depth of the storm event were obtained from the event hyetograph. The stormwater runoff volume and the maximum discharge were obtained from the event hydrograph.

The average recurrence interval (ARI) of each event was determined considering storm bursts within an event. All bursts greater than or equal to 6 minutes were considered for each selected storm event. The average intensity for these bursts was computed and plotted on the IFD curve derived for each catchment to assign ARI for the event. The event may have several such bursts which are plotted on the IFD curve. The burst which gives the highest ARI is considered as the initial bursts and the corresponding ARI is considered as the event ARI in Table 7.3. As indicated in Table 7.3, the ARIs are less than 1 year for seven events and two years for one event in catchment BA2A.

7.3 ADDITIONAL CONSIDERATIONS PRIOR TO MODEL CALIBRATION USING HYDROGRAPH MODELLING

The results of the model calibration are affected by many factors other than rainfall/runoff and catchment data. These factors need to be considered before the model calibration of study catchments. They are discussed below.

Table 7.3: Summary of Statistics of Storm Events Selected for Modelling of Catchment BA2A

Event Properties	**Calibration Events**					**Verification Events**		
Event number	C1	C2	C3	C4	C5	V1	V2	V3
Date of occurrence	14.09.97	10.11.97	16.02.98	20.04.98	19.05.98	20.05.98	25.05.98	06.06.98
Total rainfall duration (min)	426	124	370	274	286	332	344	200
Total rainfall depth (mm)	10.6	15.3	17.2	6.7	16.4	11.8	4.9	3.8
Maximum 2 min. intensity (mm/h)	18	126	12	12	42	12	4	12
ARI of storm event (years)	<1	2	<1	<1	<1	<1	<1	<1
Average intensity of most severe burst (mm/h)	10.0	70.0	10.0	8.2	36.0	10.0	3.3	12.0
Stormwater runoff volume (m^3)	543	509	917	346	802	843	268	267
Maximum discharge (m^3/s)	0.104	0.528	0.099	0.080	0.373	0.119	0.043	0.113

7.3.1 ILSAX Modelling Options

Different modelling options are available in ILSAX for subtraction of rainfall losses, routing of overland and pipe flow, and modelling of pit inlets. These options were discussed in Section 4.2 and investigated in detail in Section 6.5 using 'small' and 'large' design storms considering two urban catchments (i.e. a typical 'small' and a typical 'large'). Analysis of different modelling options of ILSAX revealed that the rainfall loss subtraction from supply rate, overland flow routing using the ARR87 method, pipe routing by the implicit scheme and consideration of actual pit capacities were the most preferable options in modelling urban drainage systems using ILSAX. These modelling options except for the pit inlet capacity option were used in this chapter. Since the collection of pit information and defining suitable pit capacity parameters for each and every pit of a catchment are extremely difficult with the available information, the infinite capacity option was reassessed for the events selected for the calibration. This is described in Section 7.3.2.

7.3.2 Pit Inlet Capacity

Inlet pit capacity separates incoming runoff to a pit into pipe flow and bypass flow. If pit capacity is not sufficient to receive the incoming inflow fully, then part of the flow goes as bypass flow and the remainder goes as pipe flow. On the other hand, if pit capacity is sufficient to capture the incoming inflows fully, the total incoming flow at the pit goes as pipe flow. Therefore, the bypass flow is a function of the pit inlet capacity. As stated in Section 4.2.4, ILSAX has two options to model pit capacity, namely infinite capacity and finite capacity.

Bypass flow from catchment H2 was analysed for these two inlet capacity options considering the selected events of each catchment for calibration. First, the inflow hydrograph at each pit was computed using the ILSAX model for a considered event. Then peak discharge of inflow hydrograph at a pit was compared with the computed pit capacity of the respective pit. It was observed that peak discharges for inflow hydrographs for all selected events for calibration was less than the respective computed pit capacities. Therefore, either pit capacity option (i.e. finite or infinite capacity) gives the same simulation results for the selected events for calibration.

These results were supported by the comments from city/shire council engineers (R. Silva, Buloke Shire, 1999 and M. Ishra, Greater Shepparton City Council, 2000). According to these discussions, the inlet pits in Victoria are designed considering a 5 year ARI (or more) design storm. Therefore, any storm event less than 5 year ARI can in general be captured at the pit completely if there is no blockage at the pit. As can be seen in Table 7.3 and tables in Appendix D, there are no rainfall/runoff events (which are used for calibration and verification) which are greater than those corresponding to 5 year ARI. This means that no bypass flow occurs for the events selected for calibration and therefore infinite capacity modelling option can be used for both calibration and verification.

7.3.3 Property Time

The property time is the time for roof runoff of a property to reach the road gutter system. The property time is included in the ILSAX model in calculating the pit entry time. This pit entry time sets the base length of the time-area diagram of the impervious area at each pit. If a higher property time is assumed, the time of entry of pit inflow hydrograph increases. As a result, the base length of inlet hydrograph increases causing a reduction in peak discharge.

The effect of property time was studied in this section using event C2 (Figure 7.3 and Table 7.3) of catchment BA2A. Two property times, 5 min and 2 min, were considered. The ARR87 and ILSAX manual recommend 5 min as the property time, while 2 min has been built into ILLUDAS-SA as the property time. Stephens and Kuczera (1999) also suggested 2 min as a property time in residential blocks. DCIA was kept constant at 35%, which was obtained from the RR plot. The results are shown in Figure 7.4. This figure shows the hyetograph, the corresponding observed hydrograph, computed hydrographs using the property time at each pit inlet of 2 and 5 min. As expected, 2-min property time produced a higher peak discharge and a lesser time to peak discharge compared to those of 5 min property time. In both cases, the runoff volume is the same, as it should be.

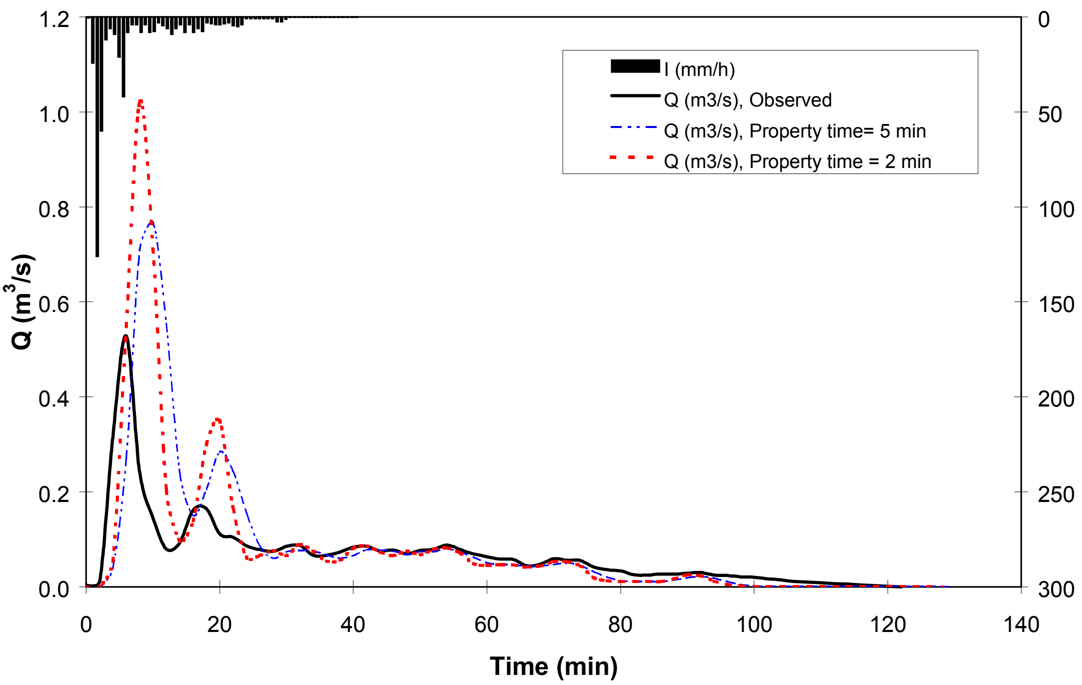


Figure 7.4: Hydrographs with Different Property Times for Catchment BA2A

For each calibration event of each catchment, the property time was initially assumed as five minutes and corresponding simulated hydrographs were obtained considering the values of DCIA and DS_i obtained from the RR plots (Section 7.2.3.3). It should be noted (as it is later described) that the runoff generated in calibration events was only from impervious areas. These simulated hydrographs were then visually compared with the observed hydrographs and investigated further whether the simulated hydrographs can be improved by changing the property time. The results of the study catchments showed that modelling could not be improved by changing the property time from five minutes to two minutes for selected calibration rainfall/runoff events. The reasons are given below:

- Five calibration rainfall/runoff events from each catchment showed different results with respect to property times. Some events even may require the larger property times (i.e. even larger than 5 min), while the others require smaller values, to match with the observed hydrographs.

- The events with multipeaks showed different characteristics. In some events, first peak requires an increase property time, while the second peak require a lower property time. Other multi-peak events had the opposite effect.
- In some cases, the property time may have to be increased to more than ten minutes to match with the observed time to peak discharge.

Since there is no strong evidence found in this study to discard the property time as five minutes, the property time of five minutes was considered in the model calibration of this Chapter.

7.3.4 Consideration of DCIA as a Parameter

Since DCIA, DS_i and N_p parameters are more sensitive to 'small' storm events, which occur frequently, an attempt was made to estimate these parameters accurately using different methods. Generally, urban drainage systems are designed using frequent 'small' runoff events to reduce 'nuisance' flooding, and therefore, the accurate estimation of these parameters is important. The directly connected impervious area (DCIA) is a very sensitive parameter in simulation of 'small' storm events. Therefore, it is very important to obtain this parameter accurately.

DCIA of a catchment is very difficult to measure physically, since it requires the identification of individual properties, which are connected to the drainage system. Ghafouri (1996) and Choi and Ball (1999) suggested that DCIA should be considered as a calibration parameter in urban drainage models. In this study, DCIA was computed using three methods as follow:

- approximate estimate using areal photographs,
- rainfall-runoff depth plots, and
- model calibration using hydrograph modelling up to five events for each catchment.

As stated earlier, the last method considers all hydrograph attributes (i.e. runoff volume, peak discharge, time to peak discharge and shape) in estimating model parameters, it was used as the preferred method to compute DCIA in this study. The other two methods were used to check the accuracy of the estimates obtained from the third method. The effect of DCIA on the simulated hydrograph was studied using catchment BA2A, considering a storm event from the selected calibration events (i.e. C2). The property time was kept constant at 5 minutes. Two values of DCIA (i.e. 31 and 46%) were considered and the results are shown in Figure 7.5. These two DCIA values were arbitrarily selected to show the effects of different DCIA on hydrograph response. As can be seen from Figure 7.5, as DCIA changes, the time to peak discharge does not change, while the peak discharge and runoff volume increase with increase in DCIA, as they should be. The reason for the same time to peak discharge may be rounding-off errors due to the computational time step of 2 min used. Because of this time step, the difference in time to peak cannot be seen.

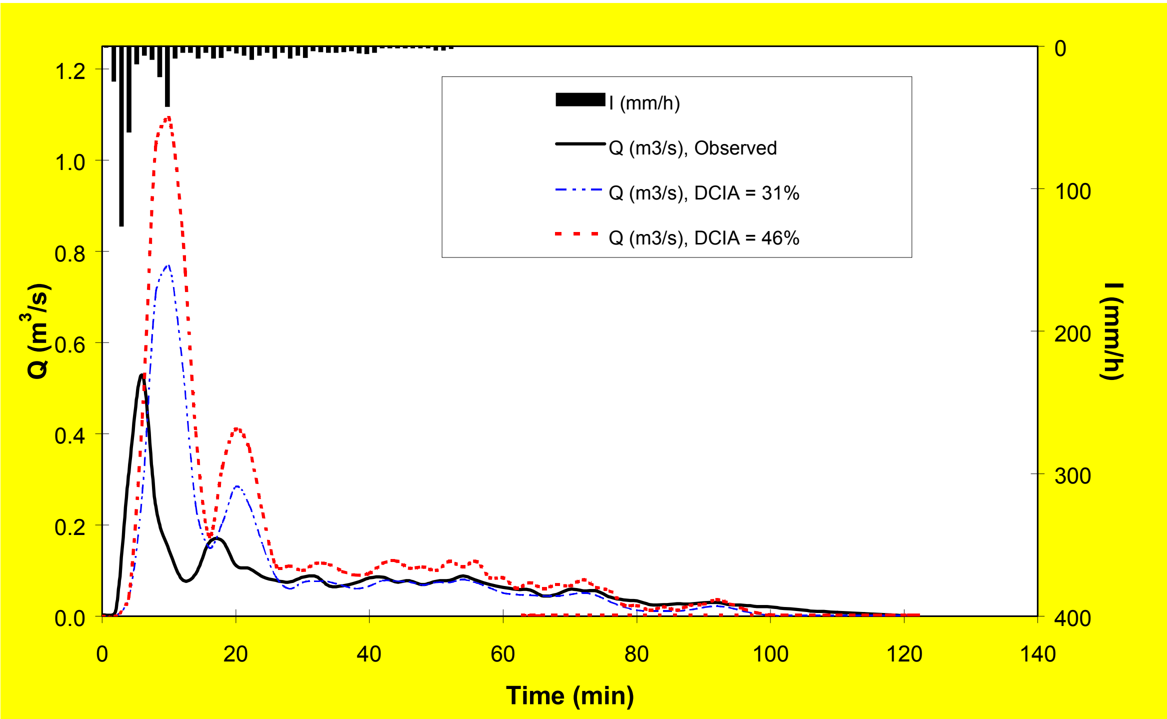


Figure 7.5: Hydrographs for Different Values of DCIA

7.3.5 Estimation of N_p

Initial numerical experiments for the selected events of this study suggested that N_p was not sensitive. Therefore, N_p was set at a constant value of 0.012.

7.3.6 Computational Time Step

Computer models that generate hydrographs can give different results for different computational time steps used in the calculations. O'Loughlin *et al.* (1998) studied the effect due to different time step using the ILSAX and DRAINS models, and found that increasing the calculation time step decreases the peak discharge by as much as 50%. Furthermore, they recommended the use 1 or 2 minutes time step for both these models.

ILSAX model can handle only up to 720 ordinates of the hydrographs. If one minute interval is used, some events cannot be analysed due to this upper limit of the number of ordinates. However, this may not be serious problem, since ILSAX can be recompiled with a higher number of time steps. Nevertheless, a computational time step of two minutes was used for all model runs. This was justified as the hydrographs were recorded at 2 minutes interval.

7.3.7 Catchment Subdivision

The results of the study on effect of catchment subdivision on hydrographs discussed in Section 6.7 showed that the peak discharge was dependent on the catchment subdivision selected. Fine-subdivision was the best modelling option, provided the required data are available. Therefore, an attempt was made to collect catchment data for fine-subdivision analysis of the study catchments, whenever possible. When required data were not available for fine-subdivision, data required at least for medium-subdivision were collected. Therefore, the study catchments were at least modelled using medium subdivision, but in most cases with fine subdivision.

The effect of using medium-subdivision (as opposed to fine subdivision) were investigated for those catchments which had data for fine subdivision modelling using the highest rainfall event selected for calibration. However, these catchments were modelled using fine subdivision in the calibration using hydrograph modelling in this chapter. Although catchment BA2A was used as the sample catchment in previous sections, the catchment H2 was used in this case since BA2A did not have information for the fine-subdivision analysis.

Table 7.4 shows the simulated and observed hydrograph attributes for fine and medium catchment subdivisions for catchment H2 for the most intensed observed storm (C4).

Table 7.4: Hydrograph Attributes for Fine and Medium Catchment Subdivision of Catchment H2 (Event C4)

Subdivision	Observed			Simulated		
	V (m^3)	P (m^3/s)	TTP (min)	V (m^3)	P (m^3/s)	TTP (min)
Fine	2116	0.762	68	2495	0.642	68
Medium				2495	0.629	68

V- Runoff volume, P- Peak discharge, TTP- Time to peak discharge

It can be seen that runoff volume does not change with subdivision, as it should be. However, the peak discharge changes with the subdivision. Theoretically, the time to peak discharge changes with the subdivision. However, it cannot be seen from the results in Table 7.4. The reason may be the difference of time to peak discharge from the two subdivisions is less than the computational time step of 2 min.

The type of subdivisions used for the study catchments is shown in Table 7.5. In total, the fine subdivision was used for 15 catchments and the medium subdivision was used for the remaining eight catchments.

Table 7.5: Catchment Subdivision used Calibration and Verification of Study Catchments

Catchment Tag	
Fine Subdivision	Medium Subdivision
BO1A, BO2A	BA2, BA2A, BA3, BA3A, BA3B
BR1, BR1A, BR2, BR2A, BR3	K2, K2A
H2, H2A	GI
K1, K1A, K1B, K3, K3A, K3B	

7.4 REVIEW OF MODEL CALIBRATION USING HYDROGRAPH MODELLING

Ideally, the mathematical models simulating the rainfall-runoff behaviour of catchments should have model parameters, which have direct physical significance and are measurable. Given the conceptual nature of mathematical models, the values of some of these parameters cannot be obtained from field measurements. For example, the impervious area depression storage (DS_i) cannot be estimated through field measurements of the catchment. Therefore, a calibration strategy is required to estimate these model parameters, which are either impossible or difficult to measure.

The goal of the calibration is to obtain the 'best' parameter set which produce a good fit between the measured and model predicted output, within a reasonable accuracy. This is achieved by adjusting the model parameters. The accuracy is fixed by establishing a criterion of goodness of fit of the simulated response of the model to that of the observed catchment response. This is commonly done using the trial and error calibration approach because of simplicity.

The trial and error method is widely being used to calibrate urban runoff models. Examples are Vale *et al.* (1986), Adams (1991); O'Loughlin *et al.* (1991); Dayaratne (1996) and Maheepala (1999). With this method, the simulated hydrographs corresponding to different parameter values are visually compared with the observed hydrograph. The parameter set, which match observed and simulated hydrographs best, are selected as the calibrated parameter set. The trial and error calibration method is subjective and time consuming.

An optimisation method was used in this study to estimate the ILSAX model parameters instead of the usual trial and error method. The parameter optimisation methods eliminate the subjectivity of the trial and error method and even reduce time spent on calibration in most cases. This was necessary in the study because a large number of catchments were used and each catchment considered several storm events in calibration. This was also important, since one of the major aims of this project was to derive the regional prediction equations for the ILSAX model parameters and that these model parameters (obtained from calibration of the study catchments) which were used in deriving these equations should be uniquely determined and free from subjective decisions as far as possible. The following sections review literature on parameter optimisation approach used to calibrate urban drainage models.

The parameter optimisation for rainfall/runoff models has been carried out in the past in two different forms. The first group (Dyer *et al.*, 1995; Dayaratne, 1996) modified the computer code of the modelling package to include an optimisation algorithm with an appropriate objective function. The advantage of this method is that the user can incorporate any objective function. However, the user needs a good knowledge of the computer programming language used in the package. The second group uses generalised parameter optimisation packages, which can be linked to the modelling package. In this method, the user does not have full control over the type of objective function to be used. However, the parameter optimisation can be carried out without the knowledge of computer programming. The examples of such generalised optimisation packages include NLFIT (Kuczera, 1987) and PEST (Watermark Numerical Computing, Australia, 1998). These two software programs have similar capabilities. In this study, the parameter optimisation was carried out using PEST. The PEST computer software is described in Section 7.4.3.

7.4.1 Objective Functions Used in Optimisation

Objective function is an essential component of a parameter optimisation procedure to measure the goodness of fit of observed and modelled output responses. The model parameter set is determined based on comparison of objective functions dealing with different parameter sets. The parameter set, which gives the 'best' objective function, is considered as the calibration result. The process does not involve visual comparison of observed and computed hydrographs for each parameter set, eliminating the subjectivity and time consuming nature of the trial and error method.

Available literature was reviewed to study the objective function used for calibration of urban drainage model parameters. All objective functions that were used in the past can be grouped into three categories. The first category considers specific features of the hydrograph such as peak of the hydrograph or hydrograph volume. These objective functions do not consider the overall shape of the hydrograph. The second category considers all hydrograph ordinates in the objective function and attempts to match the overall shape of the observed and computed hydrographs. In these two types of objective functions, calibration is done for individual storm events. The third group of objective functions consider several storm events together in calibrating the model parameters. These objective functions produce a single set of parameters that are good for simulating all those events. In some objective functions under the second and third groups, the weightage factor was used to control different parts of the hydrograph. This type of objective function allows the user to adopt different weightage factors according to the importance and accuracy of hydrograph attributes or different parts of the hydrograph. These three methods can be combined with each other to develop better objective functions. The literature in relation to the use of the three categories of objective functions is reviewed below.

The following notation is used for the review of objective functions. When objective functions are listed in this section, only the function is stated. The actual objective function was the minimisation of this function.

OF	is the objective function,

subscript 'o'	refers to observed,
subscript 'm'	refers to modelled,
V	is the runoff volume,
P	is the peak discharge,
TP	is the time to peak discharge,
Q(i)'	is the i^{th} discharge ordinate,
$\overline{Q}$	is the average discharge,
n	is the number of ordinates of hydrograph, and
N	is the number of storm events.

(a) First Group of Objective Functions

The first group of objective functions deal with only one hydrograph attribute. The obvious examples of this group are the objective functions that compare observed and simulated peak flows such as:

$$OF = P_o - P_m \tag{7.1}$$

$$OF = P_o / P_m \tag{7.2}$$

These equations can also be written using peak discharge and time to peak discharge. Similar objective functions to those of Equations 7.1 and 7.2 have been used in a number of studies (Kidd, 1978a,b; Hossain *et al.* 1978; Bouvior and Desbordes, 1990; Nathan, 1990; Naidu and Kearney, 1995 and Muncaster *et al.*, 1997). They deal with the difference between observed and modelled runoff volumes, flow peaks and times to peak discharge. These objective functions are given in Equations 7.3, 7.4 and 7.5. The main drawback of these objective functions is that only one attribute of the hydrograph is considered in the optimisation. For example, if the model is calibrated for runoff volume, then the peak discharge may not be adequately modelled with the calibrated parameter set.

$$OF = \{(V_o - V_m) / V_o\} \times 100\% \tag{7.3}$$

$$OF = \{(P_o - P_m) / P_o\} \times 100\% \tag{7.4}$$

$$OF = \{(TP_o - TP_m) / TP_o\} \text{x } 100\% \tag{7.5}$$

(b) Second Group of Objective Functions

The objective functions consider the hydrograph shape. A widely used objective function of this group of objective functions is the Integral Square Error (ISE) function (Bielawski, 1984) and its nondimensional forms. Several researchers (Kidd, 1978a,b; Sefe and Boughton, 1982; Bufill, 1989; Chiew and McMahon, 1993; and Goulburn-Murray Water, 1997) used different forms of the Integral Square Error function as given by Equations 7.6-7.13. Although, the standard ISE objective function (Equation 7.6) is biased to high flows, their nondimensional forms remove this biasness. Equation 7.10 optimises the flow in the vicinity of peak discharge.

$$\text{OF} = \sum_{i=1}^{n} (Q_o(i) - Q_m(i))^2 \tag{7.6}$$

$$\text{OF} = \frac{\sum_{i=1}^{n} (Q_o(i) - Q_m(i))^2}{\sum_{i=1}^{n} Q_o(i)} \tag{7.7}$$

$$\text{OF} = \frac{\sum_{i=1}^{n} \left((Q_o(i) - Q_m(i))^2\right)^{0.5}}{\sum_{i=1}^{n} Q_o(i)} \times 100 \tag{7.8}$$

$$\text{OF} = \frac{\sum_{i=1}^{n} \left(Q_o^2(i) - Q_m^2(i)\right)^{0.5}}{\sum_{i=1}^{n} Q_o(i)} \times 100 \tag{7.9}$$

$$\text{OF} = \frac{\sum_{i=1}^{n} \left(Q_o^2(i) - Q_m^2(i)\right)^{0.5}}{\sum_{i=1}^{n} Q_o(i)} \times 100 \text{ where } Q_o \geq P_o / 2 \tag{7.10}$$

$$\text{OF} = \frac{1}{\overline{Q_o}} \sum \left(Q_o(i)^{0.5} - Q_m(i)^{0.5} \right) \tag{7.11}$$

$$\text{OF} = \sum \left(Q_o(i)^{0.2} - Q_m(i)^{0.2} \right)^2 \tag{7.12}$$

$$\text{OF} = \sum \left(Q_o(i)^{0.5} - Q_m(i)^{0.5} \right)^2 \tag{7.13}$$

Dyer *et al.* (1994) proposed the objective function given in Equation 7.14 to calibrate the RORB Runoff Routing model Parameters. In this objective function, the hydrograph ordinates have been standardised with its peak.

$$\text{OF} = \frac{\sum_{i=1}^{n} \left(\frac{Q_0(i) - Q_m(i)}{P_o} \right)^2}{\sum_{i=1}^{n} \frac{Q_o(i)}{P_o}} \tag{7.14}$$

Lichty *et al.* (1968) proposed Equation 7.15 to consider the overall shape of the hydrograph. However, the contribution to the objective function is large for small flows especially when the base lengths of simulated and observed hydrographs are different. If there are zero flows either in the observed or the simulated hydrograph, then this objective function does not work. However, this problem can be overcome by adding a small number to all flow ordinates.

$$\text{OF} = \sum_{i=1}^{n} (\ln Q_o(i) - \ln Q_m(i))^2 \tag{7.15}$$

Bufill (1989) and Adams (1991) used Equation 7.16 in their studies. This objective function can simulate high as well as low flows better than Equation 7.15, since the differences are not squared.

$$\text{OF} = \frac{1}{n}\sum_{i=1}^{n} \left|\ln Q_o(i) - \ln Q_m(i)\right| \tag{7.16}$$

Watermark Numerical Computing (1998) used an objective function similar to ISE but incorporated some weighting factor (w_i). This objective function can be represented by Equation 7.17. The main advantage of this objective function is that the user can take control of any part of the hydrograph in optimisation by giving suitable weighting factors. The larger the weight pertaining to a particular observation the greater the contribution that the observation makes to the objective function. In most other objective functions, all observations carry equal weights in the parameter estimation process.

$$\text{OF} = \sum \left(w_i (Q_o(i) - Q_m(i))\right)^2 \tag{7.17}$$

(c) Third Group of Objective Functions

The third group of objective functions consider several storm events together in calibrating the model parameters. Under this category, the following objective functions were found in the literature. Wenzel and Voorhees (1980) used two separate objective functions (i.e. Equation 7.18 and 7.19) to calibrate the ILLUDAS model using several events simultaneously. The first objective function (Equation 7.18) considers only the runoff volume.

$$\text{OF} = \sum_{i=1}^{N}\left(\ln \frac{V_m}{V_o}\right)^2 \tag{7.18}$$

The second objective function (i.e. Equation 7.19) deals with the shape of the runoff hydrograph with more weight given to the peak discharge.

$$\text{OF} = \left[\sum_{i=1}^{N}\sum_{j=1}^{n} \frac{Q_{m,i,j}(j)}{P_{o,i}} \left(Q_{m,i,j}(j) - Q_{o,i,j}(j)\right)^2\right] \tag{7.19}$$

where $Q_{m,i,j}$ is the modelled runoff hydrograph ordinates for event i and time interval j,

$Q_{o,i,j}$ is the observed runoff hydrograph ordinates for event i and time interval j, and

$P_{o,i}$ is the observed runoff hydrograph peak for storm event i.

Alley and Smith (1990) used Equation 7.18 to calibrate the D3RM model considering several events simultaneously.

Pisaniello (1997) used the logarithmic form as in Equations 7.20 in his studies. In this equation, only peak discharge is considered.

$$\text{OF} = \sum_{i=1}^{n} (\ln P_o - \ln P_m) * 100 / \ln P_o \qquad (7.20)$$

Dayaratne and Perera (1999) used Equation 7.21, which produce the best parameter value considering the three hydrograph attributes and several storm events. In this equation, equal weight is given to the three attributes. This objective function has been successfully used in their study and also used in this thesis.

$$\text{OF} = 1/3\ \{(V_o\text{-}V_m)/V_o + (P_o\text{-}P_m)/P_o + (TP_o\text{-}TP_m)/TP_o\} \qquad (7.21)$$

These objective functions (Equation 7.18-7.21) produce a single set of parameters for storms of different magnitudes. They basically assume that model parameters are independent of the magnitude of storms. However, the objective functions of category (a) and (b) are useful when model parameters are storm dependent and when it is necessary to find a relationship between model parameters and storm characteristics.

7.4.2 Comparison of Different Objective Functions

Limited literature is available on the comparison of various objective functions related to urban drainage modelling. They are reviewed in this section. As discussed in detail in

Section 2.5.1.1, Kidd (1978a) applied Nash cascade surface routing submodel for three urban catchments to test the suitability of the objective functions. Four objective functions, Integral Square Error (ISE, Equation 7.8), Biased Integral Square Error (BISE, Equation 7.9), Partial Integral Square Error (PISE, Equation 7.10) and Peak error (PEAK, Equation 7.4) were used. Optimisation was carried out to produce the 'best' parameter set considering all events (which is called combined) and one parameter set for each individual event. Average parameter values (which is called average) were computed from the individual event parameter values. It was found that the PISE (combined) parameter set produced the best result, followed by the BISE (combined) parameter set second. The ISE (combined) parameter set and the ISE (average) set were ranked as third and fourth respectively.

This study concluded that the parameters obtained from PISE (combined) should be used if a single parameter set is to be used for modelling storms of different magnitudes. However, if values of the best parameter(s) obtained on individual events are required (to identify any possible relationship between parameters and storm characteristics for example), then the ISE function should be used.

Sorroshian and Gupta (1995) stated that the selection of objective function is user defined and can be subjective. The calibrated parameters obtained from two different objective functions are different (Kidd, 1978a and Dayaratne, 1996). Therefore, the suitable objective function should be decided by the user based on the objective of the study.

7.4.3 PEST Computer Software

The PEST (Parameter ESTimation) computer software was used to optimise the parameters of the ILSAX model for study catchments. PEST can be used with most computer simulation models without modifying the computer code of these models. PEST does not directly access the modelling package, but implicitly does it through input and output files. The modelling package can be written in any computer language.

PEST uses the Gauss-Marquardt-Levenberg method (Marquardt, 1963) for parameter estimation. In this method, parameter estimation is an iterative process. At the beginning of each iteration the relationship between model parameters and model-generated observations is linearised by formulating it as a Taylor series expansion about the currently best parameter set, hence the derivatives of all observations with respect to all parameters must be calculated. This linearised problem is then solved for a better parameter set, and the new parameters tested by running the model again. By comparing parameter changes and objective function improvement achieved through the current iteration with those achieved in previous iterations, PEST can tell whether it is worth undertaking another optimisation iteration, and if so, the whole process is repeated. This method performs well where model output is highly sensitive to the model parameters. In such cases, this method normally results in fewer model runs than most other parameter estimation methods, a definite advantage where model run-times are large (Watermark Computing, 1998).

PEST requires three types of input files. These are template files, instruction files and an input control file. Template files should be prepared for each model input file on which parameters are identified. Instruction file should be prepared for each model output file on which model simulated values are identified. An input control file supplies PEST the names of all template and instruction files, the names of the corresponding model input and output files, the problem size, the control variables, the initial parameter values, the observed values and weights, etc. For details of these files, the reader is referred to Section 7.5.2.

7.5 CALIBRATION OF STUDY CATCHMENTS USING HYDROGRAPH MODELLING

As stated previously, urban catchments respond differently to storm events of different magnitudes. If urban catchment models are calibrated without considering the magnitude of storm events, the calibrated model parameters may be in error. 'Small' events produce the runoff only from impervious area. Therefore, only the impervious area parameters need to be considered when the models are calibrated for 'small' storm events. For 'large' storm events, runoff is generally produced from both pervious and impervious areas. However, the runoff generation mechanism from impervious areas still remains the same as for 'small' storm events. Therefore, it can be said that the impervious area parameters can be calibrated using 'small' storm events first. The pervious area parameters can then be calibrated from 'large' storm events, keeping the impervious area parameters obtained from 'small' storm events constant.

When estimating model parameters using several storm events, the common practice (e.g. Kidd, 1978a, Dayaratne, 1996) had been to estimate these parameters for each storm event and then use an 'averaging' method to get a single parameter set. This method is generally satisfactory when there is only one model parameter that needs to be estimated. However, when there are several parameters that need to be calibrated, due to parameter interaction effects generally in models, the 'averaged' parameter set may not be the 'best' parameter set. Therefore, a different approach is employed in this study. The details of the approach are given in Section 7.5.1.

7.5.1 Adopted Calibration Procedure

An optimisation procedure called the two-stage inner/outer optimisation was developed to calibrate the model parameters for study catchments in this thesis. This procedure is represented by the process diagram shown in Figure 7.6. The parameter optimisation of each drainage catchment model was carried out in two stages. During the first stage, the model parameters responsible for 'small' storm events (i.e. DCIA and DS_i) were obtained. During the second stage, the additional parameters responsible for 'large' storm events (i.e. DS_p and CN) were obtained. AMC value was decided from Table 4.2 of this thesis. During the second stage, no changes were made to the parameters obtained from the first stage.

During each stage, a methodology was developed to get the 'best' set of parameters for each drainage catchment, considering all calibration storm events. The methodology is explained for the first stage. However, it is the same for the second stage, except that different model parameters are considered.

The first stage deals with the model parameters of DCIA and DS_i that are responsible for 'small' runoff events. An optimised set of the above parameters was obtained for each storm event by linking PEST with ILSAX model through input and output files. The default objective function of PEST was used in this optimisation with equal weights for all hydrograph ordinates, which minimised the sum of squared differences between modelled and observed hydrograph ordinates. This is the inner optimisation for stage 1.

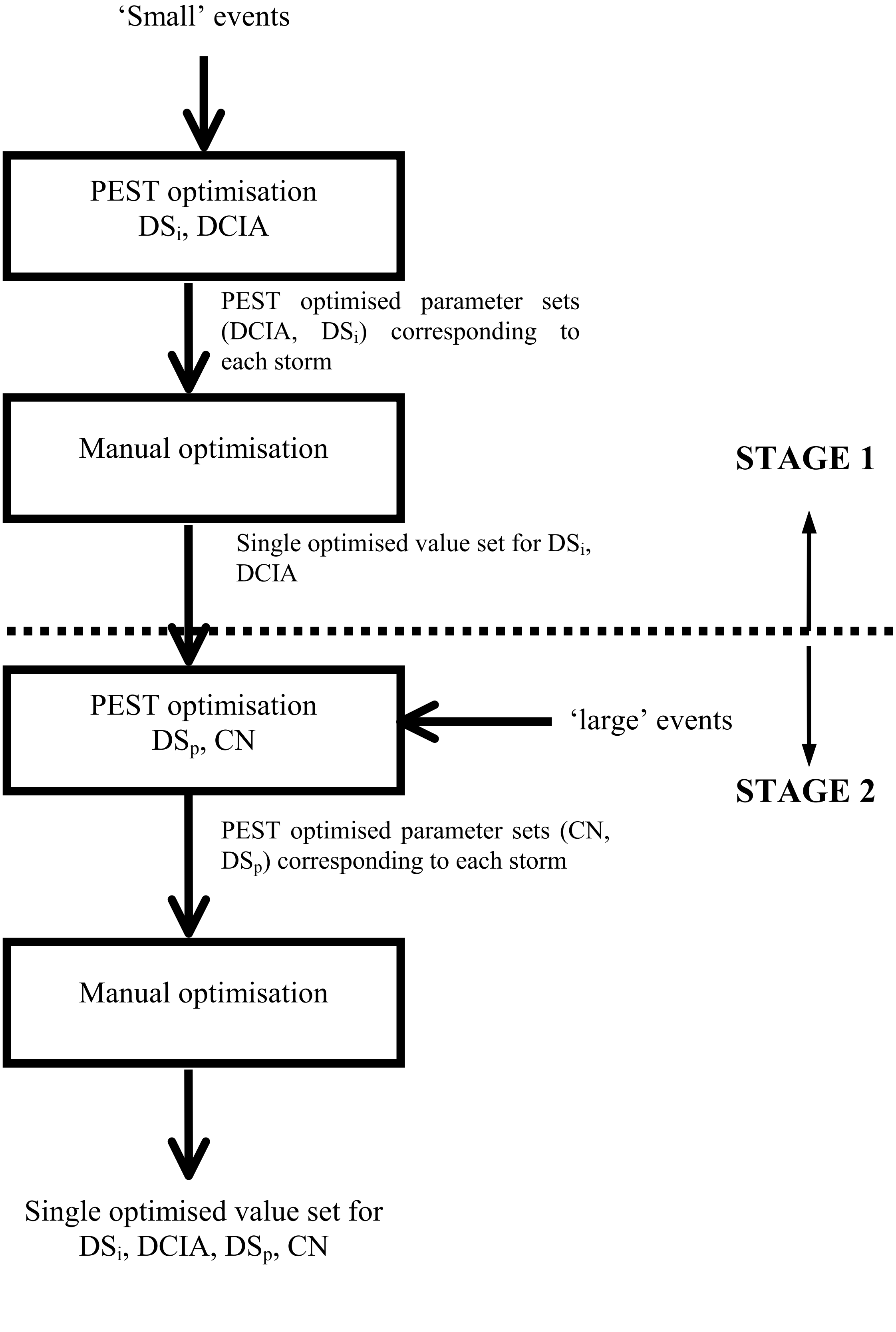


Figure 7.6: The Process Diagram for Optimisation Algorithm

An outer optimisation for stage 1 was then carried out with respect to all selected calibration storm events (5 events in this study). The objective function used in the outer optimisation considers the effect of all events and the effect of output responses of runoff volume, peak discharge and time to peak discharge. These output responses are important for water resources planners in urban stormwater management (Section 2.5.3). The outer optimisation produces the 'best' parameter set from the individual parameter sets corresponding to each event. Mathematically, the objective function used in the outer optimisation can be written as:

$$\underset{i,j}{Min} \sum_{j=1,n..} \sum_{i=V,P,TP} \left[\left(O_{i,j} - Ob_{i,j} \right) / Ob_{i,j} \right] \tag{7.22}$$

where
$O_{i,j}$ is the modelled output response (i) of V, P or TP and storm event j,
$Ob_{i,j}$ is the observed output response (i) of V, P or TP and storm event j,
V is the runoff volume,
P is the peak discharge,
TP is the time to peak discharge, and
N is the number of storm events.

The inner optimisation (by linking PEST with ILSAX model) was carried out automatically using the required data files, while the outer optimisation was carried out manually. This inner/outer optimisation procedure gives the set of model parameters, which can be considered as the 'best' for all storm events analysed. Once a single set of DCIA and DS_i are estimated from the first stage optimisation, a single set of pervious area parameters of DS_p and CN are optimised from the second stage, similar to the first stage.

7.5.2 Preparation of Data Files

The inner optimisation (through PEST) requires several data files. They can be categorised into three sets. First set is the ILSAX model data files. For each storm event (both 'small' and 'large'), two data files were created, namely the pipe file and the rainfall file. However, there were no 'large' storm events for Melbourne metropolitan catchments. Both 'small' and 'large' storm events were considered for the Giralang catchment. The parameters DCIA and DS_i obtained from the rainfall/runoff depth plot were used in pipe file and rainfall file respectively. These estimates were necessary to initiate the optimisation process in PEST. The parameters DS_p and CN parameters were also included in the rainfall file. Initial values of these two parameters were taken from the middle values of the recommended range in the ILSAX user manual. AMC value was taken from Table 4.2 of this thesis considering 5 days rainfall depth prior to the event and entered in the rainfall file. All modelling options recommended in Section 6.8 except for pit inlet capacity were used in the rainfall and pipe files. As justified in Section 7.3.2, the infinite capacity option was used for modelling pit inlets for 'small' storm events of the Melbourne metropolitan catchments. However, actual pit capacities were used for the Giralang catchment, since 'large' storm events were analysed for this catchment. A computational time step of two minutes was used in the rainfall file. Once the pipe and rainfall files were prepared for each storm event, the ILSAX model was run to produce the output files, before using PEST. The separate PEST files had to be prepared for each selected storm event for the calibration.

The second category of data file consists of files required for running PEST. As stated in Section 7.4.3, they are pipe template file, rainfall template file, output instruction file, and parameter file.

Template files can be prepared by slight modifications to data files in order to indicate where adjustable parameter values are to be written to that file. These changes are in the form of "parameter space identifiers" character strings which both identify where a particular parameter is found, and provide the parameter with a name.

PEST must be instructed on how to read a model output file and identify model-generated observations. Unfortunately, observations cannot be read from model output files using the template concept since many models cannot be relied upon to produce an output file of identical structure on each model run. Therefore, PEST requires, then, that for each model output file which must be opened and perused for observation values, an instruction file be provided detailing how to find those observations. This instruction file can be prepared using a text editor.

The third category is a PEST control file which "brings it all together", supplying PEST with the names of all template and instruction files together with the model input/output files to which they pertain. It also provides PEST with the model name, parameter initial estimates, field or laboratory measurements to which model outcomes must be matched, prior parameter information, and a number of PEST variables, which control the implementation of the Gauss-Marquardt-Lavenberg method.

These files were created for each storm event. The data file structure for PEST-calibration of this study is summarised by the flow chart shown in Figure 7.7.

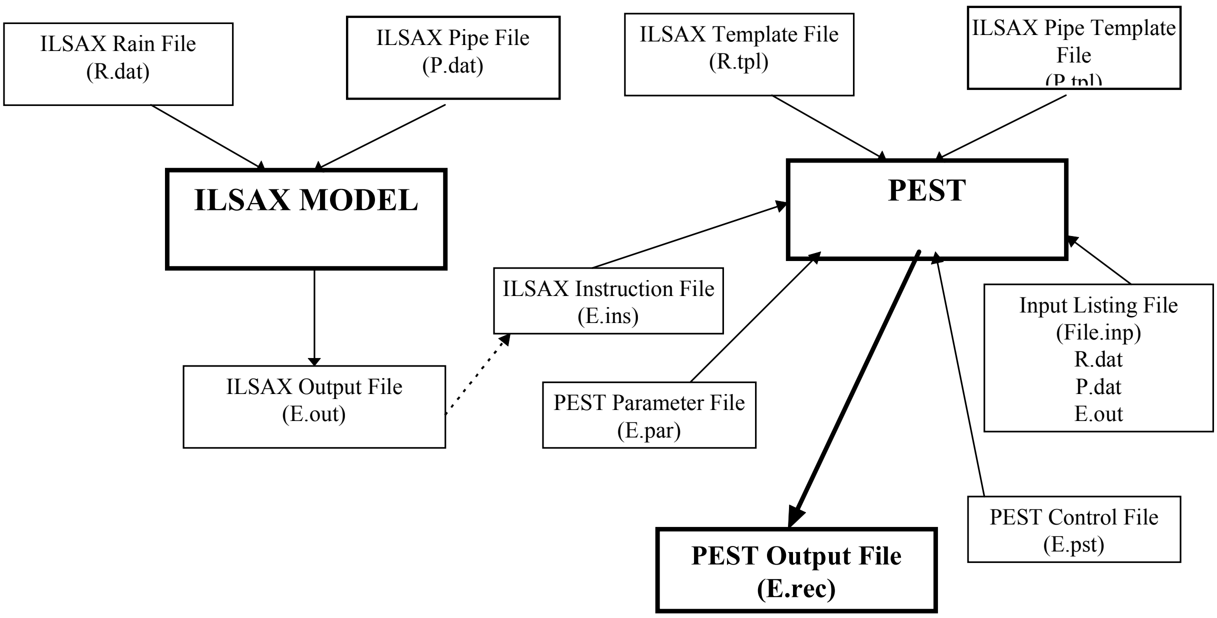


Figure 7.7: Flow Chart Representing the Data Files for PEST-Calibration

7.5.3 Calibration of Study Catchments in Melbourne Metropolitan Area

The two-stage inner/outer optimisation procedure described in the previous section was used to obtain the 'best' parameter set for each catchment. The first stage produced individual sets of model parameters (i.e. DCIA and DS_i) corresponding to each of the selected 'small' events, with one of them as the 'best' parameter set. The initial studies performed with different seed values of DCIA and DS_i showed that different optimisation runs yielded the same set of optimised parameters, and hence giving the 'optimum' set of parameters for the storm event under consideration. However, there were no ‘large’ events, large enough to produce pervious area runoff. Therefore, the stage 2 of the optimisation was not carried out for the Melbourne metropolitan catchments and the pervious area parameters (i.e. DS_p and CN) could not be estimated.

The inner optimisation (i.e. PEST optimisation) used the DCIA and DS_i obtained from the RR plots (Section 7.2.3.3) as the seed to initiate the optimisation. There is a possibility that these initial values play a role in the optimisation process and may trap the optimum parameter set to produce a ‘local’ optimum. The ranges for DCIA between 0 to 100%, and for DS_i between 0 - 2 mm were adopted in PEST optimisation.

The hyetograph, observed hydrograph and two hydrographs from the optimisation procedure corresponding to five ‘small’ storm events of catchment BA2A are shown in Figure 7.8. The two hydrographs in each plot represent the hydrographs due to the PEST-optimised parameter set for the storm event and the 'best' parameter set considering all storm events. As can be seen from Figure 7.8, the ‘best’ set of model parameters produces a reasonable match between observed and modelled hydrographs for all storm events. The calibration plots (similar to Figure 7.8) for the other catchments are given in Appendix E. Assessment of the calibration plots for all catchments is given below under separate headings.

7.5.3.1 Calibration of Catchment BA2

From the selected five events used for calibration, only four events provided successful PEST calibration results. One event did not converge to the optimum solution and the PEST optimisation stopped halfway. The results are shown in Figure E1 of Appendix E. All these events in Figure E1 had multipeaks, and the calibration showed that the shape and time to peak discharge were satisfactorily modelled for all these events with the ‘best’ parameter set. One event simulated the peak discharge well, while the other three events overestimated the peak.

7.5.3.2 Calibration of Catchment BA2A

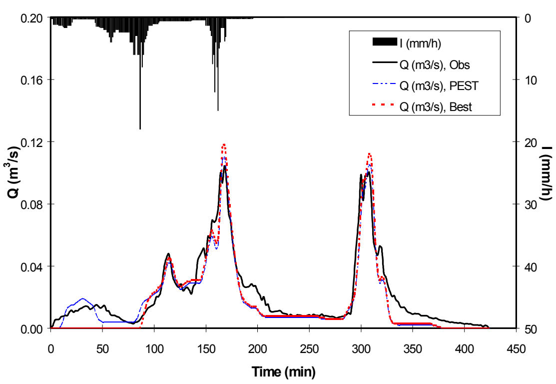
From the selected five events for calibration, all events provided successful PEST calibration results. These five calibration results are shown in Figure E2 of Appendix E. They were also shown in Figure 7.8, as a sample plot. All these events had multipeaks. Four events simulated the shape, peak discharge, time to peak discharge and multipeaks well, while the other event showed a time shift.

7.5.3.3 Calibration of Catchment BA3

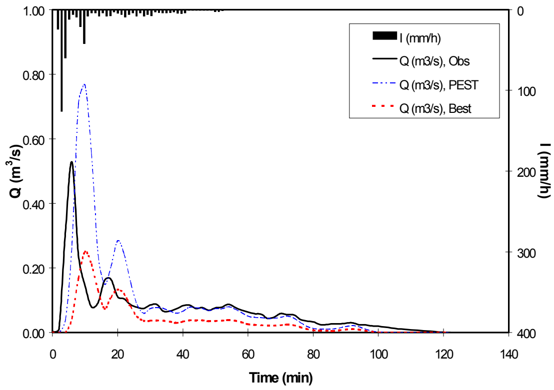
From the selected five events used for calibration, only four events provided successful PEST calibration results. One event did not converge to the optimum solution and PEST optimisation stopped halfway. The results are shown in Figure E3 of Appendix E. All these events had multipeaks, and the calibration showed that the shape, time to peak discharge and multipeaks were modelled well for all these events.

7.5.3.4 Calibration of Catchment BA3A

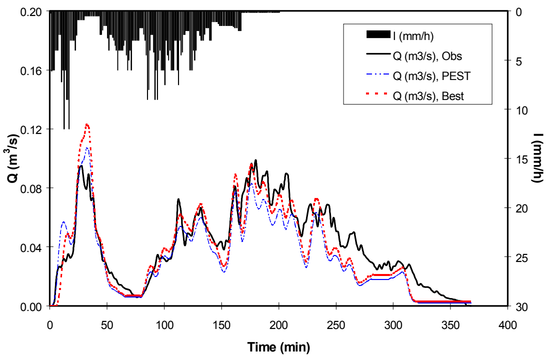
From the selected five events used for calibration, all events provided successful PEST calibration results. The results are shown in Figure E4 of Appendix E. All these events had multipeaks, and the calibration showed that the four events simulated the shape, peak discharge, time to peak discharge and multipeaks well while the other event did not simulate the peak discharge correctly.



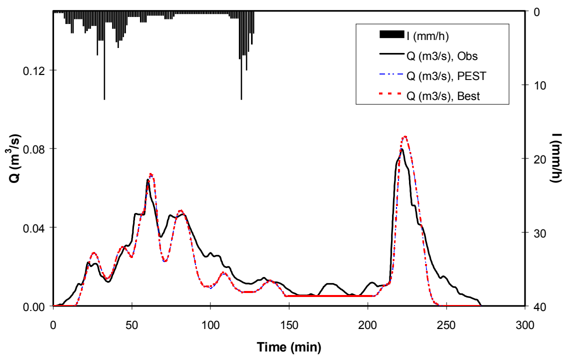
(a) Event C1



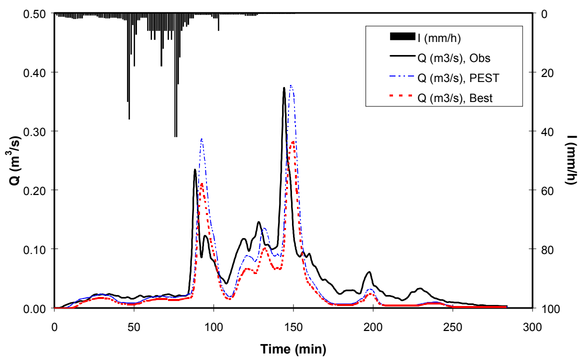
(b) Event C2



(c) Event C3



(d) Event C4



(e) Event C5

Figure 7.8:Hyetograph and Hydrographs for Calibration Events of Catchment BA2A

7.5.3.5 Calibration of Catchment BA3B

From the selected five events used for calibration, all events provided successful PEST calibration results. The results are shown in Figure E5 of Appendix E. All these events had multipeaks, and the calibration showed that the shape, peak discharge, time to peak discharge and multipeaks modelled for all these events with reasonably accuracy.

7.5.3.6 Calibration of Catchment BO1A

From the selected five events used for calibration, only four events provided successful PEST calibration results. One event did not converge to the optimum solution and the PEST optimisation stopped halfway. The results are shown in Figure E6 of Appendix E. All these events had multipeaks, and the calibration showed that the shape and time to peak discharge were correctly modelled for all these events.

7.5.3.7 Calibration of Catchment BO2A

From the selected five events used for calibration, all events provided successful PEST calibration results. One event did not converge to the optimum solution and the PEST optimisation stopped halfway. The results are shown in Figure E7 of Appendix E. All these events had multipeaks, and the calibration showed that the shape, peak discharge, time to peak discharge and multipeaks were modelled for all these events with reasonable accuracy.

7.5.3.8 Calibration of Catchment BR1

From the selected five events used for calibration, only four events provided successful PEST calibration results. One event did not converge to the optimum solution and the PEST optimisation stopped halfway. The results are shown in Figure E8 of Appendix E.

All these events had multipeaks, and the calibration showed that the shape, time to peak discharge and multipeaks were modelled satisfactorily for all these events.

7.5.3.9 Calibration of Catchment BR1A

From the selected five events used for calibration, only four events provided successful PEST calibration results. One event did not converge to the optimum solution and the PEST optimisation stopped halfway. The results are shown in Figure E9 of Appendix E. All these events had multipeaks, and the calibration showed that the shape, peak discharge and time to peak discharge were modelled satisfactorily for all these events.

7.5.3.10 Calibration of Catchment BR2

From the selected five events used for calibration, all events provided successful PEST calibration results. The results are shown in Figure E10 of Appendix E. All these events had multipeaks, and the calibration showed that the shape and time to peak discharge were modelled satisfactorily for three events, while the other two events showed in time-shift in the modelled hydrographs.

7.5.3.11 Calibration of Catchment BR2A

From the selected five events used for calibration, all events provided successful PEST calibration results. The results are shown in Figure E11 of Appendix E. All these events had multipeaks, and the calibration showed that the shape, peak discharge, time to peak discharge and multipeaks were modelled for all these events with reasonable accuracy.

7.5.3.12 Calibration of Catchment BR3

From the selected five events used for calibration, all events provided successful PEST calibration results. The results are shown in Figure E12 of Appendix E. All these events had multipeaks, and the calibration showed that the shape, peak discharge, time to peak discharge and multipeaks were modelled for all these events with reasonable accuracy.

7.5.3.13 Calibration of Catchment H2

From the selected five events used for calibration, all events provided successful PEST calibration results. The results are shown in Figure E13 of Appendix E. All these events had multipeaks, and the calibration showed that the shape, peak discharge, time to peak discharge and multipeaks were modelled for all these events with reasonable accuracy.

7.5.3.14 Calibration of Catchment H2A

From the selected five events used for calibration, only four events provided successful PEST calibration results. One event did not converge to the optimum solution and the PEST optimisation stopped halfway. The results are shown in Figure E14 of Appendix E. All these events had multipeaks, and the calibration showed that the shape, time to peak discharge and multipeaks were modelled satisfactorily for all these events.

7.5.3.15 Calibration of Catchment K1

From the selected five events used for calibration, all events provided successful PEST calibration results. The results are shown in Figure E15 of Appendix E. All these events had multipeaks, and the calibration showed that the shape, peak discharge, time to peak discharge and multipeaks were modelled for all these events with reasonable accuracy.

7.5.3.16 Calibration of Catchment K1A

From the selected five events used for calibration, all events provided successful PEST calibration results. The results are shown in Figure E16 of Appendix E. All these events had multipeaks, and the calibration showed that the shape, peak discharge, time to peak discharge and multipeaks were modelled for all these events with reasonable accuracy.

7.5.3.17 Calibration of Catchment K1B

From the selected five events used for calibration, all events provided successful PEST calibration results. The results are shown in Figure E17 of Appendix E. All these events had multipeaks, and the calibration showed that the shape, time to peak discharge and multipeaks discharge were satisfactorily modelled only for three events using the 'best' parameter set. In other two events peak discharges were significantly different.

7.5.3.18 Calibration of Catchment K2

From the selected five events used for calibration, only three events provided successful PEST calibration results. Two events did not converge to the optimum solution and the PEST optimisation stopped halfway. The results are shown in Figure E18 of Appendix E. All these events had multipeaks, and the calibration showed that simulated the shape, time to peak discharge and multipeaks were modelled well for two events, while one event overestimated the peak.

7.5.3.19 Calibration of Catchment K2A

From the selected five events used for calibration, all events provided successful PEST calibration results. The results are shown in Figure E19 of Appendix E. All these events had multipeaks, and the calibration showed that the shape, peak discharge, time to peak discharge and multipeaks were modelled for all these events with reasonable accuracy.

7.5.3.20 Calibration of Catchment K3

From the selected five events used for calibration, only four events provided successful PEST calibration results. One event did not converge to the optimum solution and the PEST optimisation stopped halfway. The results are shown in Figure E20 of Appendix E. All these events had multipeaks, and the calibration showed that the shape, peak discharge and time to peak discharge and multipeaks were modelled for all these events with reasonable accuracy.

7.5.3.21 Calibration of Catchment K3A

From the selected five events used for calibration, all events provided successful PEST calibration results. The results are shown in Figure E21 of Appendix E. All these events had multipeaks, and the calibration showed that the shape, peak discharge, time to peak discharge and multipeaks were modelled for all these events with reasonable accuracy.

7.5.3.22 Calibration of Catchment K3B

From the selected five events used for calibration, only four events provided successful PEST calibration results. One event did not converge to the optimum solution and the PEST optimisation stopped halfway. The results are shown in Figure E22 of Appendix E. All these events had multipeaks, and the calibration showed that the shape, time to peak discharge and multipeaks were modelled well for all these events.

7.5.3.23 Results of Calibration

The impervious area parameters of DCIA and DS_i were calibrated for 22 Melbourne catchments and 5 events (in most cases). In general, a good match was seen between modelled and observed hydrographs for calibration storm events. The parameter values of DCIA and DS_i obtained for these 22 Melbourne catchments are shown in Figures 7.9 and 7.10 respectively. These figures show the parameter values obtained for each storm event used in calibration and the 'best' set of parameters obtained considering all events. As can be seen from these figures, there is a fair amount of scatter in the model parameters of each catchment, obtained from different storm events. This scatter could be due to the deficiency in the model structure (i.e. model does not simulate all processes of the drainage system adequately), and inaccuracies in rainfall/runoff and other data used. In addition, when many parameters have to be calibrated simultaneously, there may be different combinations of parameters that yield (more or less) the same output response. As can be seen from these figures, based on calibration corresponding to each storm of each catchment DCIA varies from 15% to 93% and DS_i from 0 to 1.8 mm. DCIA varies from 24 to 75% (Figure 7.9) and DS_i varies from 0 to 1.8 mm (Figure 7.10) when only the 'best' parameter set is considered. The suggested range for DS_i in ILSAX model User's Manual (O'Loughlin, 1993) is 0-2 mm, which is supported from the results of this study.

7.5.3.24 Comparison of calibration results with other source of information

As stated in Section 7.3.4, three different methods were considered in this study to estimate DCIA of a catchment. The first method was to estimate DCIA from the areal photographs. The second method was to obtain DCIA from the RR plots. The third method was to obtain DCIA from model calibration using hydrograph modelling.

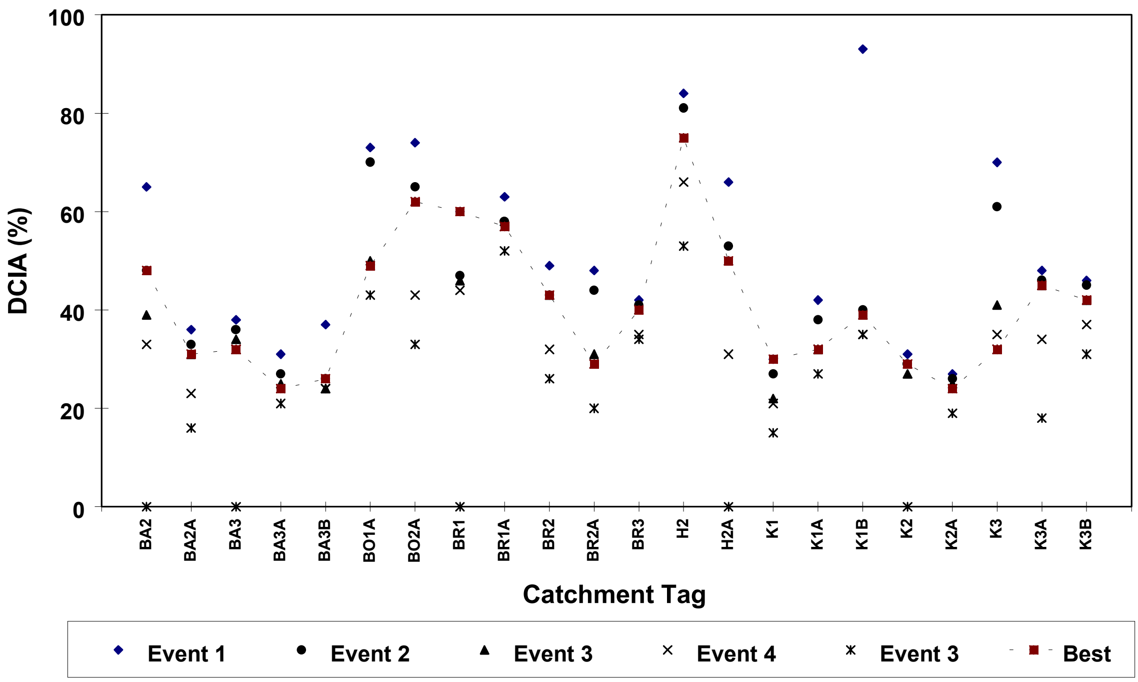


Figure 7.9: Optimised DCIA for Study Catchments

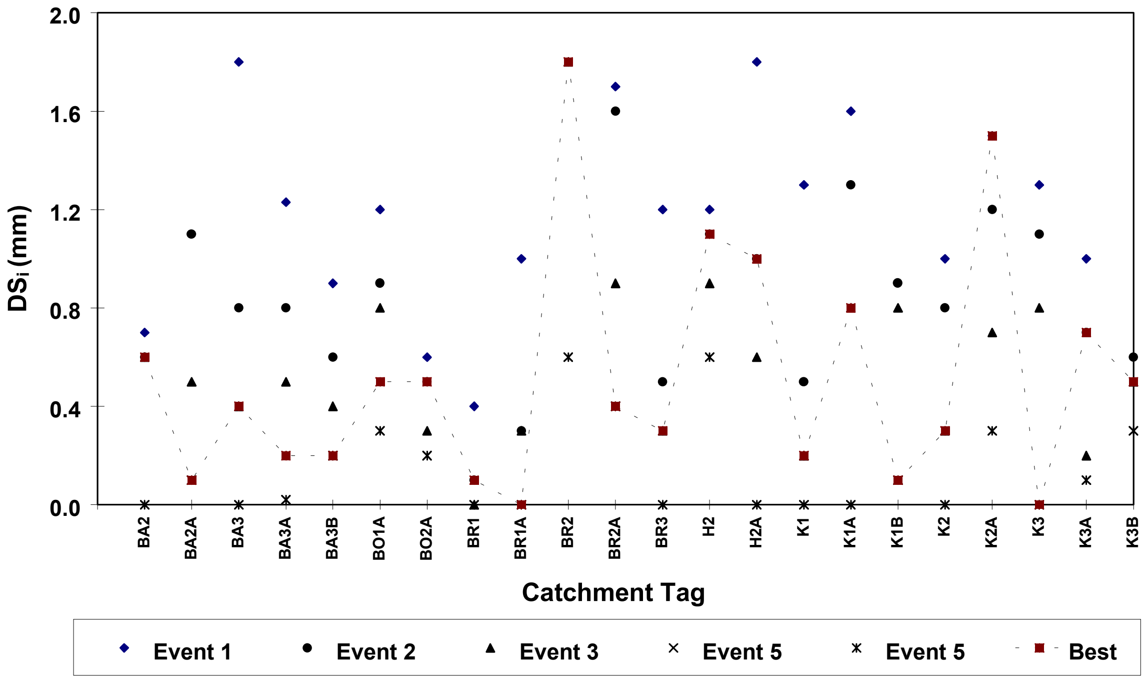


Figure 7.10: Optimised DS_i for Study Catchments

In this study, DCIA values for the study catchments were obtained from the model calibration using hydrograph modelling and the other two methods were used to check the accuracy of the estimates obtained from hydrograph modelling. The reasons for this are explained in Section 7.3.4. DS_i values for the study catchments were also selected from the model calibration using hydrograph modelling and the results were compared with the values from RR plots. DCIA and DS_i values obtained from these methods for the Melbourne metropolitan catchments are given in Table 7.6. As can be seen from Table 7.6 areal photographs were available only for few catchments. The difference of DCIA values of 17 catchments (from total 22 catchments) was within 10% of the values obtained from calibration and RR plots. The difference of DS_i values obtained from RR plots and calibration were within 0.25 mm for 11 catchments out of 22 study catchments.

The comparison plots of DCIA and DS_i from calibration using hydrograph modelling and RR plot are given in Figures 7.11 and 7.12. This figure suggests that DCIA values obtained from the both methods are not much different. However, Figure 7.12 suggests that DS_i values obtained from the two methods are significantly different.

7.5.4 Calibration of Giralang Catchment

As mentioned in Section 7.2.1, the Giralang catchment from Canberra was selected to demonstrate the calibration procedure to obtain pervious area parameters, since the recorded rainfall/runoff events in Melbourne metropolitan catchments were not large enough to calibrate these parameters. The main steps of the calibration procedure used for the Giralang catchment are summarised below, although stage 1 calibration is exactly the same as for Melbourne metropolitan catchments.

a) Rainfall/runoff events were selected from the database. Rainfall and runoff depths of each selected storm event were calculated, and plotted on RR plot. If there were events above 45^0 line of the RR plots, those were removed. However, no such events were found from the selected events of this catchment. The 'large' storm events, which show a large departure from the regression line (above the line), were separated. The remaining events were again plotted on a RR plot, and DS_i and DCIA values were obtained from the plot. This RR plot is shown in Figure 7.13. This plot shows a very good correlation among data points. Using this plot, DCIA and DS_i for the Giralang catchment were estimated as 19% and 0.26 mm respectively.

Table 7.6: DCIA and DS_i Values from Different Methods for Study Catchments

Catchment Tag	DCIA			DS_i		Comments
	Areal Photo	RR Plot	Hydrograph Modelling	RR Plot	Hydrograph Modelling	
BA2		47	48	0.57	0.60	Major
BA2A		35	31	0.37	0.10	Subcatchment
BA3		42	32	0.69	0.40	Major
BA3A		35	24	0.69	0.20	Subcatchment
BA3B		32	26	0.59	0.20	Subcatchment
BO1A		50	49	0.14	0.50	Subcatchment
BO2A		40	42	0.50	0.50	Subcatchment
BR1		53	60	0.19	0.10	Major
BR1A		58	57	0.29	0.00	Subcatchment
BR2		52	43	0.31	1.80	Major
BR2A		54	29	0.22	0.40	Subcatchment
BR3		42	40	0.50	0.30	Major
H2	86	77	75	0.90	1.10	Major
H2A	70	67	50	1.03	1.00	Subcatchment
K1	40	34	30	0.41	0.20	Major
K1A	45	44	32	0.34	0.80	Subcatchment
K1B	51	49	39	1.29	0.10	Subcatchment
K2	40	35	29	1.17	0.30	Major
K2A	35	26	24	1.77	1.50	Subcatchment
K3	40	37	32	0.08	0.00	Major
K3A	54	52	45	0.38	0.70	Subcatchment
K3B	45	28	42	0.29	0.50	Subcatchment

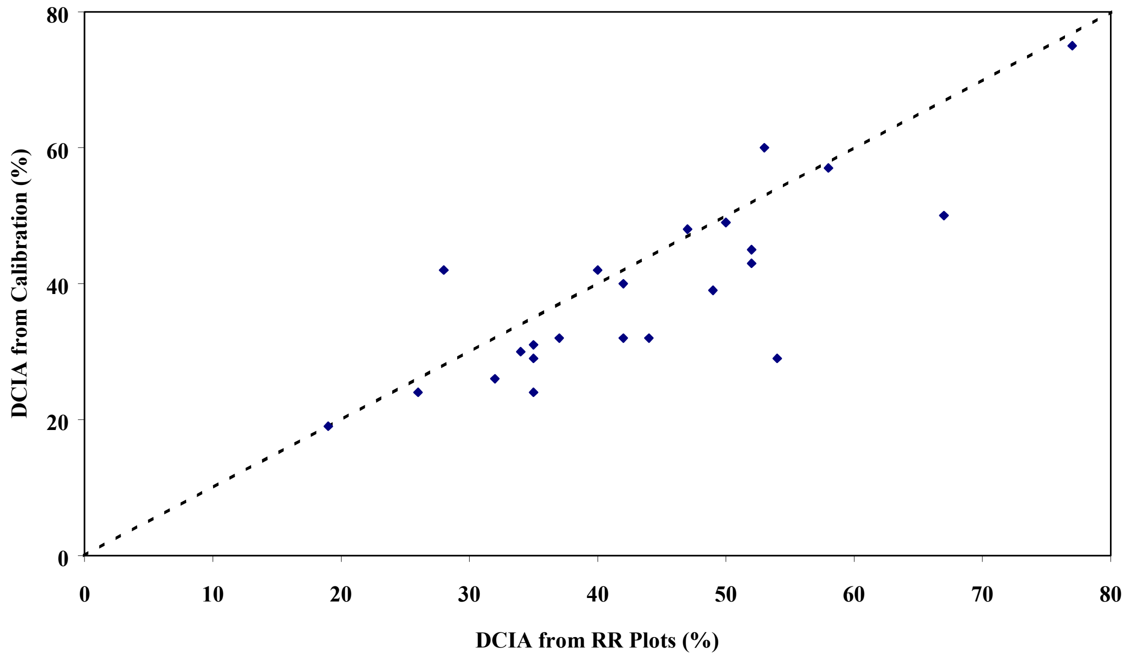


Figure 7.11: DCIA from Calibration and RR Plots for Study Catchments

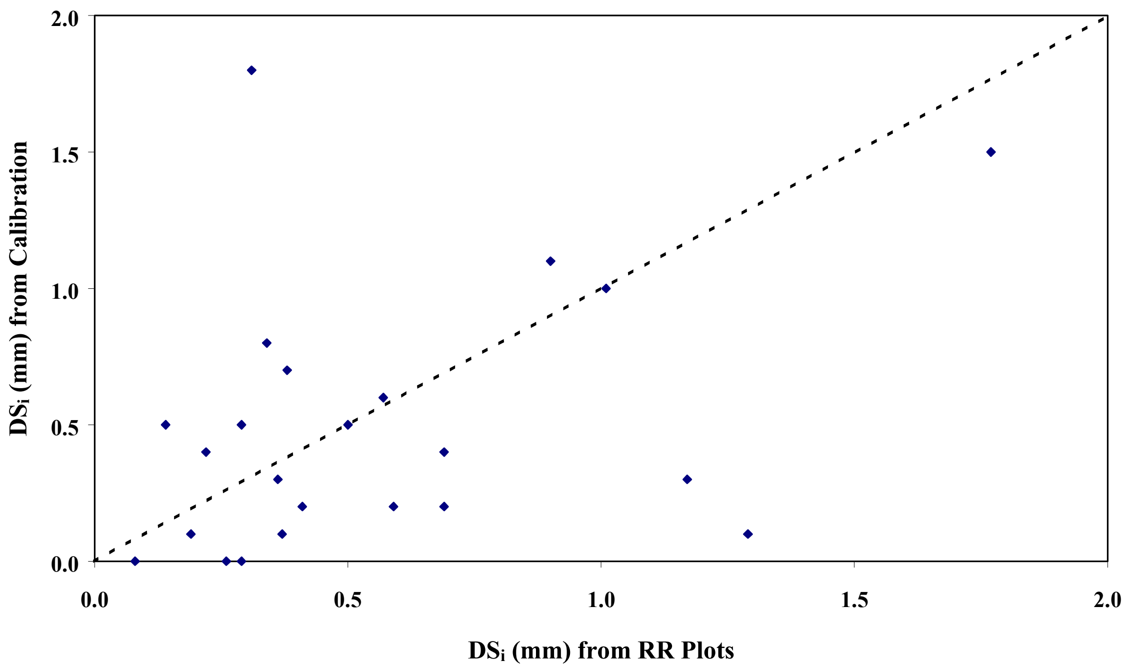


Figure 7.12: DS_i from Calibration and RR Plots for Study Catchments

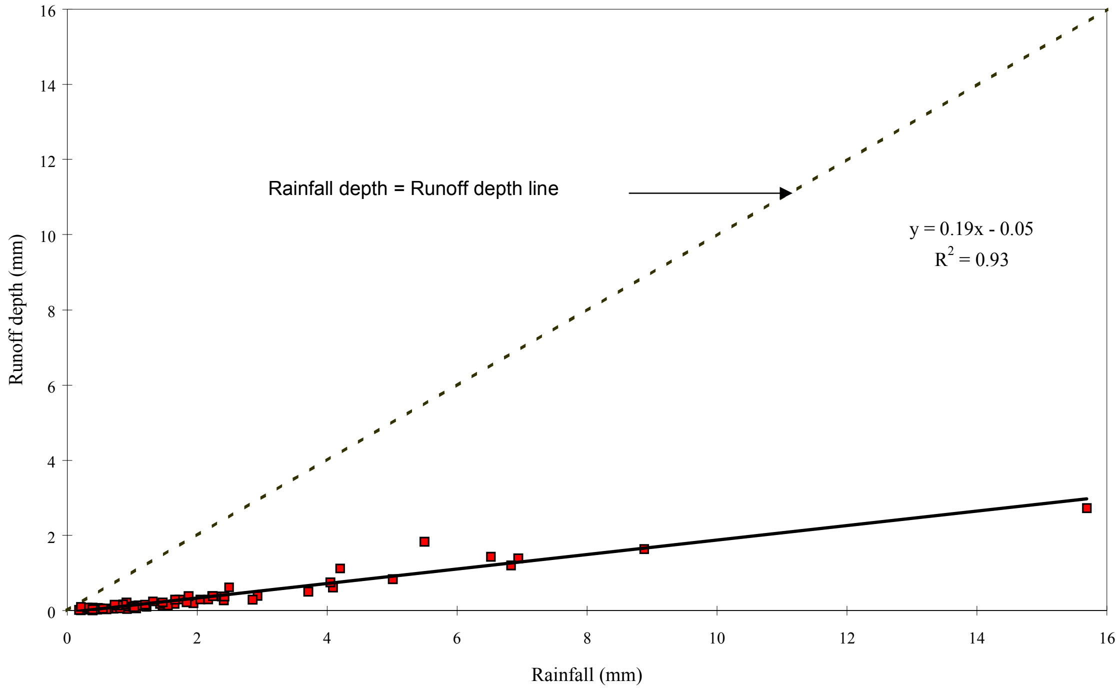


Figure 7.13: Runoff Depth versus Rainfall Depth Plot of Catchment GI

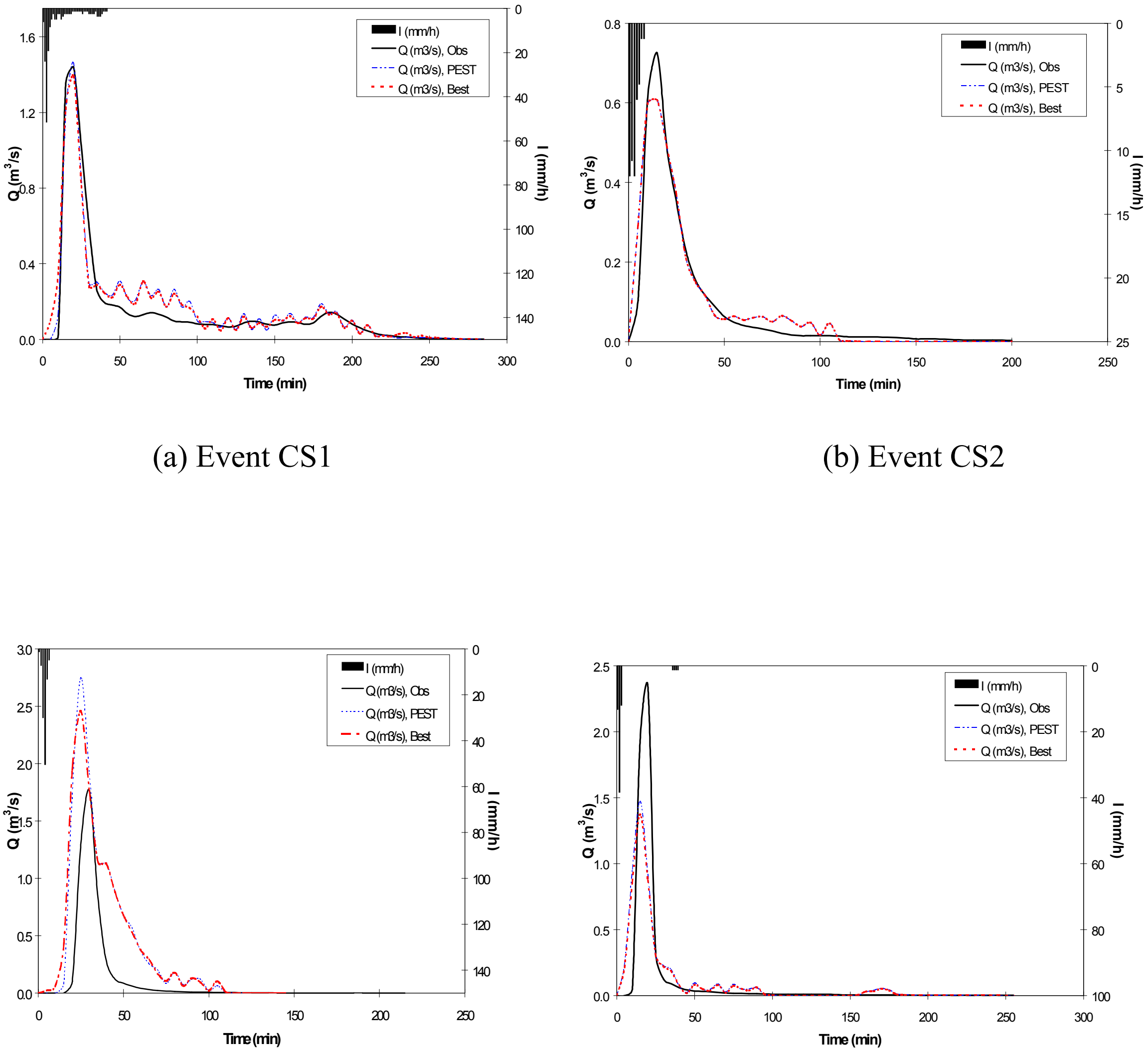
b) Five 'small' and five 'large' well-defined storm events were separated from the data set in (a). DCIA and DS_i obtained from the RR plot were used as the seed for Stage 1 optimisation. However, numerical experiments conducted with different seeds did not produce different optimum parameter sets for selected storms. The ranges for DCIA between 0 to 100%, and for DS_i between 0 - 2 mm were adopted in PEST optimisation. Five sets of parameter values for DCIA and DS_i values were obtained from the calibration of five 'small' storm events. The PEST calibrated five sets of parameter values for the Giralang catchment is given in Table 7.7.

From the calibrated results, the storm event CS5 showed a very high value for DS_i. The general range for DS_i according to most of literature is 0-2 mm (e.g. U.S. Environmental Protection Agency, 1992 and O'Loughlin, 1993). Therefore, the storm event CS5 was discarded from further analysis. The best parameter set was selected by applying Equation 7.21 as the objective function considering the remaining four events. The parameter set corresponding to storm event CS2 (i.e. DCIA = 19 and DS_i = 0) was found to be the best parameter set for the Giralang catchment. The calibration plots of the four 'small' storm events are shown in Figure 7.14. All four events simulated hydrograph shape satisfactorily with the 'best' parameter set. However, two events overestimated the peak discharge while one event underestimated. The remaining event simulated the peak discharge well.

Table 7.7: Calibration Values of Giralang Catchment for 'Small' Storm Events

Event	DCIA (%)	DS_i (mm)
CS1	20	0.60
CS2	19	0.00
CS3	25	2.00
CS4	21	0.00
CS5	25	4.50

c) For Stage 2 calibration, seed values for 'large' storm events using the fixed value for DCIA and DS_i obtained from (b). The initial values for DS_p and CN were taken as the middle value of the range given in the ILSAX manual. AMC value was taken from Table 4.2 of this thesis, according to five days total rainfall depth prior to each storm event. In Stage 2, DS_p and CN were considered as parameters in optimisation while DS_i and DCIA were fixed as the values obtained from Stage 1 optimisation. Then the PEST optimised parameter sets were obtained for five 'large' storm events. Then the PEST optimised parameter sets were obtained for five 'large' storm events and they are given in Table 7.8.



(a) Event CS1

(b) Event CS2

(c) Event CS3

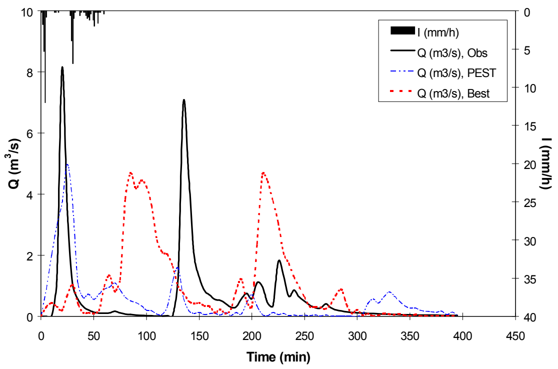
(d) Event CS4

Figure 7.14: Calibration Plots for 'Small' Storm Events of Giralang Catchment

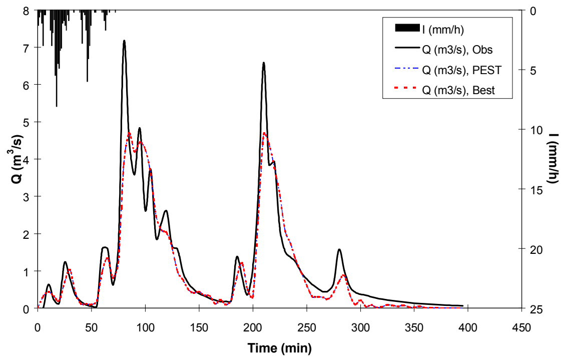
Table 7.8:Calibration Values of Giralang Catchment for 'Large' Storm Events

Event	DS_p (mm)	CN
CL1	6.6	1.8
CL2	7.9	2.3
CL3	2.2	2.0
CL4	2.3	1.5
CL5	2.0	1.0

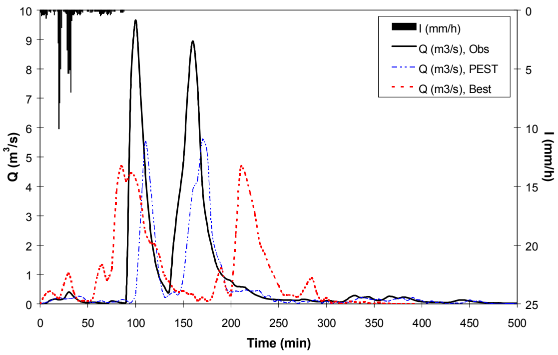
As in Stage 1 optimisation, the best parameter value set for DS_p and CN were obtained from the outer optimisation and found to be the parameter set corresponding to event CL2. The calibration plots of the five 'large' storm events are shown in Figure 7.15. From the selected five events used for calibration, all events provided successful PEST calibration results. All these events had multipeaks, and the calibration showed that the shape and time to peak discharge were satisfactory modelled for three events, while the other two events showed in time-shift in the modelled hydrographs. Therefore, the best parameter sets for the Giralang catchment is shown in Table 7.9. This table also shows DCIA and DS_i obtained from RR plots.



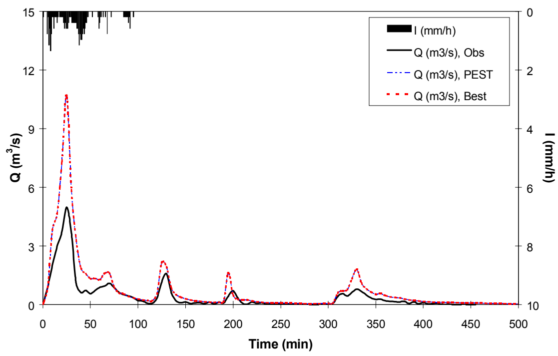
(a) Event CL1



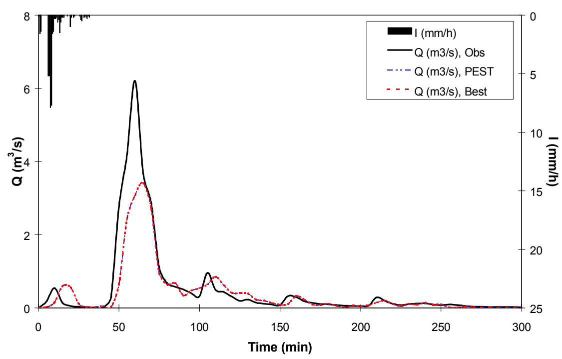
(b) Event CL2



(c) Event CL3



(d) Event CL4



(e) Event CL5

Figure 7.15: Calibration Plots for 'Large' Storm Events of Giralang Catchment

Table 7.9: Parameters from Calibration using Hydrograph Modelling and RR Plots

Parameter	From Calibration	From RR plots
DCIA (%)	19	19
DS_i (mm)	0	0.26
DS_p (mm)	2.2	-
CN	2.0	-

As can be seen from Table 7.9, the DCIA values from RR plots and calibration using hydrograph modelling were the same. However, different values for DS_i were produced by the two methods. The calibration results of DS_p and CN showed considerable variation from one storm event to another as seen from Table 7.8.

7.6 VERIFICATION OF CALIBRATION RESULTS OBTAINED FROM HYDROGRAPH MODELLING

Model verification was done to test the performance of the calibrated model parameters on independent storm events, which were not used in the calibration. Three storm events from each catchment were selected to verify the results. Again, these events did not produce pervious area runoff in Melbourne metropolitan catchments.

The ILSAX model was run for these three events, with the 'best' set obtained from calibration using hydrograph modelling (Sections 7.5.3 and 7.5.4). Giralang catchment using five-day rainfall totals prior to the events were computed and AMC were estimated for each verification event as in calibration.

The three verification events used for catchment BA2A was shown in the RR plot of Figure 7.3. The verification results for these three events are shown in Figures 7.16 and 7.17 as a sample for Melbourne metropolitan catchments and the Giralang catchment. The verification plots of other Melbourne catchments are given in Appendix F. Assessment of the verification plots for all catchments is given below under separate headings.

7.6.1 Verification of Catchment BA2

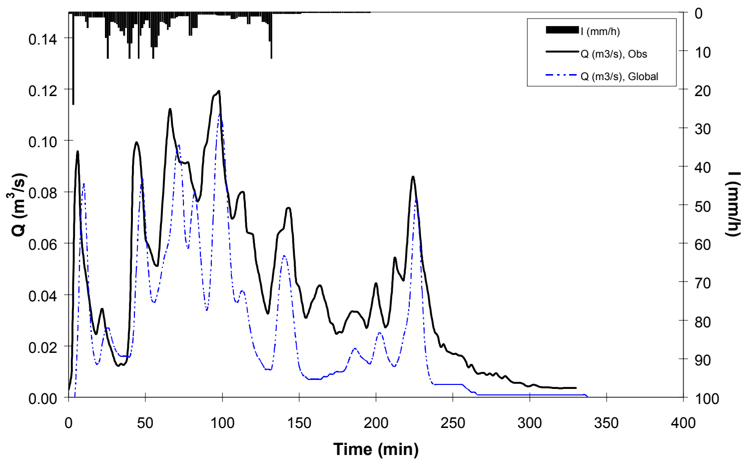
The verification plots for the selected three events are shown in Figure F1 of Appendix F. All these events in Figure F1 had multipeaks, and the verification showed that the shape peak discharge and time to peak discharge were satisfactorily modelled for two events with the 'best' calibrated parameter set, while the other event overestimated the peak.

7.6.2 Verification of Catchment BA2A

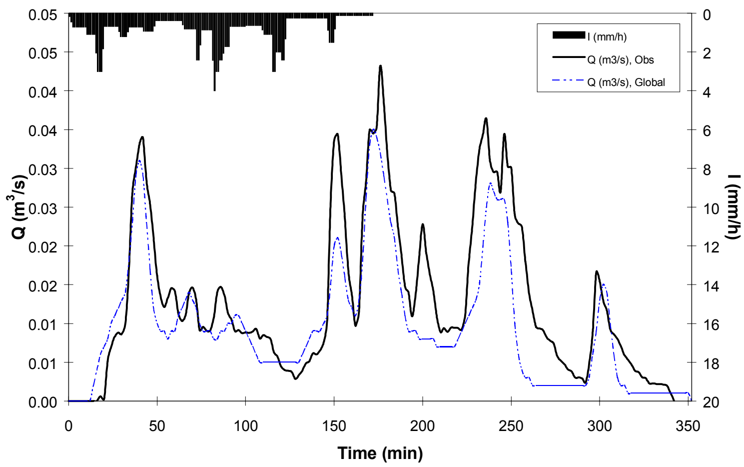
The verification plots for the selected three events are shown in Figure F2 of Appendix F (in Figure 7.16 also). All these events in Figure F2 had multipeaks, and the verification showed that the shape peak discharge and time to peak discharge were satisfactorily modelled for all three events with the 'best' calibrated parameter set.

7.6.3 Verification of Catchment BA3

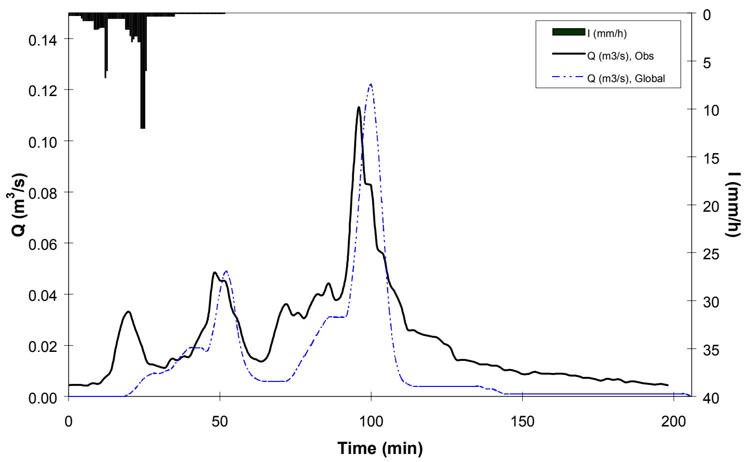
The verification plots for the selected three events are shown in Figure F3 of Appendix F. All these events in Figure F3 had multipeaks, and the verification showed that the shape peak discharge and time to peak discharge were satisfactorily modelled for all three events with the 'best' calibrated parameter set.



(a) Event V1

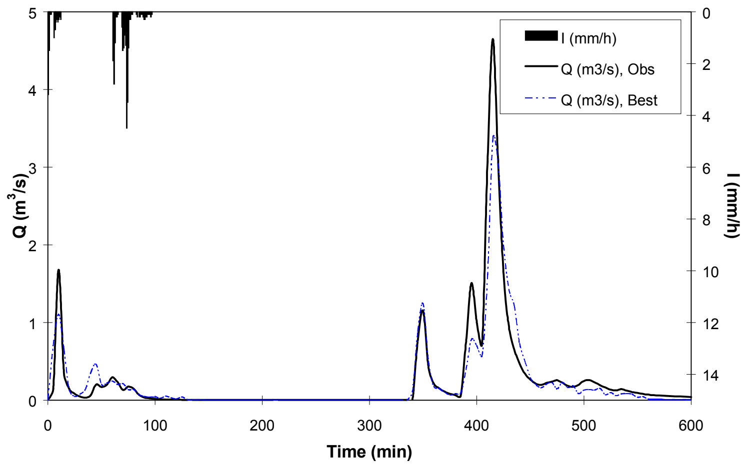


(b) Event V2

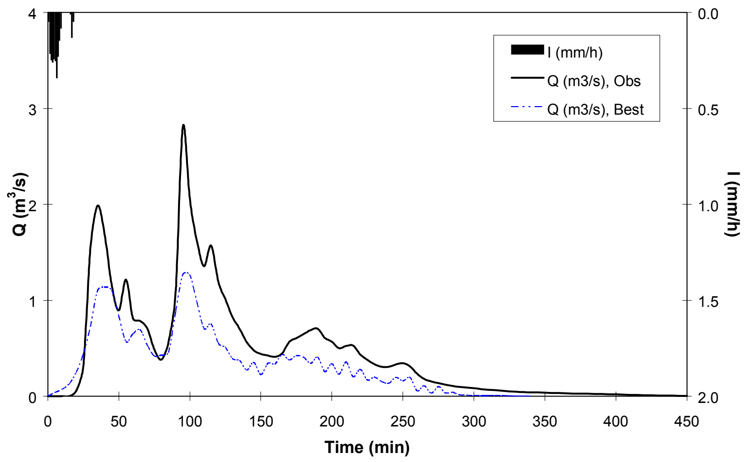


(c) Event V3

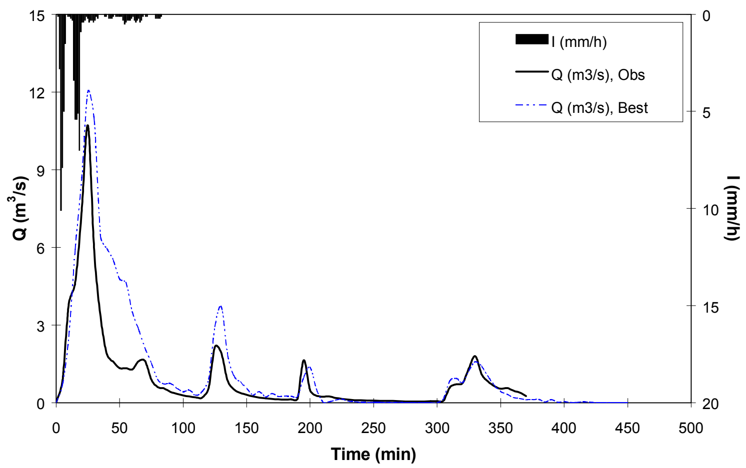
Figure 7.16: Verification Plots of Catchment BA2A



(a) Event V1



(b) Event V2



(c) Event V3

Figure 7.17: Verification Plots for Giralang Catchment

7.6.4 Verification of Catchment BA3A

The verification plots for the selected three events are shown in Figure F4 of Appendix F. All these events in Figure F4 had multipeaks, and the verification showed that the shape peak discharge and time to peak discharge were satisfactorily modelled for all three events with the 'best' calibrated parameter set.

7.6.5 Verification of Catchment BA3B

The verification plots for the selected three events are shown in Figure F5 of Appendix F. All these events in Figure F5 had multipeaks, and the verification showed that the shape peak discharge and time to peak discharge were satisfactorily modelled for all three events with the 'best' calibrated parameter set.

7.6.6 Verification of Catchment BO1A

The verification plots for the selected three events are shown in Figure F6 of Appendix F. All these events in Figure F6 had multipeaks, and the verification showed that the shape peak discharge and time to peak discharge were satisfactorily modelled for all three events with the 'best' calibrated parameter set.

7.6.7 Verification of Catchment BO2A

The verification plots for the selected three events are shown in Figure F7 of Appendix F. All these events in Figure F7 had multipeaks, and the verification showed that the shape peak discharge and time to peak discharge were satisfactorily modelled for two events with the 'best' calibrated parameter set, while the other event underestimated the peak.

7.6.8 Verification of Catchment BR1

The verification plots for the selected three events are shown in Figure F8 of Appendix F. All these events in Figure F8 had multipeaks, and the verification showed that the shape peak discharge and time to peak discharge were satisfactorily modelled for all three events with the 'best' calibrated parameter set.

7.6.9 Verification of Catchment BR1A

The verification plots for the selected three events are shown in Figure F9 of Appendix F. All these events in Figure F9 had multipeaks, and the verification showed that the shape peak discharge and time to peak discharge were satisfactorily modelled for all three events with the 'best' calibrated parameter set.

7.6.10 Verification of Catchment BR2

The verification plots for the selected three events are shown in Figure F10 of Appendix F. All these events in Figure F10 had multipeaks, and the verification showed that the shape peak discharge and time to peak discharge were satisfactorily modelled for two events with the 'best' calibrated parameter set, while the other event overestimated the peak.

7.6.11 Verification of Catchment BR2A

The verification plots for the selected three events are shown in Figure F11 of Appendix F. All these events in Figure F11 had multipeaks, and the verification showed that the shape peak discharge and time to peak discharge were satisfactorily modelled for one events with the 'best' calibrated parameter set, while the other two events underestimated the peak.

7.6.12 Verification of Catchment BR3

The verification plots for the selected three events are shown in Figure F12 of Appendix F. All these events in Figure F12 had multipeaks, and the verification showed that the shape peak discharge and time to peak discharge were satisfactorily modelled for all three events with the 'best' calibrated parameter set.

7.6.13 Verification of Catchment H2

The verification plots for the selected three events are shown in Figure F13 of Appendix F. All these events in Figure F13 had multipeaks, and the verification showed that the shape peak discharge and time to peak discharge were satisfactorily modelled for all three events with the 'best' calibrated parameter set.

7.6.14 Verification of Catchment H2A

The verification plots for the selected three events are shown in Figure F14 of Appendix F. All these events in Figure F14 had multipeaks, and the verification showed that the shape peak discharge and time to peak discharge were satisfactorily modelled for only one event with the 'best' calibrated parameter set, while the other two events underestimated the peak.

7.6.15 Verification of Catchment K1

The verification plots for the selected three events are shown in Figure F15 of Appendix F. All these events in Figure F15 had multipeaks, and the verification showed that the shape peak discharge and time to peak discharge were satisfactorily modelled for all three events with the 'best' calibrated parameter set.

7.6.16 Verification of Catchment K1A

The verification plots for the selected three events are shown in Figure F16 of Appendix F. All these events in Figure F16 had multipeaks, and the verification showed that the shape peak discharge and time to peak discharge were satisfactorily modelled for two events with the 'best' calibrated parameter set, while the other event were not correct.

7.6.17 Verification of Catchment K1B

The verification plots for the selected two events are shown in Figure F17 of Appendix F. All these events in Figure F17 had multipeaks, and the verification showed that the shape peak discharge and time to peak discharge were satisfactorily modelled for both events with the 'best' calibrated parameter set.

7.6.18 Verification of Catchment K2

The verification plots for the selected three events are shown in Figure F18 of Appendix F. All these events in Figure F18 had multipeaks, and the verification showed that the shape peak discharge and time to peak discharge were satisfactorily modelled for all three events with the 'best' calibrated parameter set.

7.6.19 Verification of Catchment K2A

The verification plots for the selected three events are shown in Figure F19 of Appendix F. All these events in Figure F19 had multipeaks, and the verification showed that the shape peak discharge and time to peak discharge were satisfactorily modelled for all three events with the 'best' calibrated parameter set.

7.6.20 Verification of Catchment K3

The verification plots for the selected three events are shown in Figure F20 of Appendix F. All these events in Figure F20 had multipeaks, and the verification showed that the shape peak discharge and time to peak discharge were satisfactorily modelled for all three events with the 'best' calibrated parameter set.

7.6.21 Verification of Catchment K3A

The verification plots for the selected three events are shown in Figure F21 of Appendix F. All these events in Figure F21 had multipeaks, and the verification showed that the shape peak discharge and time to peak discharge were satisfactorily modelled for two events with the 'best' calibrated parameter set, while the other event underestimated the peak.

7.6.22 Verification of Catchment K3B

The verification plots for the selected three events are shown in Figure F22 of Appendix F. All these events in Figure F22 had multipeaks, and the verification showed that the shape peak discharge and time to peak discharge were satisfactorily modelled for all three events with the 'best' calibrated parameter set.

7.6.23 Verification of Catchment GI

A verification plot for the Giralang catchment is shown in Figure F23 of Appendix F (and also in Figure 7.17). All these events in Figure F23 had multipeaks, and the verification showed that the shape peak discharge and time to peak discharge were satisfactorily modelled for two events with the 'best' calibrated parameter set, while the other event slightly underestimated the peak.

7.6.24 Results of Verification

The ‘best’ calibrated parameter sets of DCIA and DS_i (Sections 7.5.3 and 7.5.4) were verified for 22 Melbourne catchments and the Giralang catchment for 3 events from each catchment. In general, a good match was seen between modelled and observed hydrographs for verification storm events. Fourteen catchments showed that the calibrated parameter values were satisfactory for all three events selected for the verification. The other seven catchments showed satisfactory results only for two verification events while the remaining catchment showed satisfactory result for one verification event.

7.7 SUMMARY

To use mathematical models for modelling of urban drainage catchments, it is necessary to estimate the model parameters. The ideal method to determine these parameters is to calibrate the models using observed rainfall and runoff data, if the catchments are gauged. If the catchments are ungauged, one way of estimating these parameters is from regional equations, which relate the model parameters to catchment characteristics. To derive these regional equations, it is also important to estimate the model parameters accurately for gauged catchments with different land-uses.

The ILSAX model was used in this study and the model parameters related to pervious and impervious areas were estimated. The impervious area parameters of DS_i and DCIA were considered, while the pervious area parameters considered were DS_p and CN.

Twenty-two urban drainage catchments from five city councils located in Melbourne metropolitan area were selected for the study. Out of the 22 catchments, 11 were major catchments, while the remaining 11 were subcatchments of these major catchments. One catchment from Canberra (i.e. the Giralang catchment) was also considered in this study to demonstrate the calibration procedure to estimate pervious area model parameters, since the observed events in Melbourne metropolitan catchments were not large enough to produce runoff from pervious areas.

Three methods were employed to estimate the directly connected impervious area percentage (DCIA) of the study catchments. The first method was to estimate the total impervious area percentage (which consisted of both DCIA and supplementary area percentage) from areal photographs and therefore the estimate obtained from this method represented an upper bound for DCIA. The second method was to estimate DCIA from the rainfall-runoff depth plots using 'small' storm events. The third method was to calibrate the ILSAX model using several 'small' storm events through hydrograph modelling and obtain the best parameter set.

DS_i was obtained from the rainfall-runoff depth plots and calibration using 'small' storm events through hydrograph modelling. These two methods produce both DCIA and DS_i. Only the runoff volume is considered in the former method, while the later method considered all hydrograph attributes (i.e. runoff volume, peak discharge, time to peak discharge and hydrograph shape). Therefore, the parameters produced from calibration using hydrograph modelling can be considered more realistic compared to those from the rainfall-runoff depth plots (and also from areal photographs for DCIA).

An optimisation approach called two-stage inner/outer optimisation was developed for the study catchments to calibrate the model parameters using hydrograph modelling. This method produces the 'best' set of model parameters considering all calibration events. The parameter optimisation of each drainage catchment model was carried out in two stages. During the first stage, the model parameters responsible for 'small' storm events (i.e. DCIA and DS_i) were obtained. During the second stage, the additional parameters responsible for 'large' storm events (i.e. DS_p and CN) were obtained. During the second stage, no changes were made to the parameters obtained from the first stage. Each stage consisted of two loops (i.e. inner and outer). The inner loop uses the PEST computer software to optimise corresponding model parameters, a set of model parameters is obtained for each storm event. The outer loop optimises the above sets of model parameters to produce the 'best' set considering all calibration events and hydrograph attributes of runoff volume, peak discharge and time to peak discharge. The outer optimisation was carried out manually.

The two-stage inner/outer optimisation was used for Melbourne metropolitan catchments to optimise impervious area parameters using hydrograph modelling. It was used to estimate both impervious and pervious area parameters for the Giralang catchment. Comparing observed hydrographs with modelled hydrographs using the 'best' set of parameters obtained from the optimisation, it was found that the method produced satisfactory results.

The 'best' set of model parameters obtained from hydrograph modelling was verified using three independent events for each catchment. Generally, all the verification plots were reasonably accurate for all study catchments. The 'best' set of parameters obtained from hydrograph modelling was compared with the parameter values obtained from the other methods (i.e. RR plots and areal photographs). However, the latter estimates were used only as checks for the 'best' parameter set obtained from hydrograph modelling. By comparing the 'best' set of model parameters obtained from hydrograph modelling with those of the other methods (i.e. RR plots and areal photographs), it was found the former set of parameter estimates for the study catchments. These parameter values were used for the regionalisation study discussed in Chapter 8. Based on the results of verification plots and checks with respect to other methods, it can be said that two-stage inner/outer optimisation produced satisfactory results and therefore, it can be recommended to calibrate any urban drainage model.

For Melbourne study catchments, areal photographs were available for ten study catchments. DCIA obtained from hydrograph modelling using the two-stage inner/outer optimisation procedure and rainfall-runoff depth plots were compared with those obtained from the areal photographs, and found to be less. This is to be expected since the areal photographs give the total impervious area which includes DCIA and supplementary area. The difference of the values from both methods for the study catchments was within a 10% limit except for one catchment. The eighteen catchments out of 22 catchments gave less than 10% difference for DCIA computed from the RR plots and the optimisation methods. The 50% of study catchments gave less than 0.25 mm different for DS_i computed from the both methods.

CHAPTER 8

DEVELOPMENT OF REGIONAL RELATIONSHIPS FOR ESTIMATING IMPERVIOUS AREA PARAMETERS

8.1 INTRODUCTION

In order to use urban drainage mathematical models, the model parameters can be estimated through regional equations, as stated in Section 7.1. These regional equations define the model parameters as functions of measurable catchment properties and (in some cases) storm characteristics. In this chapter, the regional equations for impervious area parameters were developed for use in ILSAX. The process of deriving regional equations is termed as parameter regionalisation in this thesis.

In this study, model parameter regionalisation was done using only 16 catchments from the 22 Melbourne metropolitan catchments considered in Chapter 7. These 16 catchments belonged to the land-use category of residential, and therefore the regional relationships were developed for use in residential urban catchments in the Melbourne metropolitan area.

The regional equations developed in the past to estimate the model parameters for urban catchments are reviewed first in this chapter. The selected model parameters used for regionalisation and the catchment characteristics considered are described then, followed by a study of the homogeneous regions for the regionalisation study. Finally, the development of the regional equation is described together with the results.

8.2 REVIEW OF REGIONALISATION TECHNIQUES USED IN URBAN CATCHMENT MODELLING

The first step in model parameter regionalisation is to identify the homogeneous regions in terms of hydrological similarities with respect to model parameters being considered. The next step then is to develop some form of equations to estimate the model parameters using measurable and/or easily obtainable catchment and rainfall parameters. Therefore, it is necessary to estimate the model parameters for gauged catchments (when they can be determined from rainfall/runoff data) within the hydrologically similar group, and then relate these to measurable and/or easily obtainable catchment and rainfall parameters. Successful parameter regionalisation of rainfall-runoff models (both rural and urban) depend on:

- accurate estimation of model parameters for gauged catchments,
- selection of catchment and rainfall characteristics that affect the catchment response to rainfall and the model parameters.
- definition of homogeneous regions,
- degree to which the model parameters are correlated with catchment and rainfall characteristics, and
- correct specification of the regionalisation model for each hydrologically similar region.

Although there are several techniques available for regionalising hydrologic parameters of rural catchments, such as Andrews curves (Dyer *et al.*, 1994), index flood method (Rahman, 1997), and neural network approaches (Cheng and Noguchi, 1996), these methods have not been used for urban catchments. In fact, very few studies were conducted in the past in developing regional equations for urban catchment model parameters. A review is presented below on the regionalisation studies.

Aitken (1975) derived a regional equation for the storage lag parameter of RAFTS (WP Software, 1991) using 11 (Australian) catchments in Melbourne, Canberra, Sydney and Brisbane with a mixture of rural and urban land uses. The lag parameter was expressed in terms of catchment area, slope of the main drainage line and fraction urbanised. In this study, these 11 catchments were considered to be in one hydrologically homogeneous region, although these capital cities are far apart from each other with different climate and hydrological characteristics.

Few studies were found in the literature in regionalising parameters related to impervious areas. Alley and Veehuis (1983) carried out direct measurements of total and directly connected impervious areas in nineteen urban catchments in Denver, USA. They developed an equation for estimating the directly connected impervious area percentage (DCIA), from the total impervious area percentage (TIA) using data from 14 catchments. These 14 catchments were considered to be in one homogeneous region. Their equation is given below.

$$DCIA = 0.15 * TIA^{1.41} \qquad (R^2 = 0.98) \qquad (8.1)$$

According to this equation, DCIA increases with increase in TIA. This seems to be reasonable, since as the impervious area of a catchment increases, most of the impervious area is directly connected to the drainage system.

Generally, the catchment imperviousness increases with increase in population density or housing density (USEPA, 1992). Bedient and Huber (1992) developed a regional equation to estimate the total impervious area percentage (TIA) of large urban catchments in New Jersey, USA as a function of population density (PD, expressed as persons per acre). This regional equation is given in Equation 8.2.

$$TIA = 9.6 * PD^{(0.573-0.017\ln PD)} \qquad (R^2 = 0.83) \qquad (8.2)$$

Equation 8.2 was based on a regression analysis of 567 catchments in New Jersey. Equations 8.1 and 8.2 are valid for the respective cities for which these equations are developed based on data relevant to these cities. Therefore, these equations should be used with caution elsewhere. However, Equation 8.2 was suggested in SWMM (U.S. Environmental Protection Agency, 1992).

Kidd (1978b) used 368 storm events from 14 catchments in U.K. to derive regional equations for urban runoff. All catchments were considered to be in one hydrological homogeneous region. A qualitative appraisal of the relevant hydrological processes was made to identify four catchment variables (i.e. percentage imperviousness, soil type, catchment slope and proportion of roofs) and five storm variables (i.e. rainfall volume, duration, intensity and wetness condition). After studying various combinations of independent variables (i.e. catchment and storm variables), the finally-adopted regression equation was dependent on percentage imperviousness (PIMP), soil index (SOIL) and urban catchment wetness index (UCWI). The parameter SOIL and UCWI are similar to CN and AMC parameters of ILSAX. This regional equation was developed to compute the runoff as a percentage of rainfall (PRO) and given in Equation 8.3.

$$PRO = 0.924\ PIMP + 53.4\ SOIL + 0.065\ UCWI - 33.6 \qquad (R^2 = 0.73) \qquad (8.3)$$

In this equation, UCWI was defined as:

$$UCWI = 125 + 8\ API5 - SMD \qquad (8.4)$$

where API5 is the 5-day antecedent precipitation index, and
` SMD is the soil moisture deficit.

The impervious area depression storage (DS_i) represents the initial loss of the impervious areas of urban catchments in relation to runoff generation. Although DS_i could be significant in simulation of 'small' storms, its effect is negligible for 'large' storms. Using the same catchment data, a regression equation was developed by Kidd (1978b) to the estimate DS_i. The parameter DS_i was estimated from the rainfall-runoff depth plots considering 'small' storm events as described and used in Section 7.2.3.3. This regression equation is given in Equation 8.5.

$$DS_i = -0.109\ S + 0.738 \qquad (R^2 = 0.89) \qquad (8.5)$$

where S is the catchment slope (%)

In Equation 8.5, the catchment slope (S) was taken along a principal representative flow path. Where more than one principal flow path were identified, a weighted average was calculated according to the area of which a flow-path was representative. According to this equation, the maximum DS_i value is 0.74 mm and DS_i equals to zero when catchment slope exceeds 7%.

Kidd (1978a) developed another regression equation for DS_i as a function of catchment slope considering nineteen catchments from four European countries. This means that all these catchments were considered in one hydrologically homogeneous region. As in Kidd (1978b), DS_i was estimated from the rainfall-runoff plots of 'small' storm events. The developed regional equation is given by Equation 8.6. The catchment slope was defined as in Kidd (1978b). There was considerable scatter of data points around the regression line.

$$DS_i = 0.77\ S^{-0.49} \qquad (R^2 = 0.72) \tag{8.6}$$

According to Equations 8.5 and 8.6, DS_i decreases with the increase of catchment slope. This may be physically justifiable, as catchment slope increases, the retention capacity of the catchment surface reduces. This equation is suggested in SWMM (U.S. Environmental Protection Agency, 1992).

Various studies have reported either constant values or a range for DS_i (which is representative of the initial loss. For example, Danish Hydraulic Institute (1988) considered the magnitude of initial loss for impervious areas of urban catchments to be in a range from 0.5 to 1 mm. Bedient and Huber (1992) reported that DS_i varied from 0.5 to 1.5 mm. ILSAX (O'Loughlin, 1993) suggested a value between 0 and 2 mm. From the study of Chapman and Salman (1996), it was found that the average initial loss in roof surface was 0.4 mm. These values or ranges also can be considered as regional equations.

The U.S. Soil Conservation Service (1986) presented the TR-55 simplified procedures to calculate storm runoff volume, peak discharge and storage volume required for detention structures. These procedures were designed to apply for 'small' ungauged urban catchments. Rainfall was converted to runoff volume using a runoff curve number (CN), which depends on soil, plant cover, amount of impervious area, interception and surface storage. The runoff volume was then transformed into a hydrograph using the unit hydrograph theory and routing procedures that depended on runoff travel time through segments of the catchment. Runoff depth versus rainfall depth curves were developed for different CN values to estimate the runoff volume. A set of curve numbers was available for the whole of the United States, which implicitly considered the whole country as one homogeneous region. However, CN depends on many factors as discussed earlier which allows for non-homogeneity.

Two regression models were developed by Driver and Tasker (1988) for estimating storm-runoff pollutant loads and volumes of urban catchments in the United States. In this study, 269 catchments of areas between 30 and 29,000 ha were used. The United States was divided into three homogeneous regions on the basis of mean annual rainfall. For each homogeneous region, two regression models were developed using physical, land-use and climatic characteristics such as total contributing drainage area, impervious area and total rainfall. Total rainfall and total contributing drainage area were the most significant variables in all six regression models. From these models, the most accurate models were those for the more arid Western United States that had less rainfall and the least accurate models were those for the East Coast and Southern United States that had high mean annual rainfall.

The ACT Department of Urban Services (1996) in Canberra (Australia) developed a regional equation for pervious area runoff coefficient, which was defined as in the Statistical Rational method. This equation is given in Equation 8.7.

$$C_p = 0.91 - 3.14\ I^{-0.594} \qquad (8.7)$$

where C_p is the runoff coefficient for pervious grassed surfaces, and
I is the rainfall intensity (mm/h).

Equation 8.7 is applicable for pervious areas in residential developments with densities in the range of 10-15 blocks per hectare.

8.3 CATCHMENT SELECTION FOR REGIONALISATION

In Chapter 7, the model parameters were calibrated for 22 study catchments. From these 22 study catchments, 11 were major catchments and the remaining 11 were their subcatchments. However, these catchments had different land-uses such as residential, industrial, commercial and institutional. From these 11 major catchments, three major catchments were industrial and institutional. They were excluded from the model parameter regionalisation study, since the study was conducted on the residential catchments. The remaining eight major catchments were residential. However, from these eight residential catchments, one catchment had a large reserve, a school and a retirement village. This catchment was also excluded from the model parameter regionalisation, since the characteristics of this catchment were entirely different from the other residential catchments. Therefore, seven residential major catchments from the study catchments and their subcatchments were selected for the regionalisation study.

8.4 SELECTED MODEL PARAMETERS AND OTHER CANDIDATE VARIABLES FOR REGIONALISATION

A detailed study on the sensitivity analysis of the ILSAX model parameters described in Section 6.6 indicated that the runoff volume and peak discharge were sensitive to DS_i for ‘small’ storm events, and to AMC and CN for ‘large’ storm events. These together with DS_p are the hydrological parameters in the ILSAX model. If regional equations are developed for these parameters, then they can be used to estimate these parameters if they cannot be obtained from field measurements or if rainfall-runoff data are not available to calibrate the ILSAX model.

These hydrological parameters (i.e. DS_i, DS_p, CN and AMC) of the ILSAX model are ideal candidates for parameter regionalisation, since they tend to depend on catchment characteristics and are more variable in a catchment compared to routing parameters. However, only DS_i was considered in this regionalisation study, since the pervious area hydrological parameters could not be calibrated from the recorded storm events, as described in Section 7.2.1. In addition to DS_i, the impervious area land-use parameters of DCIA and the supplementary area percentage (SA) were included in the regionalisation study. DCIA is an important parameter in estimating the runoff volume and peak discharge for both ‘small’ and ‘large’ storm events. SA plays a similar role for ‘large’ storm events.

8.4.1 Estimation of Candidates Variables for Regionalisation

(a) DCIA and DS_i

Three different methods were considered in Chapter 7 to estimate DCIA of a catchment. Of these three methods, DCIA and DS_i of the study catchments obtained from model calibration using hydrograph modelling were considered as the model parameters. The results from the areal photographs and rainfall-runoff depth (RR) plots were used to check the accuracy of the above estimates. Similarly, DS_i obtained from hydrograph modelling was considered as the model parameter, although DS_i values were compared with those obtained from the RR plots. Therefore, DCIA and DS_i obtained from model calibration using hydrograph modelling were used in the regionalisation study.

The RR plot and hydrograph modelling used flow data at the catchment outlets in estimating DCIA and DS_i. Therefore, these parameters are relevant to the whole catchment or its subcatchment depending on the flow data used. For the catchments that included one or two subcatchments, it is more accurate to consider DCIA and DS_i for the part of the major catchment, which excludes the subcatchments (i.e. 'remaining' catchment). This is because it is likely that the subcatchments and the 'remaining' catchment will have different catchment properties (DCIA and DS_i), although these properties will be fairly uniform within the subcatchments and the 'remaining' catchment. It should also be noted that different subcatchments were monitored in these major catchments because of their different land use characteristics such as household densities, catchment slope etc., which define DCIA and DS_i.

Therefore, it is necessary to develop a method to estimate DCIA and DS_i for the 'remaining' catchment, when lumped values are obtained for the major catchment from rainfall-runoff data. Consider the hypothetical catchment shown in Figure 8.1, which has two subcatchments (i.e. subcatchments A and B). The 'remaining' catchment is subcatchment C.

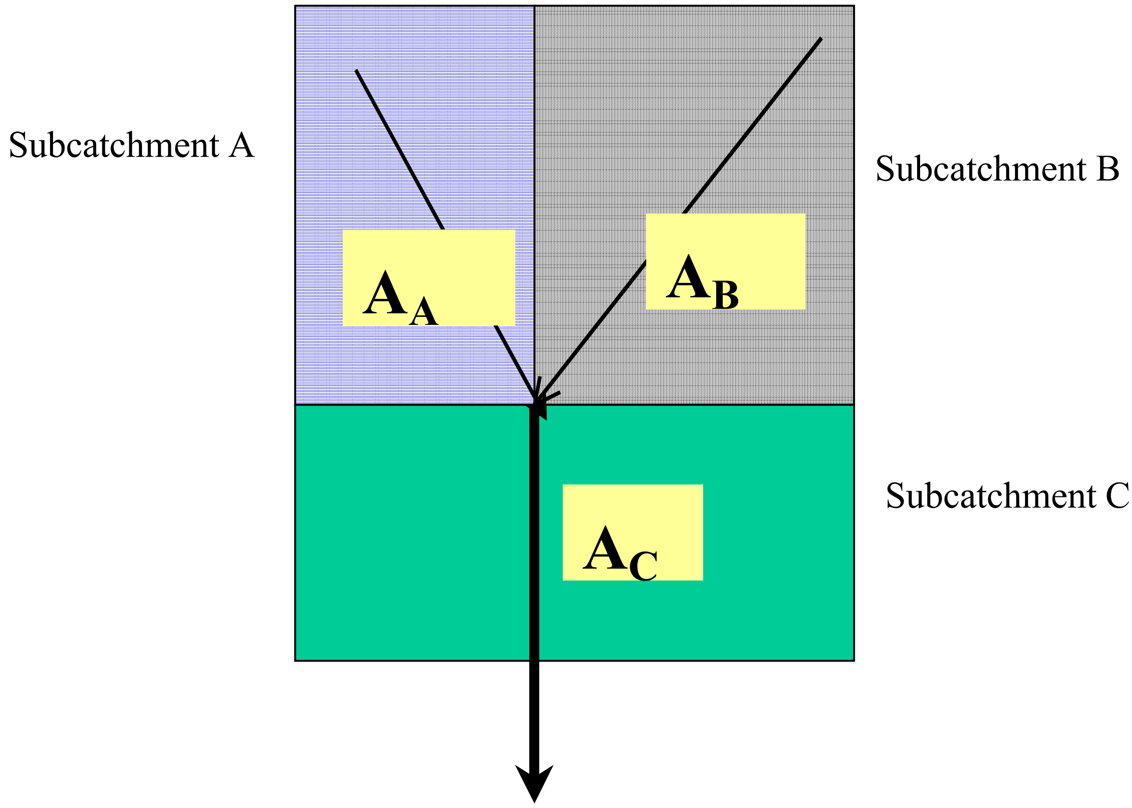


Figure 8.1: Subcatchments and 'Remaining' Catchment in a Major Catchment

If the catchment areas of the major catchment and two subcatchments are A, A_A and A_B respectively, the area of the remaining part of major catchment (A_c) will be ($A-A_A-A_B$). Consider the directly connected impervious area and its depression storage for the major catchment as DCIA and DS_i. The corresponding values for subcatchments A, B and C are then ($DCIA_A$, $DS_{i,\,A}$), ($DCIA_B$, $DS_{i,\,B}$) and ($DCIA_C$, $DS_{i,\,C}$) respectively. Then, $DCIA_C$ and $DS_{i,\,C}$ can be expressed as:

$$DCIA_C = (DCIA.\ A - DCIA_A\ .\ A_A - DCIA_B.\ A_B) / A_C \qquad (8.8)$$

$$DS_{i,\,C} = (DS_i\ .\ A - DS_{i,\,A}\ .\ A_A - DS_{i,\,B}.\ A_B) / A_C \qquad (8.9)$$

Of the seven major catchments considered in this study, five catchments had subcatchments. DCIA and DS_i were then computed for the 'remaining' catchment of these five major catchments using Equations 8.8 and 8.9, corresponding to the 'best' set of parameters obtained from calibration using hydrograph modelling and from RR plots. These estimates together with the estimates for the major catchments and their subcatchments are shown in Table 8.1. Note that in Table 8.1, the major catchments BA2 and BR1 have only one subcatchment, while BA3, K1 and K3 have 2 independent subcatchments. For these catchments, the 'remaining' catchment tag is defined as XXXI, where XXX refers to the major catchment tag. The major catchment BA3 has two subcatchments BA3A and BA3B, and BA3B is a subcatchment of BA3A. Therefore, it is necessary to consider two 'remaining' catchments for this case (i.e. BA3I as the 'remaining' catchment of the major catchment that excludes BA3A and BA3I(2) as the 'remaining' catchment of BA3A that excludes BA3B). Although DCIA and DS_i of 'remaining' catchments were calculated corresponding to both RR plots and hydrograph modelling, the parameters corresponding to hydrograph modelling were used in the regionalisation.

(b) SA

As discussed in Section 4.2, the supplementary areas are the impervious areas that are not directly connected to the drainage system, but runoff from these areas flows over pervious areas before reaching the drainage system. The supplementary areas of the urban catchments cover mainly footpaths, driveways and backyard sheds. The supplementary areas of each study residential catchment were identified and quantified from field visits. SA for the study catchments varied from 2% to 5%. These values are given in Table 8.2. This table also shows DCIA and DS_i values that would be used for regionalisation.

Table 8.1: DCIA and DS_i Values from Different Methods for Study Catchments

Catchment Tag	DCIA (%)			DS_i (mm)		Comments
	Areal Photo	RR Plot	Hydrograph Modelling	RR Plot	Hydrograph Modelling	
BA2		47	48	0.57	0.60	Major
BA2A		35	31	0.37	0.10	Subcatchment
BA2I		52	56	0.66	0.83	'Remaining'
BA3		42	32	0.69	0.40	Major
BA3A		35	24	0.69	0.20	Subcatchment
BA3B		32	26	0.59	0.20	Subcatchment
BA3I*		59	51	0.70	0.88	'Remaining'
BA3I(2)**		37	23	0.74	0.20	'Remaining'
BO1A		50	49	0.14	0.50	Subcatchment
BO2A		40	42	0.50	0.50	Subcatchment
BR1		53	60	0.19	0.10	Major
BR1A		58	57	0.29	0.00	Subcatchment
BR1I		48	63	0.09	0.19	'Remaining'
K1	40	34	30	0.41	0.20	Major
K1A	45	44	32	0.34	0.80	Subcatchment
K1B	54	49	39	1.29	0.10	Subcatchment
K1I		10	21	0.00	0.00	'Remaining'
K3	40	37	32	0.08	0.00	Major
K3A	54	52	45	0.38	0.70	Subcatchment
K3B	45	28	42	0.29	0.50	Subcatchment
K3I		36	19	0.00	0.00	'Remaining'

* BA3I = BA3 - BA3A; BA3A is a subcatchment of BA3

** BA3I(2) = BA3A - BA3B; BA3B is a subcatchment of BA3A

Table 8.2: Selected Parameter Values for Regionalisation

Catchment Tag	DCIA (%)	DS_i (mm)	SA (%)	Comments
BA2A	31	0.10	2.7	Subcatchment
BA2I	56	0.83	4.0	'Remaining'
BA3A	24	0.20	2.3	Subcatchment
BA3B	26	0.20	2.4	Subcatchment
BA3I	51	0.88	3.6	'Remaining'
BA3I(2)	23	0.20	2.2	'Remaining'
BO1A	49	0.50	3.6	Subcatchment
BO2A	42	0.50	3.2	Subcatchment
BR1A	57	0.00	4.1	Subcatchment
BR1I	63	0.19	4.4	'Remaining'
K1A	32	0.80	2.7	Subcatchment
K1B	39	0.10	3.1	Subcatchment
K1I	21	0.00	2.2	'Remaining'
K3A	45	0.70	3.3	Subcatchment
K3B	42	0.50	3.2	Subcatchment
K3I	19	0.00	2.0	'Remaining'

8.5 SELECTED CATCHMENT PROPERTIES FOR REGIONALISATION

The effect of hydrological cycle is inherently spatially varied and depends on such factors as the shape, size, slope, drainage network, surface cover, soil characteristics and land use patterns of the drainage basin (Goonetilleke and Jenkins, 1997). These factors affect the runoff response to rainfall in both urban and rural catchments, and should be considered in regionalisation studies of the model parameters. As stated in Driver and Tasker (1988), the commonly used physical and land-use characteristics in regional equations are total catchment area, impervious area, land-use type (i.e. industrial, commercial, residential and non-urban land-uses) and population density (or housing density). In addition, the climatic characteristics have been used in regionalisation of pervious area parameters (Driver and Tasker, 1988). The common climatic characteristics were total storm rainfall, duration of storm, 24-hour precipitation intensity that has a 2-year recurrence interval and mean annual rainfall. Since only the impervious area parameters were considered in this regionalisation study, the climatic characteristics were not important and therefore they were not included in this study. As stated in Section 8.3, the regionalisation study was restricted to residential land-use catchments.

It is reasonable to assume that a suburb closer to the Central Business District (CBD) of a city is more urbanised compared to a suburb far from the city, and hence a higher DCIA for suburbs closer to the city. Therefore, the distance from Melbourne CBD (MCBD) was considered as a parameter in the regionalisation study. As the urban catchments progressively urbanise, the household density increases, which increases DCIA, and therefore the household density was also considered as a parameter in the regionalisation study.

The retention capacity of an urban catchment depends on land-use type and its slope. Kidd (1978a,b) found that the impervious area depression storage (DS_i) depended on catchment slope and therefore the catchment slope was included as an independent parameter in the regionalisation of DS_i.

8.5.l Estimation of Catchment Properties

The catchment area had already been measured for modelling of the study catchments. The household density was estimated by counting the total number of households within the study catchments through drainage plans and verified by field visits. The average slope of the catchments was estimated in three methods. They were the arithmetic mean, area weighted mean and geometric mean. For each pit, the catchment slope was first computed considering the flow path length and the ground elevation difference corresponding to the flow path length. Then the average catchment slopes were computed using subcatchment slopes based on above three methods. Distance to the catchment from MCBD was measured from topographical maps. These catchment properties for the study catchments used for regionalisation are given in Table 8.3. As can be seen, the average catchment slope of a catchment estimated from the three methods are similar.

8.6 IDENTIFICATION OF HOMOGENEOUS REGIONS

Several attempts were made in defining homogeneous regions for the study catchments. The first attempt was based on the geographical boundaries. In this case, the geographical boundary was considered as the city council boundary. To study the homogeneous regions based on council boundaries, DCIA, SA and DS_i values of the catchments were plotted as shown in Figures 8.2-8.4. According to these three figures, there is no pattern for DCIA, SA and DS_i. Even if homogeneous regions can be selected in this way, the regional equations cannot be developed through regression because of the small number of data points under each council. Therefore, the selection of the homogeneous regions based on council boundaries was discarded from further analysis.

Then, all study catchments were considered to form one homogeneous region. This approach is conceptually satisfactory since all catchments were within 30 km radius from MCBD and they have similar land-use characteristics such as impervious area details.

8.7 DEVELOPMENT OF REGIONAL EQUATIONS

This section discusses the development of regional equations for DCIA, SA and DS_i. First the catchment characteristics that influence these parameters were identified. Then, regression analyses were conducted by relating the influential catchment characteristics to these parameters. A split sample procedure was used in developing the regional equations.

Table 8.3: Properties of Selected Residential Catchments for Regionalisation

Council Code	Council Name	Catchment Code	Catchment Area (ha)	Residential Density (houses/ha)	Distance From MCBD (km)	Average Catchment Slope (%)		
						Arithmetic	Weighted	Geometric
I	Banyule	BA2I	30.61	10.8	12.1	3.66	3.66	3.67
		BA2A	13.92	8.3	12.5	4.62	5.46	5.23
		BA3I	12.60	12.0	18.9	5.22	4.97	4.42
		BA3I (2)	19.47	7.2	18.5	3.12	4.53	2.31
		BA3A	29.93	6.6	18.2	4.06	4.75	3.68
		BA3B	10.46	8.4	18.7	5.00	4.97	5.05
II	Borrondara	BO1A	3.12	8.0	12.5	4.90	4.82	4.75
		BO2A	5.38	9.1	7.5	2.93	2.81	2.93
III	Brimbank	BR1I	3.99	11.4	19.7	0.44	0.33	0.40
		BR1A	3.73	14.7	20.2	0.50	0.55	0.50
IV	Knox	K1I	7.20	10.2	28.9	5.63	5.45	6.59
		K1A	9.93	5.4	28.7	5.16	5.79	5.02
		K1B	4.88	7.8	29.2	3.04	2.38	2.91
		K3I	19.09	4.1	26.6	5.25	6.03	4.95
		K3A	8.73	9.6	26.2	3.58	3.64	1.84
		K3B	13.48	4.8	27.1	3.17	3.74	3.58

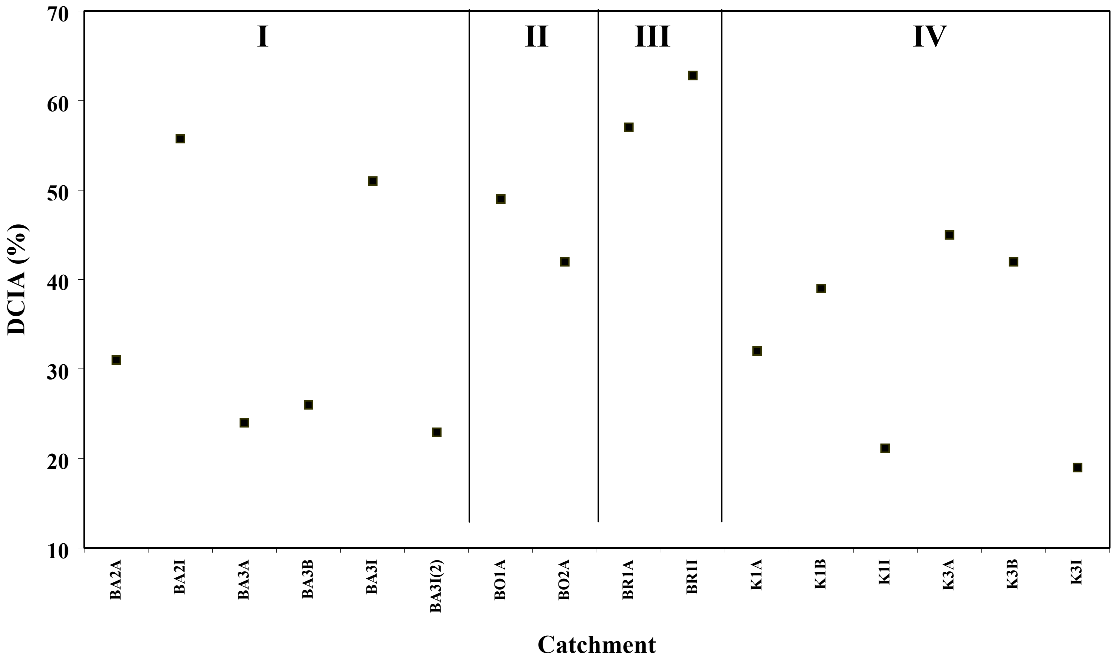


Figure 8.2: DCIA on Council Basis

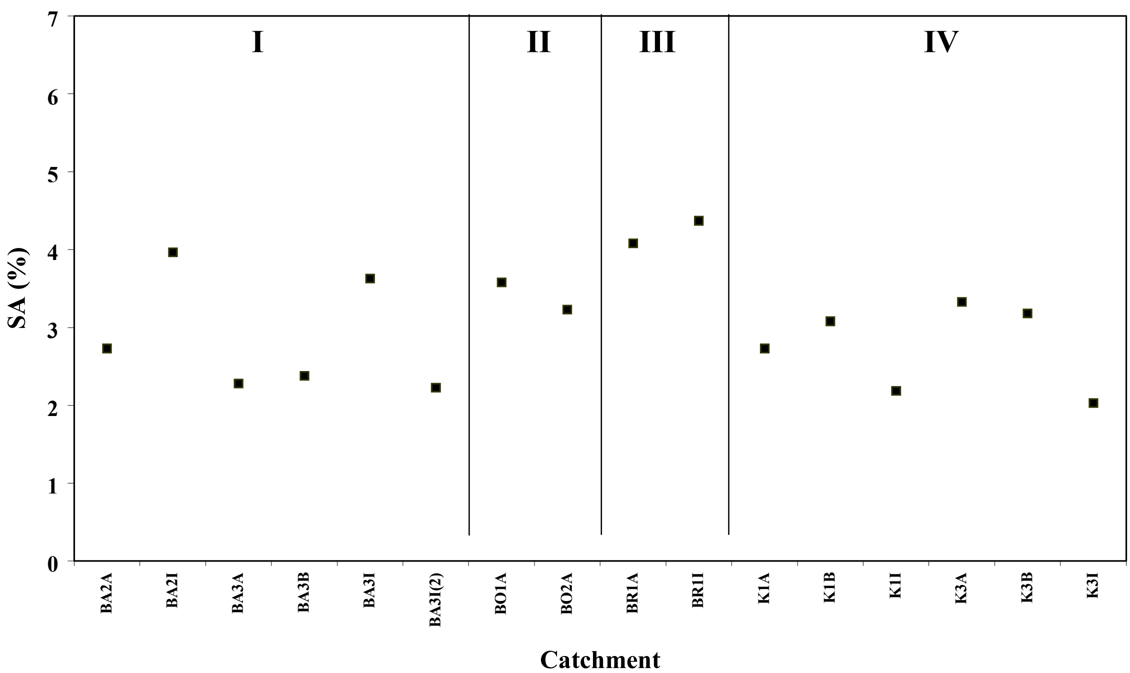


Figure 8.3: SA on Council Basis

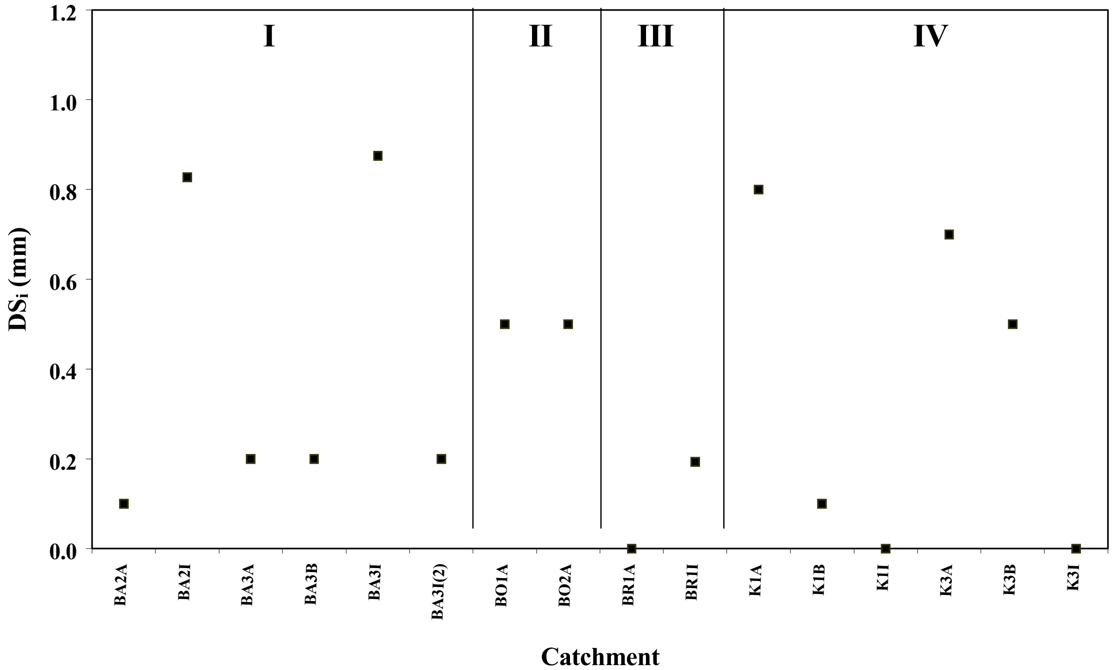


Figure 8.4: DS_i on Council Basis

8.7.1 Split Sample Procedure

If sufficient data are available, it is desirable to use a split sample procedure. In the split sample procedure, the available data are partitioned into two groups. The first group is used to derive the regional equations (known as the calibration in this chapter) and then these equations are tested for its ability to reproduce the data of the second group (known as the verification in this chapter). If the test results lie within acceptable limits, the data and the form of the equation are accepted, and redevelopment of the equation carried out using all available data. If the test results lie outside acceptable limits, the basic assumptions and data used in regional equations are thoroughly checked. Then, only the good data are used for the development of the regional equations, but the confidence in the results is then reduced because of the small data set and lack of verification. The split sample procedure as described above was also suggested by Maidment (1993).

The study catchments were selected for split sample procedure as given in Table 8.4. In general terms, one catchment (i.e. either a subcatchment or the 'remaining' catchment) from each major catchment was considered for verification, and the others were used for calibration. From the study catchments, 10 catchments were used to derive the regional equations and six catchments were used to test the derived equation.

Table 8.4: Catchment Selection for Split Sampling Procedure

Calibration Catchments	Verification Catchments
BA2A	BA2I
BA3A, BA3I, BA3I(2)	BA3B
	BO1A
BO2A	-
BR1A	BR1I
K1A, K1I	K1B
K3A, K3I	K3B

8.7.2 Regionalisation of DCIA

8.7.2.1 Identification of influential catchment characteristics

(a) <u>Relationship between DCIA and total catchment area</u>

DCIA may change with the size of catchment. In large catchments, there may be more open space areas such as reserves, playgrounds and parks. Therefore, DCIA may be less in larger urban catchments.

DCIA of the study residential catchments were plotted against the total catchment area, as shown in Figure 8.5. As can be seen from this figure, a fair amount of scatters exists, although there is a general tendency that DCIA decreases with increase in the total catchment area. DCIA can be different for catchments of the same size, especially for small catchments. Figure 8.5 does not clearly support the hypothesis that larger catchment have smaller DCIA and vice versa, because of the scatter of data points. Therefore, it was not included as a variable in deriving the regional relationships for DCIA. Furthermore, the catchment area is already included in DCIA, since DCIA is expressed as a function of the total catchment area.

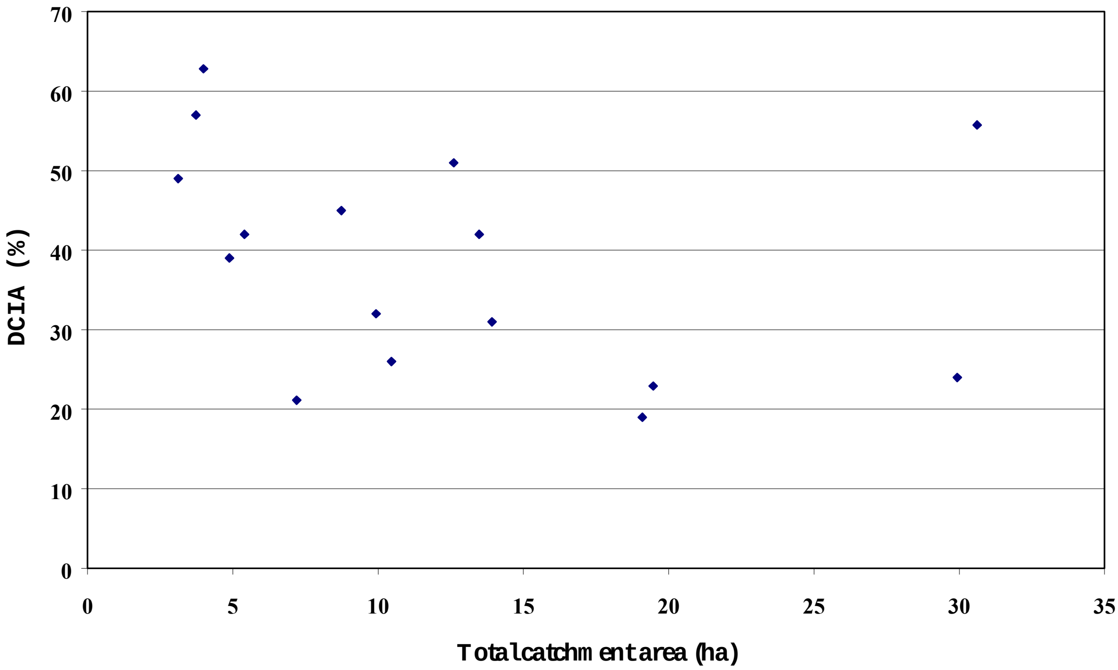


Figure 8.5: DCIA versus Total Area for Study Catchments

(b) ***<u>Relationship between DCIA and household density</u>***

DCIA was plotted against the household density for the study catchments, as shown in Figure 8.6. Although there is some scatter in the plot, there is a clear trend in the plot. The increase of household density means that there are more impervious areas in catchment and most of these impervious areas are connected to the drainage system. The household density was included in deriving the regional equations for DCIA.

(c) ***<u>Relationship between DCIA and distance from MCBD</u>***

DCIA of the study catchments were plotted against the distances to the centroid of the catchment from MCBD, as shown in Figure 8.7. No trend can be seen from this figure between DCIA and the distance from MCBD. Therefore, the distance from MCBD was not included in deriving regional expressions for DCIA.

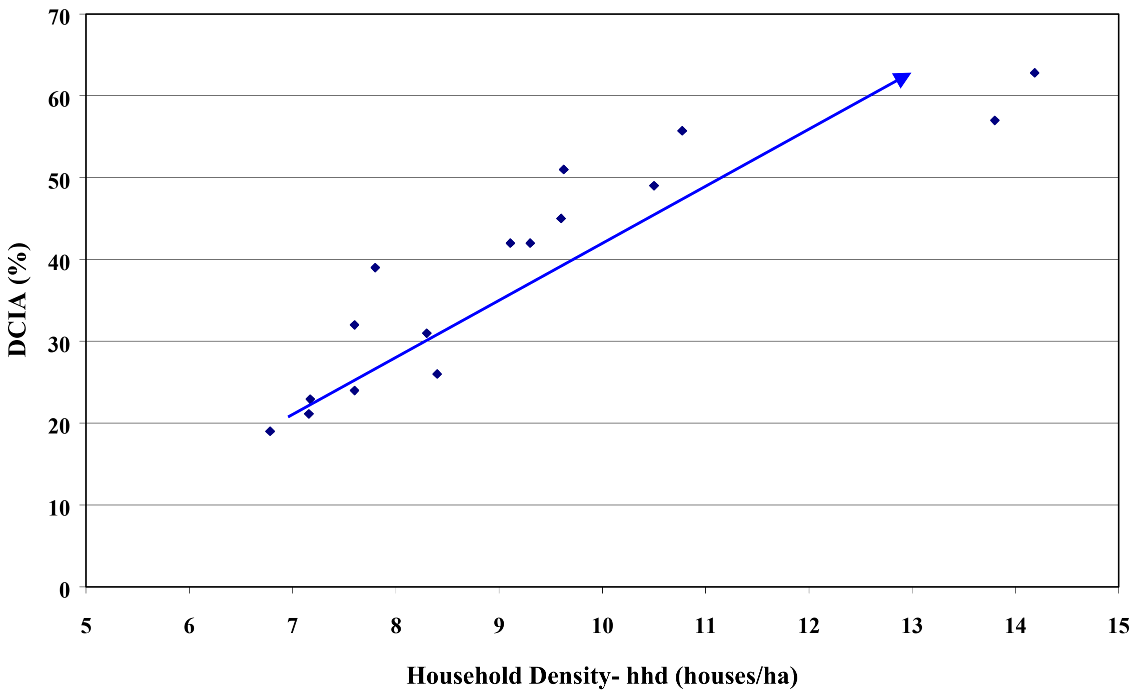


Figure 8.6: DCIA versus Household Density for Study Catchments

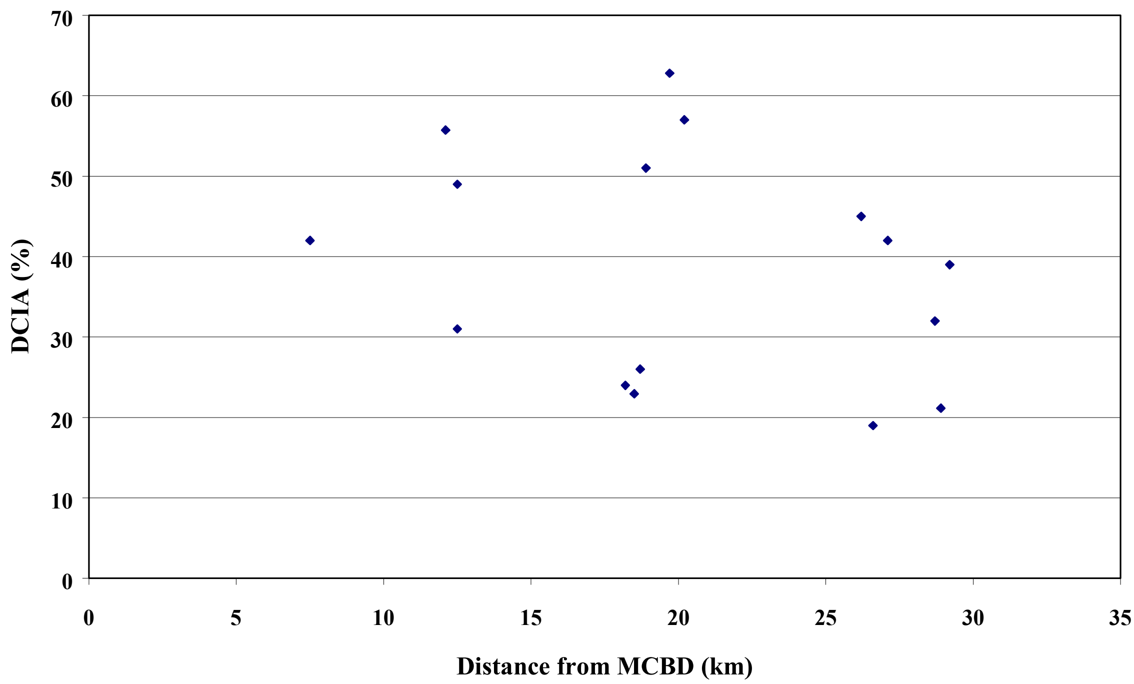


Figure 8.7: DCIA versus Distance from Melbourne CBD for Study Catchments

(d) Selected parameters

From the results of (a)-(c) above, only the household density is important in deriving regional equations for DCIA. Therefore, this eliminates the need to perform multiple regression analyses for deriving regional equations.

8.7.2.2 Development of regression equation for DCIA

The split sample procedure described in Section 8.7.1 was used to derive the regional equation for DCIA.

(i) Calibration

Several linear and non-linear functions were considered in the regression analysis for calibration catchments having independent and dependent variables as household density (hhd) and DCIA respectively. For non-linear functions, the functions of logarithmic, polynomial (of different orders), power and exponential were considered. The best fits were obtained with respect to the polynomial equations. When the order of the polynomial is increased beyond 2, the results did not improve significantly. The best fit is shown in Figure 8.8 and given in Equation 8.10, where hhd is the household density.

$$\text{DCIA} = -1.09\ \text{hhd}^2 + 28.13\ \text{hhd} - 123.20 \qquad (R^2 = 0.95) \qquad (8.10)$$

(ii) Verification

DCIA was estimated for verification catchments using Equation 8.10 and compared them with the original DCIA values of these catchments (Table 8.4). This comparison is shown in Figure 8.9.

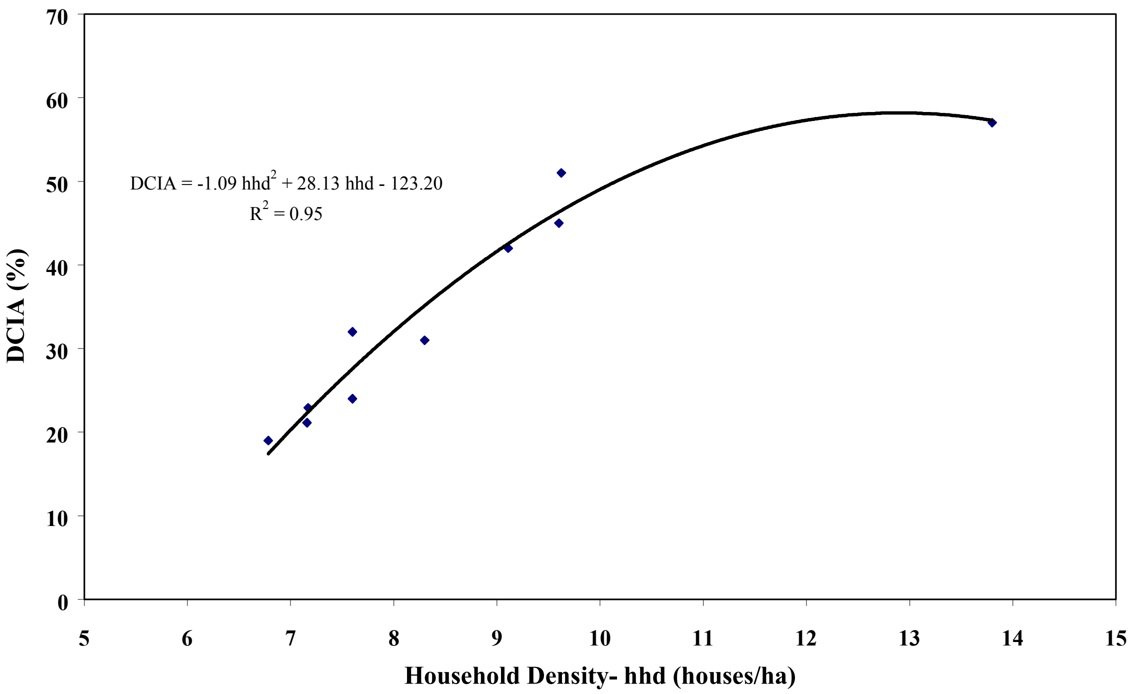


Figure 8.8: DCIA versus Household Density for Calibration Catchments

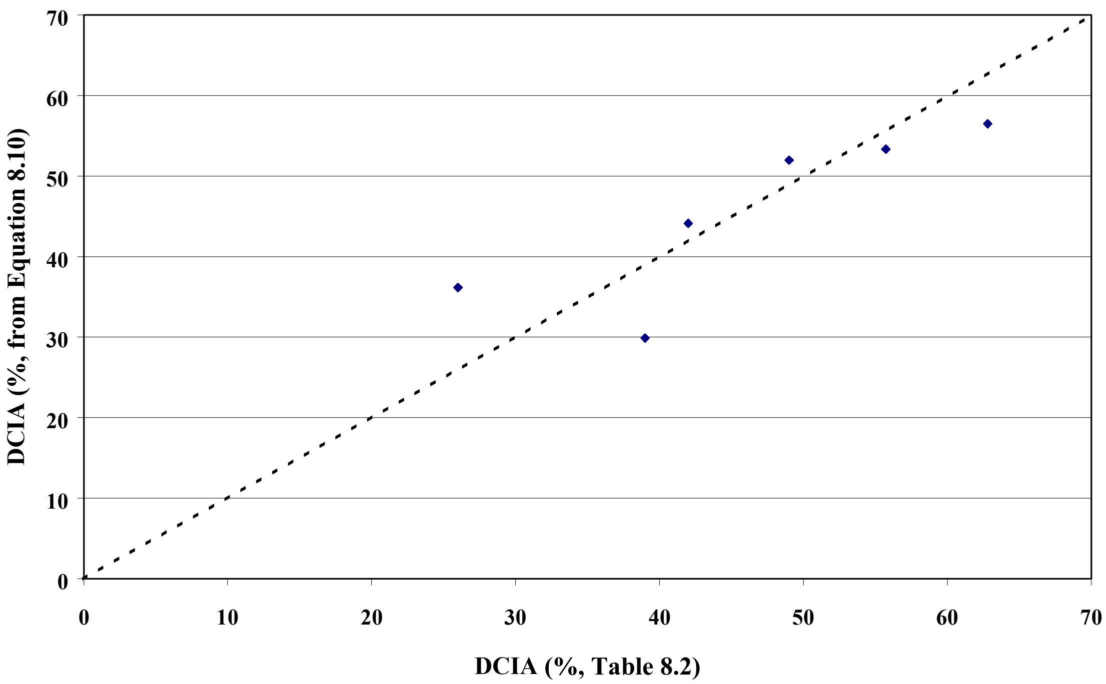


Figure 8.9: Verification of Regional Equation of DCIA versus Household Density

(iii) __Final Regional equation__

Results are reasonably accurate for the verification catchments (Figure 8.9) and therefore, all catchments were used for fine-tuning the derived regional equation. This is shown in Figure 8.10 and the regional equation is given by Equation 8.11. Although this equation reduces the correlation coefficient slightly (R^2 changes from 0.95 to 0.90), the validity of this equation is more since more data points were used in this derivation compared to the equation derived just from the calibration catchments.

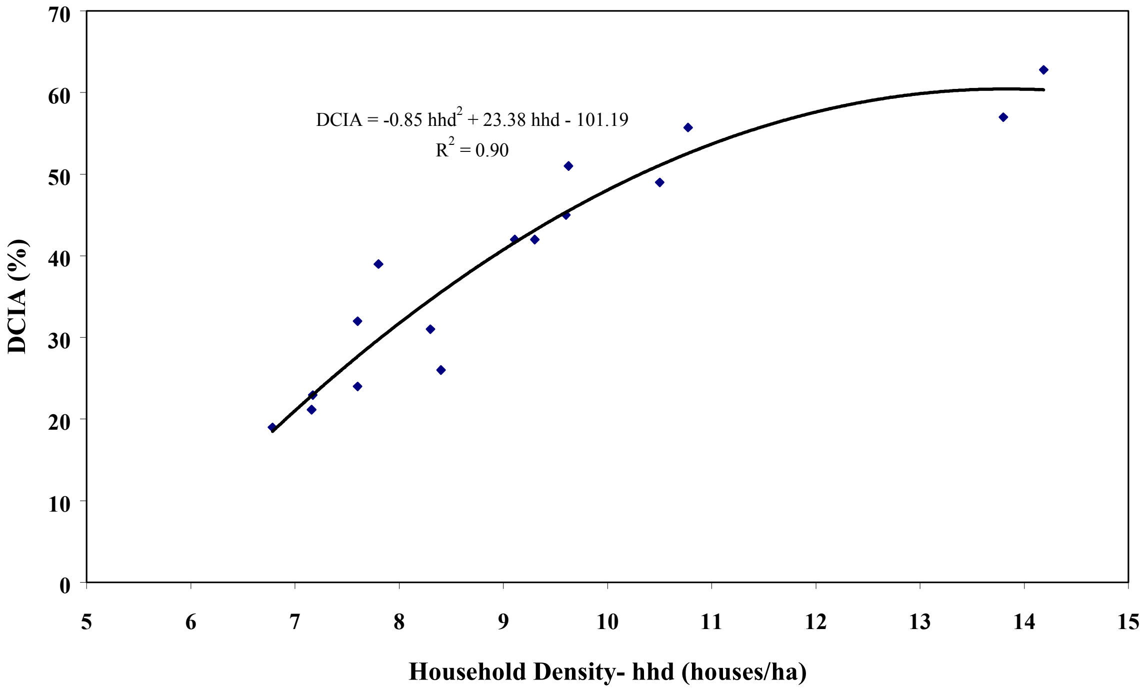


Figure 8.10: DCIA versus Household Density for Study Catchments

Therefore, the final regional equation is:

$$DCIA = -0.85\ hhd^2 + 23.38\ hhd - 101.19 \qquad (R^2=0.90) \qquad (8.11)$$

8.7.3 Regional Equation for Supplementary Area Percentage (SA)

Based on the analysis conducted on DCIA with respect to independent variables, it is reasonable to assume that SA behaves similar to DCIA and correlated only with the household density.

(i) ***Calibration***

SA was plotted against household density as shown in Figure 8.11 for the calibration catchments. Data points of SA for the study catchments were fitted with different type of linear and nonlinear functions. The best-fitted function was nonlinear and given in Equation 8.12.

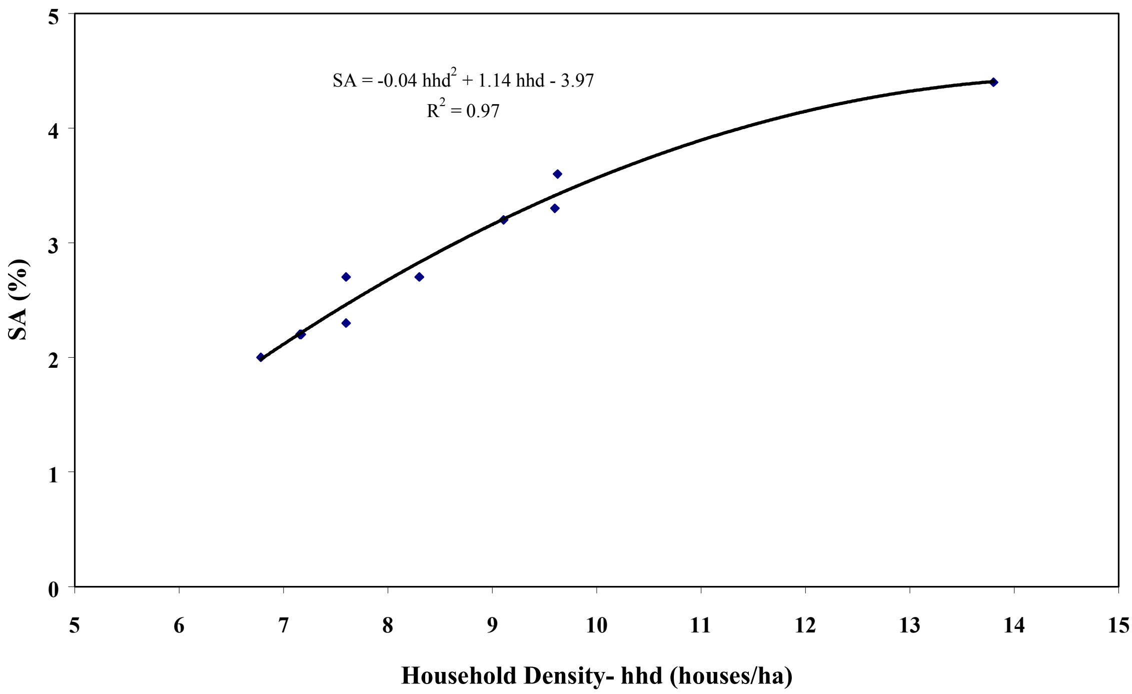


Figure 8.11: SA versus Household Density for Calibration Catchments

$$SA = -0.04\ hhd^2 + 1.14\ hhd - 3.97 \qquad (R^2 = 0.97) \qquad (8.12)$$

(ii) ***Verification***

The above derived equation (Equation 8.12) was used to estimate SA of the verification catchments. The estimated values from Equation 8.12 were compared with the original SA estimates of the verification catchments (Table 8.4). The comparison results are shown in Figure 8.12. This figure suggests that derived equation is reasonably valid for the verification catchments.

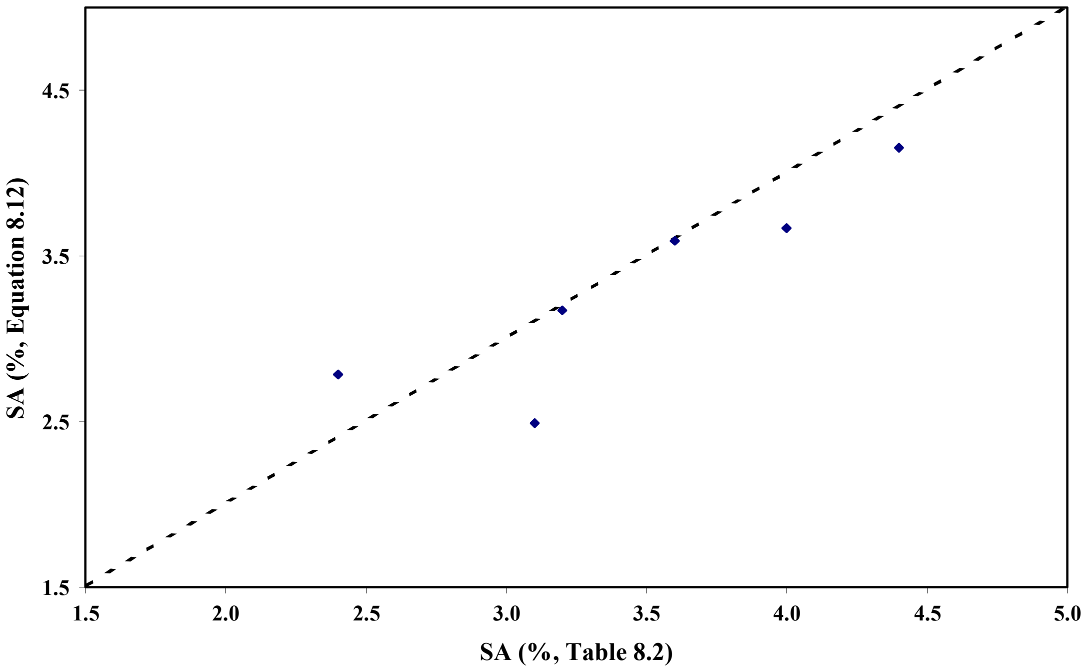


Figure 8.12: Verification of Regional Equation of SA versus Household Density

(iii) Final regional equation

Since the verification catchments produced reasonably successful results, all catchments were used to derive the final regional equation. This equation is shown in Figure 8.13 and given by Equation 8.13.

$$SA = -0.04\ hhd^2 + 1.13\ hhd - 3.79 \qquad (R^2 = 0.91) \qquad (8.13)$$

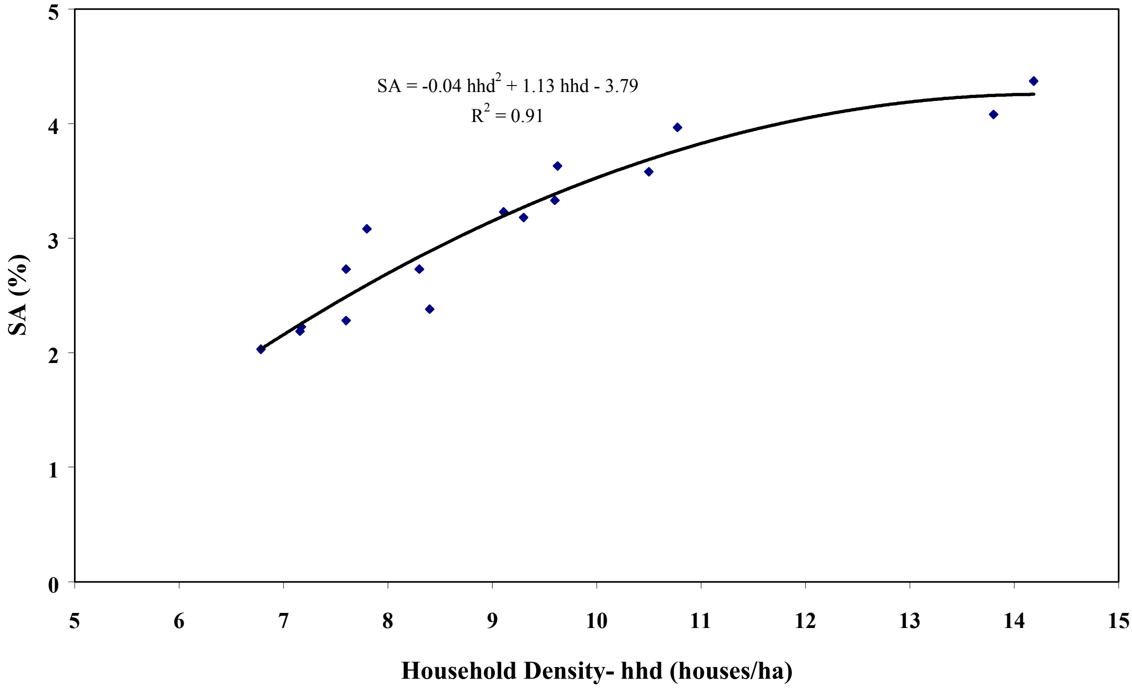


Figure 8.13: SA versus hhd for Study Catchments

8.7.4 Regionalisation of DS_i

8.7.4.1 Identification of influential catchment characteristics

Based on past research such as those of Kidd (1978a,b), the regional equations of DS_i had included only the catchment slope, since the catchment slope had some effect on the retention capacity of runoff, which is directly related to DS_i. However, as stated in Section 8.2, there are other studies (Danish Hydraulic Institute, 1988; Bedient and Huber, 1992; O'Loughlin, 1993) that recommended constant values or a range for DS_i.

In this section, an attempt was made to correlate DS_i with the catchment slope. In addition to the catchment slope, DCIA was also considered.

(a) ***Relationship between DS_i and catchment slope***

Since the study catchments consist of a number of subcatchments draining to their own pits, three methods (i.e. arithmetic mean, area weighted mean and geometric mean) were used to compute the catchment slope (Section 8.5.1).

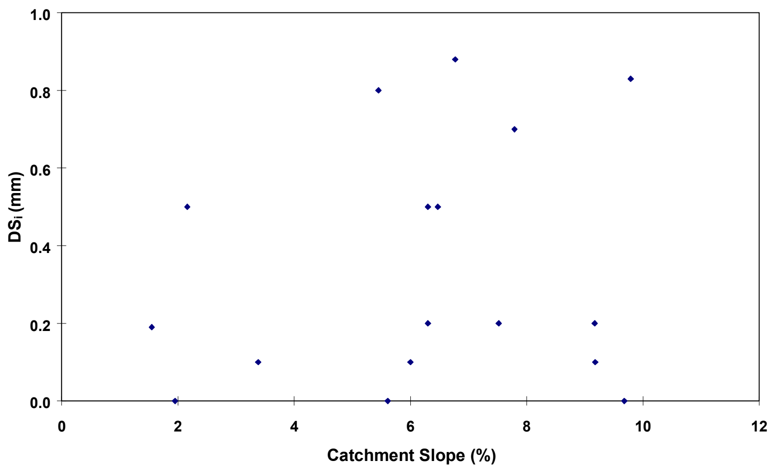
DS_i versus different definitions of catchment slopes are shown in Figure 8.14. As can be seen from this figure, there is no correlation at all between DS_i and the catchment slope. This is in contrast to the findings of Kidd (1978a,b). However, the values of DS_i are between 0 and 1 mm. This could be one of the reasons that the fixed values within a range had been recommended by Danish Hydraulic Institute (1988), Bedient and Huber (1992) and O'Loughlin (1993).

(b) ***Relationship between DS_i and DCIA***

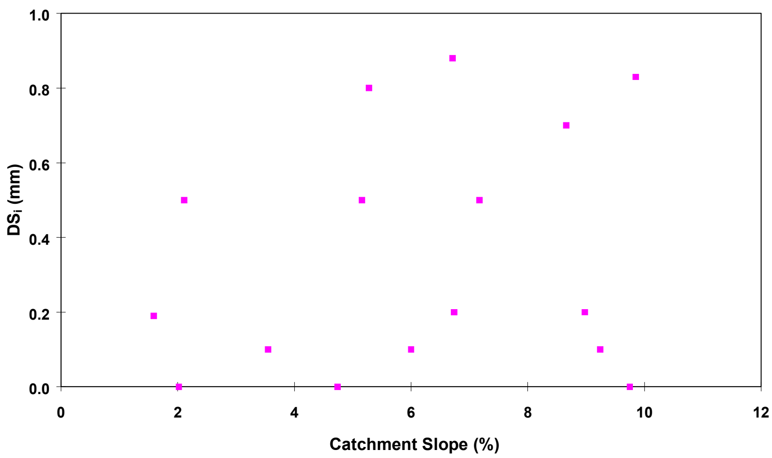
Figure 8.15 shows DS_i versus DCIA for study catchments. This figure suggests that there is no correlation between these two parameters.

8.7.4.2 Regionalisation equation for DS_i

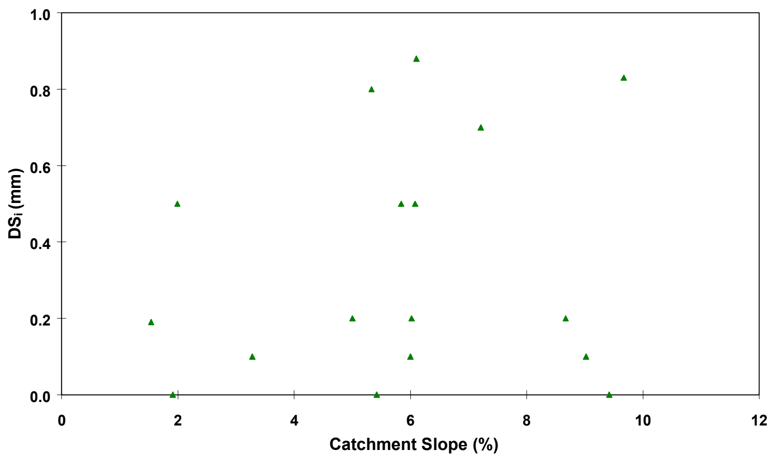
Both catchment slope and DCIA did not show any correlation with DS_i. However, DS_i of the study catchments were within the range of 0-1 mm. This range for DS_i is comparable with the results of previous studies (Danish Hydraulic Institute, 1988; Bedient and Huber, 1992; O'Loughlin, 1993). Therefore, it is recommended that DS_i between 0 and 1 mm can be used for the ungauged urban residential catchments in the Melbourne metropolitan area, where there are no rainfall/runoff data available for calibrating DS_i. Since a range (0 - 1 mm) was recommended for DS_i, the sensitivity of DS_i was revisited to see the effect on various DS_i values within the range on the hydrograph attributes (i.e. runoff volume, peak discharge and time to peak discharge). This was also extended to DCIA. This is discussed in Section 8.7.5.



(i) DS_i Versus Arithmetic Average Catchment Slope



(ii) DS_i Versus Area Weighted Average Catchment Slope



(iii) DS_i Versus Area Geometric Average Catchment Slope

Figure 8.14: DS_i versus Average Catchment Slopes for Study Catchments

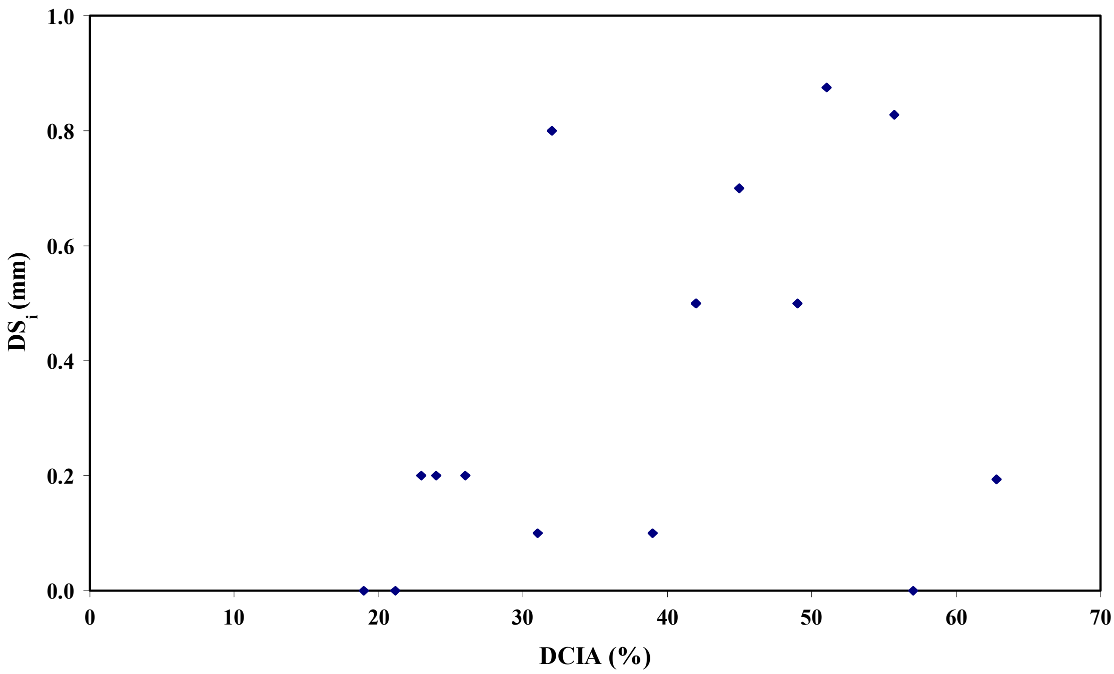


Figure 8.15: DS_i Versus DCIA for Study Catchments

8.7.5 Further Sensitivity Studies of DS_i and DCIA

The sensitivity study was conducted using two design storms (i.e. 1 and 100 year ARIs) and two catchments used in Section 6.4. These two design storms represented a 'small' and 'large' storm event, and the two catchments represented a 'small' and a 'large' urban catchment with typical urban densities, pits and pipes. Three values for DS_i (i.e. 0.0, 0.5 and 1.0 mm) and three values for DCIA (i.e. 20%, 40% and 60%) were used.

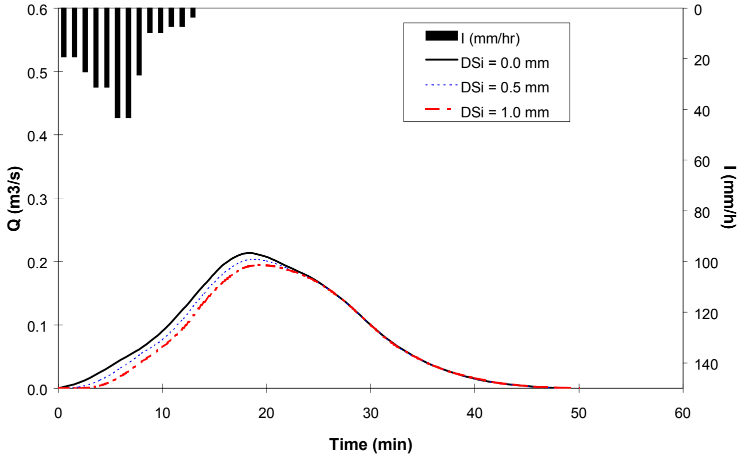
For each DCIA value, the outlet hydrographs were computed using the ILSAX model corresponding to three DS_i values. Table 8.5 shows the details of the hydrograph attributes for the two catchments corresponding to 1-year ARI storm event. Figures 8.16 and 8.17 show the outlet hydrographs for different values of DS_i and DCIA for the 1-year ARI storm event. From Figures 8.16 and 8.17, and Table 8.5, it can be seen that outlet hydrographs change slightly with different DS_i values for a given DCIA. The significant difference is that as DS_i increases, the starting time of the hydrograph is delayed as a result of higher initial losses. There is also a difference in runoff volume, although peak and shape are not that different with respect to different DS_i values. However, the variation is very high with different DCIA values.

Table 8.5: Hydrograph Attributes for 1-year ARI Storm Event

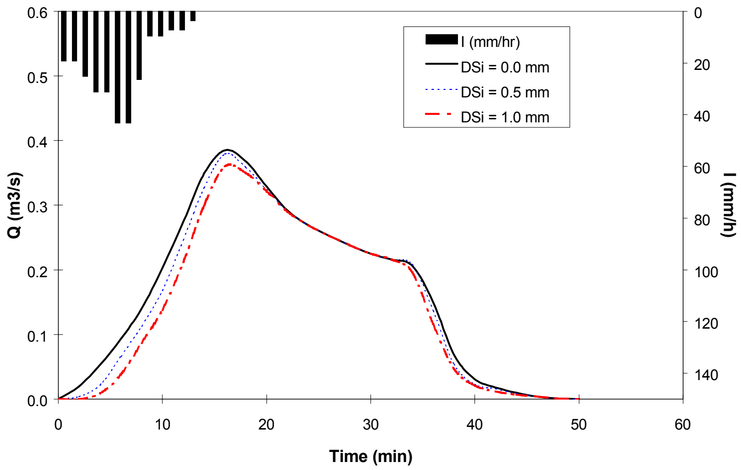
Attributes	DS_i (mm)	Altona Medows Catchment			Therry Street Catchment		
		DCIA			DCIA		
		20%	40%	60%	20%	40%	60%
Volume (m^3)	0.0	253	504	776	1035	2081	3138
Peak (m^3/s)		0.213	0.385	0.503	1.089	1.997	2.510
Time to peak (min)		20	18	18	20	18	18
Volume (m^3)	0.5	239	474	730	970	1949	2934
Peak (m^3/s)		0.202	0.379	0.503	1.081	1.984	2.479
Time to peak (min)		20	18	18	20	20	18
Volume (m^3)	1.0	226	445	695	917	1833	2755
Peak (m^3/s)		0.194	0.360	0.491	1.078	1.998	2.494
Time to peak (min)		22	18	18	18	18	18

The results for the 100-year ARI are given in Table 8.6 and Figures 8.18 and 8.19 for both catchments. The table and plots suggest that DS_i does not make any significant difference to outlet hydrographs for this event. However, DCIA makes a significant difference.

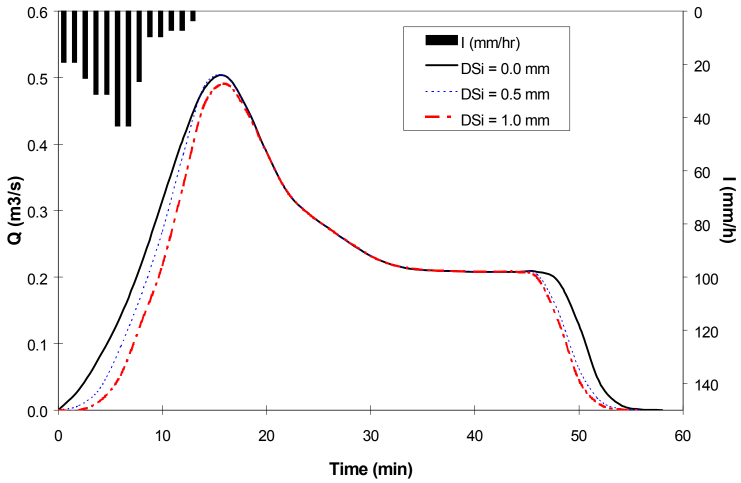
As can be seen from the results of this sensitivity study, DCIA is very sensitive for both 'small' and 'large' storm events compared to DS_i. The effect of DS_i on 'large' storm events is insignificant, while the peak discharge is not affected by DS_i for 'small' events. In urban drainage studies the peak discharge is more important in most cases. Therefore, the range between 0-1 mm can be used for DS_i for ungauged catchments.



(a) For DCIA = 20%

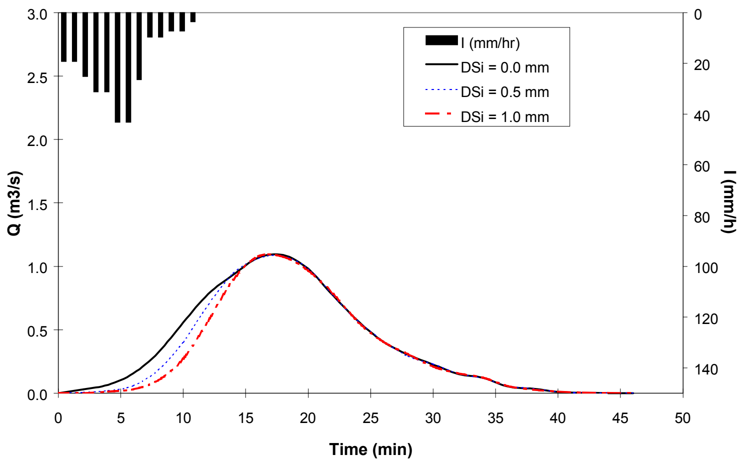


(b) For DCIA = 40%

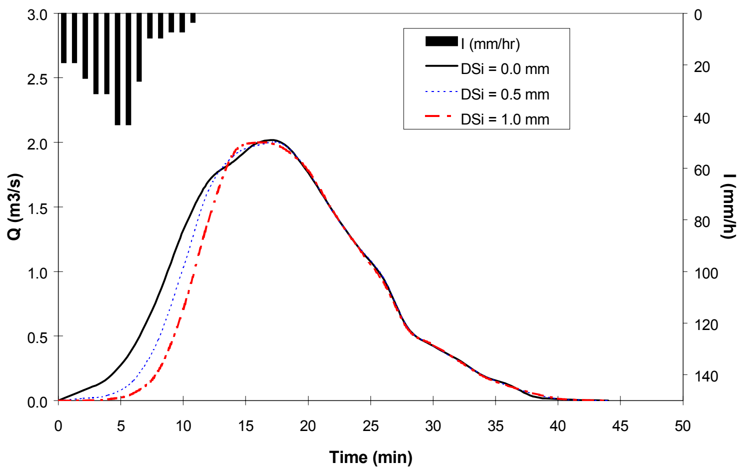


(c) For DCIA = 60%

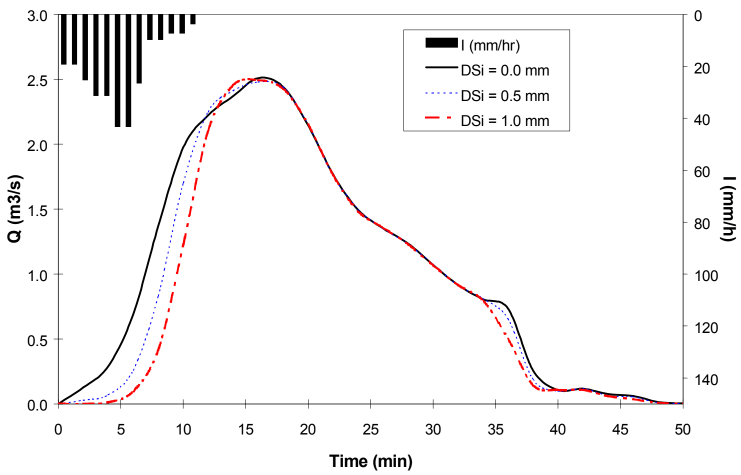
Figure 8.16: Sensitivity of DS_i and DCIA for Altona Meadows Catchment for 1 year ARI Storm Event



(a) For DCIA = 20%



(b) For DCIA = 40%



(c) For DCIA = 60%

Figure 8.17: Sensitivity of DS_i and DCIA for Therry Street Catchment for 1 year ARI Storm Event

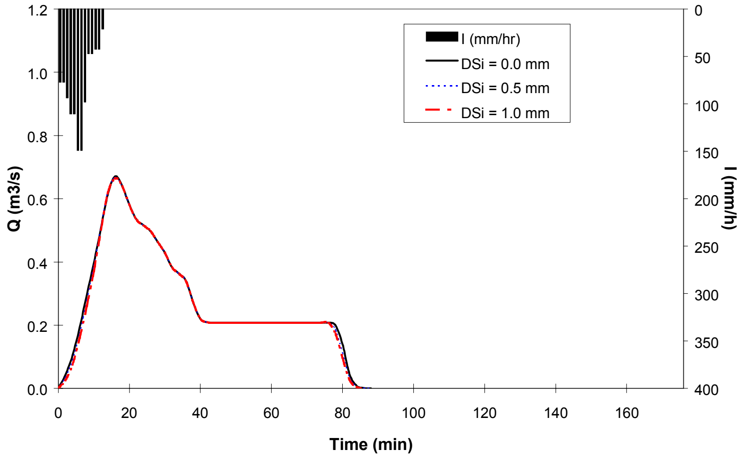
Table 8.6: Hydrograph Attributes for 100-year ARI Storm Event

Attributes	DS_i (mm)	Altona Medows Catchment			Therry Street Catchment		
		DCIA			DCIA		
		20%	40%	60%	20%	40%	60%
Volume (m^3)	0.0	1465	2346	3235	6945	10266	13593
Peak (m^3/s)		0.671	0.859	1.045	2.677	2.782	2.912
Time to peak (min)		18	18	18	18	16	18
Volume (m^3)	0.5	1451	2317	3183	6865	10135	13372
Peak (m^3/s)		0.667	0.859	1.044	2.677	2.787	2.906
Time to peak (min)		18	18	18	18	16	18
Volume (m^3)	1.0	1436	2287	3138	6817	10014	13184
Peak (m^3/s)		0.665	0.859	1.044	2.677	2.790	2.909
Time to peak (min)		18	18	18	18	18	16

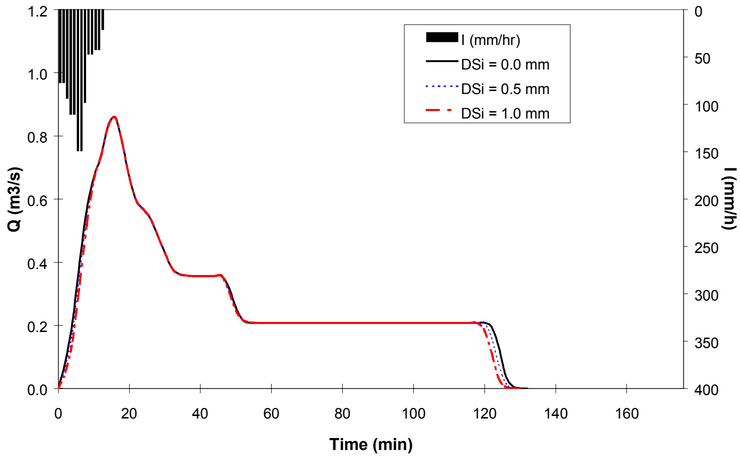
8.7.6 Limitation of Regional Equations

The regional equations were derived in this study only for residential catchments. The catchment area of the study catchments used in regionalisation varied from 3 to 30 ha. These sizes represent typical ‘small’ and ‘medium’ size urban catchments in Melbourne metropolitan areas. Therefore, the derived regional equations are valid only for the residential catchments whose catchment area is less than 30 ha.

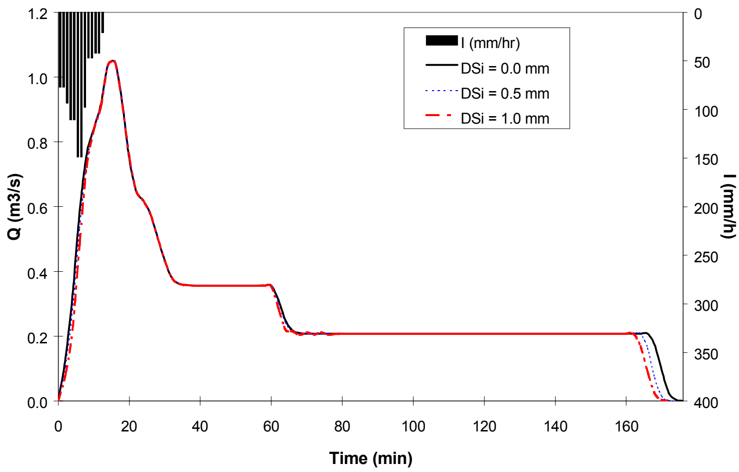
There were several independent and dependent variables used in the regional equations. The statistics (i.e. minimum, mean and maximum) of these variables are given in Table 8.7. As found by other researchers (e.g. Dyer *et al.,* 1994, 1995), the regional equations are valid for use with catchment variables within the range that was used in derivation of these equations. Therefore, the validity of the regional equations outside of these ranges and mixed land-use catchments should be tested before using them.



(a) For DCIA = 20%

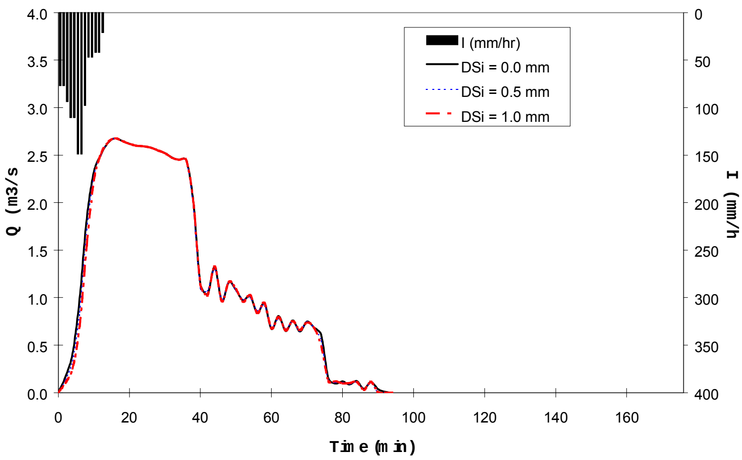


(b) For DCIA = 40%

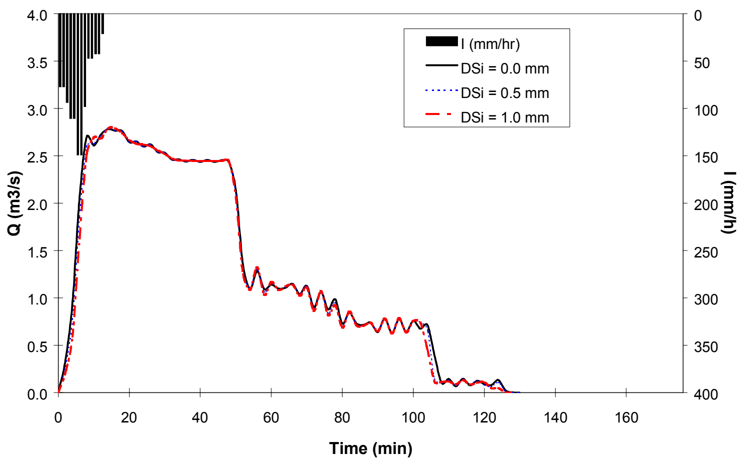


(c) For DCIA = 60%

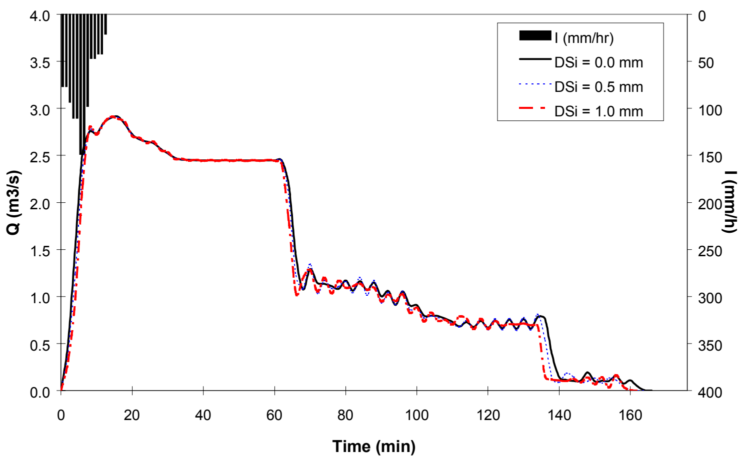
Figure 8.18: Sensitivity of DS_i and DCIA for Altona Meadows Catchment for 100 year ARI Storm Event



(a) For DCIA = 20%



(b) For DCIA = 40%



(c) For DCIA = 60%

Figure 8.19: Sensitivity of DS_i and DCIA for Therry Street Catchment for 1 year ARI Storm Event

Table 8.7: Limitation of Variables for Regional Equations

Variable	Minimum	Mean	Maximum
Catchment area (ha)	3	12	30
Household density (houses/ha)	7	9	14
Average catchment slope (%)	0.3	4.0	6.6
DCIA (%)	19	39	63
SA (%)	2	3	5
DS_i (mm)	0	0.4	0.9

8.8 SUMMARY

Sixteen urban residential catchments from the Melbourne metropolitan area were used to develop regional equations for impervious area parameters directly connected impervious area (DCIA), supplementary area (SA) and directly connected impervious area depression storage (DS_i). These equations were derived for use in ILSAX for ungauged Metropolitan catchments where there are no rainfall-runoff data available to calibrate these parameters. These 16 catchments were subcatchments of seven major residential catchments, which were monitored for rainfall and runoff during 1996 to 1999. The pervious area parameters were not considered in this regionalisation, since the runoffs from pervious areas were not observed during the monitoring period. The parameter values (i.e. DCIA and DS_i) obtained from calibration using hydrograph modelling of 'small' storm events were used in the regionalisation. These two parameters are related to the directly connected impervious area. SA was obtained from the information on drainage plans and through field visits.

Two regional equations, one for DCIA and the other for SA were developed. The independent variable considered in these equations was the household density. A split sample procedure was used to derive the regional equations. From the study catchments, 10 catchments were initially used to derive the regional equations and these equations were tested using the remaining six catchments. Then, all 16 catchments were used to derive the final regional equations. The derived regional equations for DCIA and SA can be applied for residential catchments in the Melbourne metropolitan area whose areas are less than 30 ha. If the household density of existing or proposed catchment is known, then DCIA and SA of the catchment can be estimated using these equations.

DS_i is the only ILSAX model parameter, which is responsible for modelling 'small' storm events. Although the catchment slope was believed to represent the retention capacity (in turn related to DS_i) of the catchment, no correlation was found between DS_i and the average catchment slope of study catchments. Therefore, a range between 0 to 1 mm was recommended in this study for DS_i. Further sensitivity analysis of DS_i and DCIA showed that DS_i was not significantly sensitive to the outlet hydrograph compared to DCIA for both 'small' and 'large' storm events, especially in term of peak discharge, which is more important in urban drainage studies in general terms.

It is important to note that the derived regional equations for DCIA and SA and the range recommended for DS_i should be used for Melbourne metropolitan residential catchments whose properties are within the range of independent and dependent variables used in the regionalisation study. The validity of the equations for DCIA and SA, and the range for DS_i outside the ranges used for regionalisation study should be tested before using them.

CHAPTER 9

SUMMARY OF METHODOLOGY, CONCLUSIONS AND RECOMMENDATIONS

9.1 SUMMARY OF METHODOLOGY

The main aim of the research study in this thesis is to develop improved methods in design and analysis of urban stormwater drainage systems. To achieve this objective, the following methodological aspects were considered.

1) Review of literature related to urban drainage modelling and assessment of current urban drainage modelling practice in Victoria (Australia).
2) Study of modelling options of ILSAX, parameter sensitivity and catchment subdivision.
3) Model parameter optimisation.
4) Development of regional equations for modelling of urban catchments.

A brief summary of the methodology under each of these aspects is given below. The conclusions related to these aspects are given in Section 9.2.

A literature review was conducted on the physical processes of rainfall-runoff modelling of urban catchments and how these processes are modelled by different urban drainage models. In particular, a review was conducted on errors associated with urban drainage models in predicting runoff peak and volume. A customer survey was then conducted in May 1997 to identify the current practice in stormwater drainage design and analysis. Due to resource constraints, the survey was restricted to city/shire councils and consultants in Victoria. From the results of the survey, it was found that the ILSAX model was the widely used stormwater drainage computer model in Victoria. Therefore, the ILSAX model was used in this study to develop improved methods for design and analysis of urban stormwater drainage systems.

The next step was to develop a consistent strategy for modelling the study catchments. A consistent approach was necessary since the study dealt with a large number of catchments. Therefore, a detailed study was conducted to select the appropriate modelling options (out of many available in the ILSAX model) for modelling various urban drainage processes, to study the sensitivity of model parameters on simulated storm hydrographs and the effect of catchment subdivision on storm hydrographs. This was done using two urban catchments in Melbourne considering four design storms. The two catchments represented a typical 'small' and a 'large' urban catchment in the Melbourne metropolitan area. The four design storms ranged from storms of average recurrence interval (ARI) of 1 year to 100 year (including one considerably above 100 year ARI). The results from this detailed study were subsequently used in model parameter calibration.

The third step was to calibrate the ILSAX model for the study catchments using available rainfall-runoff data. A two-stage inner/outer optimisation procedure was developed in this study, to estimate the model parameters of ILSAX. The method was designed to provide the 'best' set of model parameters that considers several storm events simultaneously. Impervious area parameters were obtained from frequent 'small' storm events, while the pervious area parameters were obtained from less-frequent 'large' events. The PEST computer software was used to optimise the model parameters. Twenty-two urban catchments in the Melbourne metropolitan area and one urban catchment in Canberra were used as study catchments. The parameters obtained from the two-stage inner/outer optimisation were compared with the values obtained from rainfall-runoff depth plots and areal photographs (if available) for their validity.

The calibrated model parameters and catchment characteristics were used to develop the regional equations for estimating impervious area parameter for use in ILSAX. These equations can then be used for ungauged catchments, when there are no rainfall-runoff data available to calibrate these parameters. Sixteen residential catchments in the Melbourne metropolitan area were used to develop these regional equations. The dependent variables considered were the directly connected impervious area percentage (DCIA), the supplementary area percentage (SA), and directly connected impervious area depression storage (DS_i). DCIA and DS_i were obtained from the two-stage inner/outer optimisation (but verified with rainfall-runoff plots and areal photographs), while SA was estimated from field visits. Pervious area parameters were not considered in the regionalisation, as the observed runoff events did not have any pervious area runoff. The independent variables considered in the regionalisation were catchment area, distance from Melbourne Central Business District, household density and average catchment slope.

9.2 CONCLUSIONS

9.2.1 Literature Review

Following conclusions were drawn from the literature review conducted in this study.

- The main processes of rainfall-runoff modelling in urban catchments are related to estimation of rainfall excess in pervious and impervious areas, and routing of this rainfall excess through different drainage system components. Different methods are employed by different models to compute the rainfall excess and routing through the drainage system. These different methods in general produce different results.

- Errors related to output responses such as runoff volume, peak discharge and time to peak discharge were quantified using published results of past modelling work. The error analysis was conducted as applicable to both 'calibrated and verified' catchments and ungauged catchments. The results of this study showed that the peak discharge runoff volume simulated by urban drainage models had significant errors. Also, the simulation results from ungauged catchments were less accurate compared to the results from gauged catchments, which is the case in general.

- The literature review also showed that model error was depended on storm characteristics and land-use type of catchments.

9.2.2 Customer Survey

Following important findings were extracted based on the analysis of responses of the customer survey conducted in Victoria in May 1997.

- Majority of respondents (about 80%) used methods such as the Statistical Rational method, which involve many assumptions, for urban drainage design and analysis. The remaining 20% used computer models, which simulate hydrologic and hydraulic processes related to urban drainage systems. This is a disturbing fact, since large annual expenditures are spent on urban drainage construction works throughout Victoria and other parts of Australia.

- Lower use of computer models was due to:
  - lack of user-friendliness of the computer models,
  - adequate guidelines were not available to select the model parameters (especially for ungauged catchments), and
  - adequate guidelines were not available to select the appropriate modelling options (out of many available in these models), which can be used to model various processes.

- The ILSAX model was the widely used urban drainage computer model in Victoria.

9.2.3 Data Collection and Analysis

The accuracy of any modelling exercise largely depends upon the accuracy of the data. For the calibration of rainfall-runoff models, accurate catchment data and rainfall-runoff data are required. The rainfall-runoff data can be obtained through a well maintained data acquisition program with pluviometers and automatic flowmeters installed at strategic locations of the stormwater drainage system. Raw hydrological data thus acquired should be carefully checked for accuracy and consistency. Graphical time series plots of recorded flow depth and velocity, hyetograph and hydrograph on the same chart, and rainfall-runoff depth plots were the some techniques used in this study to check the accuracy and consistency of rainfall-runoff data and found to be useful techniques.

The catchment data can be obtained from drainage plans, land-use maps, contour maps, soil maps and areal photographs of the catchments. However, the information was obtained from above source should be verified through field visits.

9.2.4 Selection of Appropriate Modelling Options

Most drainage models have more than one option to model various processes related to urban drainage systems. In analysing a large number of catchments as in this study, it is convenient to use a single modelling option in modelling a process of all study catchments. This becomes necessary when the results are used in a regionalisation. The ILSAX model has more than one option in modelling the following processes.

- Loss subtraction method for pervious area
- Time of entry for overland flow routing
- Pipe and channel routing
- Modelling of pit inlets

Based on analysis of modelling of various options using two urban catchments (one typical 'small' and one typical 'large') and four design storms (ARI ranging from 1-year to larger than 100-year), the following conclusions were made. These conclusions were later used in the calibration of the study catchments.

- For pervious area loss modelling, the 'supply' rate method was used, since this method allows infiltration to occur after rainfall has stopped, which is closer to the reality. Data requirements for both methods (i.e. losses subtracted from 'supply' rate and losses subtracted from rainfall) are the same.
- For overland flow routing, the ARR87 method was used from the available two methods (i.e. ILLUDAS-SA procedure and ARR87 method), since this method models overland drainage components (e.g. surface, gutter flow) more realistically. However, data requirements are more in the ARR87 method compared to the ILLUDAS-SA method.

- For pipe routing, the implicit method was used, since this method considered both time lag effects and storage routing effects, which is closer to the reality. Data requirements for both methods (i.e. time-shift and implicit methods) are the same.
- Modelling pit capacity is important since the separation of pipe flow and bypass flow is due to pit capacity. Studies conducted with study catchments revealed that either pit capacity option (i.e. finite or infinite capacity) could be used for modelling of pits for 'small' storm events whose ARI is less than 5-year. If the storm event has an ARI greater than 5 years, the finite pit capacity option (which reflects the actual pit capacity) should be used to model bypass flow and pipe flow accurately. However, the data requirements for finite pit capacity option are more. The other problem with this method is that the pit capacity parameters for all currently used pit types and sizes are not available. These pit capacity parameters are currently obtained from physical hydraulic model studies.

9.2.5 Sensitivity Analysis

In simulation of 'small' storm events, the hydrograph attributes (i.e. runoff volume, peak discharge, time to peak discharge and hydrograph shape) were sensitive to impervious area parameters, while they were not sensitive to pervious area parameters. This is because there is no pervious area runoff for 'small' runoff events. The impervious area parameters are also important for 'large' events, since runoff for these events is generated from both impervious and pervious areas. Therefore, the accurate estimation of impervious area parameters is very important. The only impervious parameter of the ILSAX model is DS_i. The sensitivity of DS_i decreases with the increase of storm magnitude. The pervious area parameters were sensitive only for 'large' storm events in simulating the runoff volume and peak discharge. The pervious area parameters of ILSAX are antecedent soil moisture content (AMC), soil curve number (CN) and pervious area depression storage (DS_p), and these parameters were considered in the sensitivity analysis. The sensitivity transfers from impervious area parameters to pervious area parameters when storm magnitude increases. These impervious and pervious parameters were called hydrologic parameters in this thesis.

The pit capacity parameters of on-grade pits (CAP3 and CAP4), choke factor (CF), gutter factor (GUT) and pipe roughness (N_p) were also sensitive parameters in simulating the peak discharge in addition to hydrologic parameters. Retardance coefficient of pervious area (N_r) was the least sensitive parameter to the peak discharge. These parameters are called routing parameters in this thesis. They do not affect the runoff volume.

9.2.6 Catchment Subdivision

To account for areal variability of rainfall and losses, and to model different travel times of various parts of the catchment to the outlet, the catchment should be subdivided into a number of subcatchments. In general, three levels of subdivisions can be considered, namely coarse subdivision (neglecting all lateral drains and considering each catchment to contribute directly to the inlets of the main drain), a medium subdivision (considering main drain and first order laterals) and a fine subdivision (considering all drains). The effect of the catchment subdivision is significant for the catchments, where routing effects dominate the rainfall-runoff process.

The results of the study suggested that the catchment subdivision was important in modelling the peak discharge. This was because of the dependence of peak discharge on the overland flow behaviour, the storage effects of the drainage system and the pit capacity, which were modelled differently for different levels of subdivision. In medium and coarse subdivisions, some pits are not modelled. For larger storm events (i.e. greater than 5 year ARI), if finite pit capacity option is used to model bypass flow, the storage effects of pits that were not considered in modelling should be taken into account. One way to handle this is to define an equivalent pit capacity for the modelled pits, to allow for the capacity of ignored pits in modelling. However, currently there is no method to compute the equivalent pit capacity. Therefore, the equivalent capacity of the modelled pits can be estimated based on engineering judgement considering some increase for the pit capacity through pit capacity parameters.

Based on the analysis of different subdivisions, any form of subdivision can be used if only the runoff volume is of interest. For the accurate simulation of peak discharge, the fine subdivision should be used.

9.2.7 Model Calibration

Three methods were used to estimate the DCIA of study catchments. They are:

1) Use of areal photographs
2) Use of rainfall/runoff depth (RR) plots
3) Hydrograph modelling of storm events

Of these methods, the latter two methods also produced DS_i. The RR plot gives only DCIA and DS_i, while the hydrograph modelling produces the pervious area parameters (i.e. CN and DS_p) in addition to DCIA and DS_i.

The first method gives the total impervious area, which is the sum of DCIA and supplementary area percentage. Therefore, the first method gives only an upper bound for DCIA. The second and third methods require rainfall-runoff data and therefore the catchments should be gauged. The RR plots consider only the runoff volume, while the hydrograph modelling calibration considers all hydrograph attributes such as runoff volume, peak discharge, time to peak discharge and hydrograph shape. Since the third method considers all hydrograph attributes in estimating parameters, it is preferred but should be verified against the other methods. The hydrograph modelling approach was used in this thesis to calibrate the model parameters for study catchments. An optimisation approach called two-stage inner/outer optimisation was developed in this study to calibrate the model parameters using hydrograph modelling.

This method produces the 'best' set of model parameters which considering all calibration events. The parameter optimisation of each drainage catchment model was carried out in two stages. During the first stage, the model parameters responsible for 'small' storm events (i.e. DCIA and DS_i) were obtained. During the second stage, the additional parameters responsible for 'large' storm events (i.e. DS_p and CN) were obtained. During the second stage, no changes were made to the parameters obtained from the first stage. Each stage consisted of two loops (i.e. inner and outer). The inner loop uses PEST computer software to optimise corresponding model parameters, a set of model parameters is obtained for each storm event. The outer loop optimises the above sets of model parameters to produce the 'best' set considering all calibration events and hydrograph attributes of runoff volume, peak discharge and time to peak discharge. The outer optimisation was carried out manually.

The two-stage inner/outer optimisation was used for Melbourne metropolitan catchments to optimise impervious area parameters using hydrograph modelling. It was used to estimate both impervious and pervious area parameters for the Giralang catchment. Comparing observed hydrographs with modelled hydrographs using the 'best' set of parameters obtained from the optimisation, it was found that the method produced satisfactory results.

The 'best' set of model parameters obtained from hydrograph modelling was verified using three independent events for each catchment. Generally, all the verification plots were reasonably accurate for all study catchments. The 'best' set of parameters obtained from hydrograph modelling was compared with the parameter values obtained from the other methods (i.e. RR plots and areal photographs). However, the latter estimates were used only as checks for the 'best' parameter set obtained from hydrograph modelling. By comparing the 'best' set of model parameters obtained from hydrograph modelling with those of the other methods (i.e. RR plots and areal photographs), it was found the former set of parameter estimates for the study catchments. These parameter values were used for the regionalisation study. Based on the results of verification plots and checks with respect to other methods, it can be said that two-stage inner/outer optimisation produced satisfactory results and therefore, it can be recommended to calibrate any urban drainage model. The calibrated parameter values for the Melbourne catchments and the Canberra catchment were within the limit of the values given in the literature.

For Melbourne study catchments, areal photographs were available for ten study catchments. DCIA obtained from hydrograph modelling using the two-stage inner/outer optimisation procedure and rainfall-runoff depth plots were compared with those obtained from the areal photographs, and found to be less. This is to be expected since the areal photographs give the total impervious area which includes with DCIA and supplementary area. The difference of the values from both methods for the study catchments was within 10% limit except one catchment. The eighteen catchments out of 22 catchments gave less than 10% difference for DCIA computed from the RR plots and the optimisation methods. The 50% of study catchments gave less than 0.25 mm different for DS_i computed from the both methods.

9.2.8 Parameter Regionalisation

Two equations were developed to estimate DCIA and SA from the household density. Sixteen catchments were used in the regionalisation study. A split sample procedure was used to derive the regional equations. From the study catchments, 10 catchments were initially used to derive the regional equations and these equations were tested using the remaining six catchments. Then, all 16 catchments were used to derive the final regional equations. The derived regional equations for DCIA and SA can be applied for residential catchments in the Melbourne metropolitan area whose areas are less than 30 ha. If the household density of existing or proposed catchment is known, then DCIA and SA of the catchment can be estimated using these equations.

Although the catchment slope was believed to represent the retention capacity (in turn related to DS_i) of the catchment, no correlation was found between DS_i and the average catchment slope of study catchments. Therefore, a range between 0 to 1 mm was recommended in this study for DS_i. Further sensitivity analysis of DS_i and DCIA showed that DS_i was not significantly sensitive to the outlet hydrograph compared to DCIA for both 'small' and 'large' storm events, especially in term of peak discharge, which is more important in urban drainage studies in general terms.

The derived two equations for DCIA and SA, and the recommended range for DS_i are valid for only residential catchments in the Melbourne metropolitan area, whose catchment area is less than 30 ha. Therefore, it is necessary to check whether the catchment under investigation falls into this category and within the range of catchment parameters used to determine these equations. Where the variables of the catchment under investigation falls outside the range of the variables of the catchments used to define the regional equations, the modeller should apply these equations to this catchment with caution.

9.2.9 Other Issues

Although ILSAX is widely used by the local government authorities and consultants in Australia to design and analyse of urban drainage systems, there are no adequate guidelines available to develop models for both gauged and ungauged urban catchments. The user manual (O'Loughlin, 1993) provides the information on how to assemble data to construct a model and some guidelines. However, there are many other important decisions the user has to make before constructing a model, which have not been covered in detail in the user manual. Some of these have already been explained in the previous conclusions. Some other issues are briefly discussed below as further conclusions.

9.2.9.1 Catchment data

Unless catchments are fully developed, urban catchments should be treated as dynamic systems, in which catchment properties change with time. Therefore, the ILSAX model parameters, especially the parameter related to impervious area, changes with the development. If rainfall-runoff data are available for a catchment for a certain time of the year, then the other catchment data should also be collected relevant to the same period from other sources such as areal photographs and maps.

9.2.9.2 Subcatchment slope

The times of entry (or times of concentration) are required to set the base lengths of time-area diagrams in ILSAX for impervious and pervious areas. The user has to enter flow path lengths and slopes into the model. Therefore, it is important to identify the flow paths, and estimate their lengths and slopes correctly.

The subcatchment of a pit may consist of several household properties. In estimating the subcatchment slope, it is assumed that water flows to the pit in the direction of the subcatchment slope to pit. Therefore, the usual practice is to compute the slope from the flow path determined by contour maps without considering actual flow paths. In urban catchments, house-fencing around properties significantly changes the flow path. These fences are usually made of colour bond steel or solid timber. Therefore, there is no free flow between adjoining properties, and runoff from a property can then be analogues to runoff from a rectangular tank, which is open on one side and water flows freely towards this open end. Therefore, the length and slope of subcatchments relevant to modelled pits should consider flow paths of individual properties instead of considering several properties together and the flow path determined from contours.

9.2.9.3 Computational time step

The ILSAX model allows the user to specify the time step for hydrograph simulation. This computational time step should be less than the time increment of rainfall input so that rainfall can be adequately modelled. Therefore, a time interval less than two minutes is recommended based on the studies of this thesis. Smaller time intervals increase the accuracy of modelling at the expense of computer time. ILSAX has a limitation of 720 time steps in defining the simulated hydrograph, which limit the use of smaller time intervals. This may not be a serious problem, since ILSAX can be recompiled with a larger number of time steps.

9.2.9.4 Property time

For the Australian urban catchments, the property time is between 2-7 min according to the published literature (e.g. The Institution of Engineers, 1987; O'Loughlin, 1993; Stephens and Kuczera, 1999). The best way to handle the unknown property time of a catchment is to assume 5-min property time in modelling the catchment and then compare the time to peak discharge of the simulated hydrograph with that of observed hydrograph. If the time to peak discharge is considerably different, then the property time should be changed considering values between 2-7 min until get the best match is obtained. However, this method can be used only for gauged catchments, where rainfall/runoff data are available.

9.2.10 Transferability of Results to Other Models

The findings regarding the computation of losses, routing methods, pit inlet modelling, and catchment subdivision can be extended to any type of urban drainage models where these options are available for modelling of the hydrologic and hydraulic processes. The proposed calibration method (i.e. two-stage inner/outer optimisation) can also be used with other urban drainage models to calibrate their parameters.

The derived regional equations for DCIA and SA can be used for any other urban stormwater drainage model in which subcatchment surface is modelled using three types of land parameters namely directly connected impervious area, supplementary area and pervious area. The recommended values for DS_i can also be used in any urban drainage model, which model initial losses separately.

9.3 RECOMMENDATIONS FOR FUTURE RESEARCH

9.3.1 Equivalent Pit Capacity

The main difficulty with medium and coarse subdivision is to estimate the pit capacity of the selected pits for modelling, to account for the effects of the other pits that were not modelled. One way would be to define an equivalent pit capacity for the modelled pits to account for ignored ones. Previous studies on urban drainage analysis have not considered this effect and therefore, further research is required to compute the equivalent pit capacity.

9.3.2 Choke Factor

The results of this study showed that the choke factor (CF) was sensitive in simulating the peak discharge in drainage pipes. CF is a dynamic factor for a catchment. This factor depends on the frequency of street cleaning, dry and wet periods, and location of a pit, and therefore, may be correlated with pollutant build-up and washoff patterns in the catchment. Further research should be conducted to determine the appropriate value of CF for design and evaluation of urban drainage systems based on above dependencies.

9.3.3 Property Time

The ILSAX model uses 5-min property time as default. Some other studies had suggested that the property times between 2 to 7 min (e.g. The Institution of Engineers, 1987; O'Loughlin, 1993; Stephens and Kuczera, 1999). Further research is required to determine the suitable property time.

9.3.4 Determination of AMC

At present, ILSAX uses four AMC levels corresponding to total rainfall depth in five days preceding the storm. It does not consider the distribution of the rainfall during the previous five days. In reality, the condition that 25 mm of rainfall occurs in the first day and the condition that 25 mm of rainfall occurs in the fifth day is not the same. Therefore, this method has to be improved to get better results. The <u>author</u> suggests computing the AMC, which relates to the antecedent precipitation index (API), by one of the two methods given in Equations 9.1 and 9.2.

$$API_0 = P_0 + P_1K + P_2K^2 + \dots\dots\dots\dots + P_nK^n \qquad (9.1)$$

or

$$API_0 = P_0 + API_i * K \qquad (9.2)$$

where	K	is a recession factor less than unity,
	P_i	is the 24 hour rainfall depth (mm) on i^{th} day, and
	Subscripts of 0, 1, 2	Days prior to storm event; 0 being for the current

day, 1 for one day prior to storm, etc.

These forms of equations have been used by Cordery (1970) and Loy *et al.* (1996).

9.3.5 Curve Numbers

ILSAX defines four soil classifications, designated as A, B, C and D for the soil underlying the pervious portions of the catchment. These curves were developed by U.S. Soil Conservation Service for U.S. catchments. Applicability of these curves to Victorian (or Australian) urban catchments have not been tested properly. Therefore, further research is required to identify whether these curves are suitable for Victorian (or Australian) urban catchments. These curves can be tested using results from field infiltrometer tests with different antecedent moisture conditions (AMCs).

9.3.6 Effect of Scaling Factors

It is expected that there would be rainfall variation within the catchment, especially for large rainfalls. An investigation of this aspect would be worthwhile to pursue in future.

9.3.7 Modelling Concepts and Parameter Settings

Once the important parameters are identified through the sensitivity analysis and calibration, it is worthwhile to investigate the modelling concepts and parameter settings for calibration events of each catchment. These effects were not investigated in detail in the thesis because of the large amount of work involved in such a study. Furthermore, such an analysis would deviate from the main objective of the thesis. Nevertheless, it is useful to carry out this investigation and proposed for future work.

REFERENCES

1. ACT Department of Urban Services, (1996), Urban Stormwater- Edition 1, ACT, Australia.

2. Adams, A., (1991), "Application and Comparison of RORB, WBNM and RAFTS Runoff Routing Models", International Hydrology and Water Resources Symposium, Perth, (volume 2), pp 485-491.

3. Aitken, A.P., (1973), "Hydrologic Investigation and Design in Urban Areas", A Review, Australian Water Resources Council, Technical Paper No. 5, Canberra, Australia.

4. Aitken, A.P., (1975), "Hydrologic Investigation and Design of Urban Stormwater Drainage Systems", Australian Water Resources Council, Technical Paper No.10, Canberra, Australia.

5. Alley, W. M. and. Smith, P.E., (1990), "Distributed Routing Rainfall-Runoff Model (DR3M)", Mississippi, Water Resources Division, Gulf Coast Hydroscience Center.

6. Alley, W.M. and Veenhuis, J.E., (1983), "Effective Impervious Area in Urban Runoff Modeling", Journal of Hydraulic Engineering Vol. 109, No. 2, pp 313-319.

7. Argue, J., (1986), "Storm Drainage Design in Small Urban Catchments", Special Report No. 34, Australian Road Research Board, Australia.

8. Australian Water Resources Council, (1973), Australian Representative Basin Programme- Progress 1973, Hydrological Series No.8.

9. Ball, J.E., (1987a), "A Comparison of the Hydraulics in Three Urban Drainage Models", Conference on Hydraulics in Civil Engineering, Melbourne, 12-14 October 1987, pp 88-92.

10. Ball, J.E., (1987b), "Modelling of Unsteady Flows in Urban Networks Including Some Effects of Manholes", PhD Thesis, Department of Civil Engineering and Surveying, University of Newcastle, Australia.

11. Ball, J.E., (1992), "The Influence of Temporal Patterns on Surface Runoff Hydrographs", International Symposium on Urban Stormwater Management, Sydney, pp 64-69.

12. Bauer, L.R. and Hamby, D.M., (1991), "Relative Sensitivities of Existing and Novel Model Parameters in Atmospheric Tritium Dose Estimates", Rad. Prot. Dosimetry 37, pp 253-260.

13. Bedient, P.B. and Huber, W.C., (1992), "Urban Hydrology. Hydrology and Floodplain Analysis", Addison-Wesley Publishing Company, USA.

14. Bielawski, Z., (1984), "The Nash Cascade or a Single Linear Reservoir for Modelling Urban Runoff", Proceeding of the Third International Conference on Urban Drainage, Sweden, pp 203-211.

15. Black, D.C. and Aitken, A.P., (1977), "Simulation of the Urban Process", Australian Water Resources Council, Technical Paper No. 26, Canberra, Australia.

16. Bouvior, C. and Desbordes, M., (1990), "Urban Runoff Modelling in Western Africa", Fifth International Conference on Urban Storm Drainage, Japan, pp 251-256.

17. Boyd, M.J., Bufill, M.C., and Knee, R.M., (1993), "Pervious and Impervious Runoff in Urban Catchments", Hydrological Sciences, Journal, pp. 463-478.

18. Boyd, M.J., Rigby, T. and Drie, R.V., (2000), Watershed Bounded Network Model, WBNM2000, Internet, http://www.uow.edu.au/eng/research/wbnm.html.

19. Breshears, D.D., (1987), Uncertainty and Sensitivity Analyses of Simulated Concentrations of Radionuclides in Milk, M.Sc. Thesis, Fort Collins, CO: Colorado State University,.

20. Bufill, M.C. and Boyd, M.J., (1992), "A Simple Flood Hydrograph Model for Urban Catchments", International Symposium on Urban Stormwater Management, Sydney, Australia, pp 98-103.

21. Bufill, M.C., (1989), "Effect of Urbanisation on Floods", PhD thesis, Department of Civil and Mining Engineering, The University of Wollongong, Australia.

22. CEPA (Commonwealth Environment Protection Agency), (1993), "Urban Stormwater", Australia.

23. Chapman, T.G. and Salmon, M.A., (1996), "Roof Runoff Data Acquisition and Analysis", 23rd Hydrology and Water Resources Symposium, Hobart, Australia, pp 657-661.

24. Cheng, X. and Noguchi, M., (1996), "Rainfall-Runoff Modelling by Neural Network Approach", Proceeding of the International Conference on Water Resources and Environmental Research (Volume II), Japan, pp 143-150.

25. Chiang, C.Y. and Bedient, P.B., (1986), "PIBS Model for Surcharge Pipe Flow", Proceedings of the American Society of Civil Engineers, Journal of the Hydraulic Engineering, Vol. 112, No 3, pp. 181-192.

26. Chiew, F.H.S. and McMahon, T.A., (1993), "Data and Rainfall-Runoff Modelling in Australia", Hydrology and Water Resources Symposium, Newcastle, Australia, 1993, pp 305-316.

27. Choi, K. and Ball, J.E., (1999), "Estimation of Control Parameters for Urban Runoff Modelling by Using a Hydroinformative System", Proceedings of 8th International Conference on urban Storm Drainage, Sydney, pp 1776-1783.

28. Chow, V.T., (1973), "Open-Channel Hydraulics", McGraw-Hill Book Company.

29. Cordery, I., (1970), "Initial Loss for Flood Estimation and Forecasting", Journal of the Hydraulic Division, ASCE(HY 12), pp 2447-2465.

30. Crawford, N.H. and Linsley, R.K., (1966), "Digital Simulation in Hydrology, Stanford Watershed Model IV", University of Stanford, California, U.S.A., Tech. Rep. 39.

31. Crick, M.J., Hill, M.D. and Charles, D., (1987), "The Role of Sensitivity Analysis in Assessing Uncertainty", Proceedings of a NEA Workshop on Uncertainty Analysis for Performance Assessments of Radioactive Waste Disposal Systems, Paris, OECD.

32. Cullino, C., (1995), "Urban Stormwater Management in Search of Best Practice", The Second International Symposium on Urban Stormwater Management, Melbourne, Australia, pp 49-53.

33. Danish Hydraulic Institute and Barnett Consultants Ltd, (1993), MIKE11- A Computer Based Modelling System for Rivers and Channels (Manual, Version 3.01).

34. Danish Hydraulic Institute, (1988), "Modelling of Urban Sewer Systems on Microcomputers", MOUSE V3.0, Users Guide and Technical Reference, Lawson and Treloar Pty Ltd, Australia.

35. Dayaratne, S.T. and Perera, B.J.C., (1999), "Parameter Optimisation of Urban Stormwater Drainage Models", Proceedings of the 8th International Conference on Urban Storm Drainage, Sydney, Australia, pp1768-1775.

36. Dayaratne, T.S., (1996), "Assessment of Errors in Commonly Used Event Hydrograph Urban Catchment Models", Master Thesis, Department of Civil and Environmental Engineering, University of Melbourne, Australia.

37. Department of Primary Industries and Energy, (1992), Floodplain Management in Australia, Volume 1: Executive Summary. Australian Water Resources Council, Water Management Series No. 21.

38. Driver, N.E. and Tasker, G.D., (1988), "Techniques for Estimation of Storm Runoff Loads, Volumes and Selected Constituent Concentrations in Urban Watersheds in the United States", USA, U.S. Geological Survey.

39. Dyer, B. G., Nathan, R. J., McMahon, T. A. and O'Neill, I. C., (1995), "Prediction Equations for the RORB Parameter k_c Based on Catchment Characteristics", Australian Journal of Water Resources, Vol. 1, No. 1, pp 29-38.

40. Dyer, B.G., Nathan, R.J., McMahon, T.A. and O'Neill, I.C., (1994), "Development of Regional Prediction Equations for the RORB Runoff Routing Model", Cooperative Research Centre for Catchment Hydrology, Report 94/1.

41. Fleming, N.S., (1994), An Investigation into Rainfall-Runoff Relationships, Department of Civil and Environmental Engineering, The University of Adelaide, Australia.

42. Frankel, R.J. and Hansen, W.W., (1968), "Biological and Physical Responses in a Freshwater Dissolved Oxygen Model", Advances in Water Quality Improvement, Eds. Gloyna, E. and Eckenfelder, University of Texas Press, London, pp 126-140.

43. Fuchs, L. and Verworn, H.R., (1988), Microcomputer in der Stadtentwasserung, Kannalnetzberechnung, Modellbeschreibung, Hannover.

44. Gardner, R.H., O'Neill, R.V., Mankin, J.B., Carney, J.H. and Jones, J., (1980), "Application of Error Analysis to a Marsh Hydrology Model", Water Resources Research 16, pp 659-664.

45. Ghafouri, R.A., (1996), "Deterministic Analysis and Simulation of Runoff in Urban Catchments", PhD Thesis, Department of Civil and Mining Engineering, University of Wollongong, Australia.

46. Giancarro, F.B., (1995), "Management of a Stormwater Drainage System within an Established Urban Area", The Second International Symposium on Urban Stormwater Management, Melbourne, Australia, pp 21-25.

47. Goonetilleke, A. and Jenkins, G., (1997), "GIS as a Tool in Predicting Urban Catchment Response", 24th Hydrology and Water Resources Symposium Proceedings, Auckland, New Zealand, pp 392-397.

48. Goulburn-Murray Water, (1997), "Simulation Model of Birch Creek System", Final Report, Victoria, Australia.

49. Goyen, A.G., Phillips, B.C. and Blaik, S., (1989), "Australian Urban Stormwater Drainage Practice- The Technology Transfer Dilema", Hydrology and Water Resources Symposium, Christchurch, NZ, pp 459-463.

50. Grandjean. I. and Zech, Y., (1991), "Rainfall-Runoff Modelling of Partly Urbanized Watersheds - Use of Digital Terrain Models", New Technologies in Urban Drainage (UDT 1991, Dubrovnik) Elsevier Applied Sciences, London, pp 365-372.

51. Green, W.H. and Ampt, C.A., (1911), "Studies on Soil Physics; Flow of Air and Water Through Soils", Journal of Agriculture Science 4, pp 1-24.

52. Hall, M.J., (1984), Urban Hydrology, Elsevier Applied Science Publishers, London-New York.

53. Hamby, D.M., (1994), “A Review of Techniques for Parameter Sensitivity Analysis of Environmental Models”, Environmental Monitoring and Assessment 32, Kluwer Academic Publishers, Netherlands, pp 135-154.

54. Hammer, M.J., (1977), Water and Waste-Water Technology, John Wiley & Sons Inc., New York.

55. Heeps, D.P. and Mein, R.G., (1973a), “Independent Comparison of Three Urban Runoff Models”, Proceedings of the American Society of Civil Engineers, Journal of the Hydraulics Division, Vol. 100, No Hy 11, pp 995-1009.

56. Heeps, D.P. and Mein, R.G., (1973b), “An Independent Evaluation of Three Urban Stormwater Models”, Report No. 4/1973, Civil Engineering Research Reports, Monash University, Australia.

57. Heeps, D.P., (1973), “An Evaluation of Three Urban Hydrologic Models for the Prediction of Stormwater Runoff”, the Thesis of Master of Engineering Science, Monash University, Australia.

58. Hoffman, F.O. and Gardner, R.H., (1983), “Evaluation of Uncertainties in Environmental Radiological Assessment Models” in Till, J.E. and Meyer, H.R. (eds), Radiological Assessments: a Textbook on Environmental Dose Assessment, Washington, DC: U.S. Nuclear Regulatory Commission Report No. NUREG/CR-3332.

59. Horton, R.E., (1940), "An Approach Toward a Physical Interpretation of Infiltration Capacity”, Proceeding of Soil Science 5, pp 399-417.

60. Hossain, A., Rao, R. and Delleur, J.W., (1978), “Estimation of Direct Runoff from Urban Watersheds”, Journal of the Hydraulics Division, Proceedings of the American Society of Civil Engineers, Vol 104, No HY2, pp 169-188.

61. Hydraulic Research Limited, Wallingford, (1991), WALLRUS General Reference Manual, Fourth Edition, Wallingford, U.K.

62. Hydrologic Engineering Centre (HEC), (2000), "HEC-RAS User's Manual", Internet, http://www.wrc.hec.usace.army.mil/software-distib/hec-ras/hecrasprogram.html.

63. Johanson, R.C., Imhoff, J.C., Kittle, J.L. Jr. and Donigian, A.S. Jr., (1984), "Hydrological Simulation Program- FORTRAN (HSPF): Users Manual for Release 8.0", Office of Research and Development, Environmental Research Laboratory, Athens, GA. Volumes 1 and 2.

64. Jones, D.E., Jr., (1971), "Where is Urban Hydrology Practice Today", Proceeding of American Society of Civil Engineering, Hydr. Div. 97 (HY 2), pp 257-264.

65. Kemp, D.J., (1994), "Calibration of the ILSAX and RAFTS Models on Two Urban Catchments in Adelaide, South Australia", Surface Hydrology and Water Resources Papers, Volume 3, Water Down Under 94 Conference, Adelaide, Australia, 21-25 November 1994, pp 461-464.

66. Kidd, C.H.R, (1978a), "Rainfall-Runoff Processes Over Urban Surfaces", Proceedings of an International Workshop, Report No 53, Institute of Hydrology, Wallingford, Oxon.

67. Kidd, C.H.R., (1978b), "A Calibrated Model for the Simulation of the Inlet Hydrograph for Fully Sewered Catchments", Urban Storm Drainage, Proceedings of the International Conference held at the University of Southampton, U.K., pp 172 - 186.

68. Kuczera, G., (1987), Fitting and Testing Mathematical Hydrologic Models: A User Manual for Program Suite NLFIT Version 2, Department of Civil Engineering and Surveying, The University of Newcastle, Australia.

69. Kumar, S. and Jain, S., (1982), "Application of SCS Infiltration Model", Water Resource Bulletin, 18(3), pp 503-507.

70. Laurenson, E.M. and Mein, R.G., (1990), "RORB-Version 4, Runoff Routing Program", User Manual, Department of Civil Engineering, Monash University, Australia.

71. Laurenson, E.M., (1964), "A Catchment Storage Model for Runoff Routing", Journal of Hydrology, Vol. 2, pp 141-163.

72. Laurenson, E.M., Codner, G.P., and Mein, R.G., (1985), “Giralang/ Gungahlin Paired Catchment Study, A Review”, Department of Housing and Construction.

73. Lei, J.H. and Schilling, W., (1994), “Parameter Uncertainty Propagation Analysis for Urban Rainfall Runoff Modelling”, Water Science and Technology, U.K., Vol 29, No 1-2, pp 145-154.

74. Lei, J.H., (1996), “Uncertainty Analysis of Urban Rainfall-Runoff Modelling”, PhD Dissertation, Faculty of Civil Engineering, Norwegian University of Science and Technology, Trondheim, Norway.

75. Lichty, R.W., Dawdy, D.R. and Bergmann, J.M., (1968), “Rainfall Runoff Model for Small Basin Flood Hydrograph Simulation”, Proc. Symp. The Use of Analogue and Digital Computers in Hydrology, Vol 2, Tucson, IASH/UNESCO, pp 356-367.

76. Lindberg, S. and Car, R.S., (1992), “MOUSE- Towards on On-Line Modelling System”, International Symposium on Urban Stormwater Management, Sydney, pp 294-297.

77. Linsley, R.K., Kohler, M.A. and Paulhus, J.L.H., (1982), Hydrology for Engineers, McGraw-Hill Inc.

78. Loy, A., Ayre, R.A. and Ruffini, J.L., (1996), “Regional Loss Model Relationships for Catchments in South East Queensland”, 23rd Hydrology and Water Resources Symposium, Hobart, Australia, pp 133-140.

79. Maheepala, U.K, Dayaratne, S.T. and Perera, B.J.C., (1999), “Diagnostic Checking and Analysis of Hydrologic Data of Urban Stormwater Drainage Systems”, 25th Hydrology and Water Resources Symposium and 2nd International Conference on Water Resources and Environment Research, Queensland, Australia, 6-8 July 1999, pp 813-818.

80. Maheepala, U.K. and Perera, B.J.C., (1999), “Quality Assurance Checks for a Hydrologic Data Acquisition Program for Urban Stormwater Drainage Systems”, 8th International Conference on Urban Stormwater Drainage, Australia, pp 2115-2122.

81. Maheepala, U.K., (1999), "Data Acquisition and Modelling of Urban Drainage Catchment in the City of Knox", Masters Thesis, Department of Civil Engineering, Monash University, Australia.

82. Maheepala, U.K., Nichol, E.R., Takyi, A.K. and Perera, B.J.C., (1998), "Occupational Health and Safety Aspects of an Data Acquisition Program", Proceedings of the 'Safety in Action' Conference, Melbourne, Australia.

83. Maheshwari, B.L., (1988), Evaluation of Border Irrigation Models for South-East Australia, PhD Thesis, Department of Civil and Agriculture Engineering, University of Melbourne, Australia.

84. Maidment, D.R., (1993), Hand Book of Hydrology, McGraw-Hill, USA.

85. Marquardt, D.W., (1963), "An Algorithm for Least-Squares Estimation of Nonlinear Parameters", Journal of Social Industrial and Applied Mathematics, 11(2), pp 431-441.

86. McCuen, R.H., (1972), "The Role of Sensitivity Analysis in Hydrologic Modelling", Journal of Hydrology, Vol. 18, pp 37-53.

87. McIllawraith, J.F., (1959), "The Stormwater Drainage of Roofs, Street Gutters and Gullies", Metropolitan Water, Sewerage and Drainage Board, U.K.

88. Mein, R.G. and Goyen, A.G., (1988), "Urban Runoff.", Civil Engineering Transactions, CE 30 (No. 4), pp 225-238.

89. Mein, R.G. and Larson, C.L., (1971), "Modelling Infiltration During a Steady Rain", Water Resources Research, Vol. 9, No. 2, pp 384-394.

90. Mein, R.G. and O'Loughlin, G.G., (1985), "Application of ILLUDAS-SA to Gauged Catchments", Institution of Engineers, Australia, Hydrology and Water Resources Symposium, Sydney, Australia, pp 151-152.

91. Monro, J.C., (1971), "Direct Search Optimisation in Mathematical Modelling and a Watershed Model Application", National Oceanic Atmospheric Administration, National Weather Service, U.S. Department of Commerce, Technical Memo, NWS HYDRO 12, p 52.

92. Muncaster, S.H., Weinmann, P.E. and Mein, R.G., (1997), "An Application of Continuous Hydrologic Modelling to Design Flood Estimation", 24th Hydrology and Water Resources Symposium Proceeding, Auckland, pp 77-82.

93. Mutreja, K.N., (1990), Applied Hydrology, Tata McGraw-Hill Publishing Company Limited, New Delhi.

94. NAASRA (National Association of Australian State Road Authorities), (1986), Guide to the Design of Road Surface Drainage, Sydney.

95. Naidu, C. and Kearney, M.A., (1995), "Central Business District Catchment Study", The Second International Symposium on Urban Stormwater Management, I.E. Aust., Melbourne, pp 181-187.

96. Nandakumar, N., Mein, R.G. and Siriwardena, L., (1994), "Loss Modelling for Flood Estimation- A Review", Cooperative Research Centre for Catchment Hydrology, Australia, Report 94/4.

97. Nathan, R.J., (1990), "Low Flow Hydrology: Application of a Systems Approach", PhD Thesis, University of Melbourne.

98. O'Loughlin, G. and Goyen, A.G., (1990), "Computer Models for Urban Stormwater Drainage in Australia", Urban Planning and Stormwater management Proceedings, Kular Lumper, May 28-June 1, pp 485-494.

99. O'Loughlin, G. and Stack, B., (1998), DRAINS User's Manual, Watercom Pty Ltd, Sydney.

100. O'Loughlin, G., (1993), "The ILSAX Program for Urban Storm Water Drainage and Analysis", User's Manual for Microcomputer, Version 2.13, School of Civil Engineering, School of Civil Engineering, University of Technology, Sydney, Australia.

101. O'Loughlin, G., Stack, B. and Wilkinson, A, (1998), "Effects of Varying Time Steps in Urban Rainfall-Runoff Models", HydraStorm'98, Australia, pp 57-62.

102. O'Loughlin, E.M., Young, W.J. and Molloy, J.D., (1992b), Urban Stormwater: Impacts on the Environment. Australia, CSIRO Australia, Division of Water Resources.

103. O'Loughlin, G. and Avery, G.C., (1980), “Recurrence Intervals for Minor Urban Drainage Works”, Hydrology and Water Resources Symposium, Adelaide, pp 39-43.

104. O'Loughlin, G., Darlington, D. and House, D., (1992a), “Mathematical Description of Pit Entry Capacities”, International Symposium on Urban Stormwater Management, Sydney, pp 247-249.

105. O'Loughlin, G.G. and Joliffe, I.B., (1987), “Historical Overview of Urban Stormwater Drainage Hydraulics”, Conference on Hydraulics in Civil engineering, Melbourne, pp 82-87.

106. O'Loughlin, G.G., Haig, R.C., Attwater, K.B. and Clare, G.R., (1991), “Calibration of Stormwater Rainfall-Runoff Models”, International Hydrology and Water Resources Symposium, Perth, Australia, pp 60-65.

107. Papadakis, C.N. and Preul, H.C., (1972), “University of Cincinnati Urban Runoff Model”, Journal of the Hydraulic Division, Proceedings of the American Society of Civil Engineering, Vol 98, No. Hy10, pp 1319-1335.

108. Pezzaniti, D., Johnston and Argue, J.R., (1999), “Road Surface Stromwater Drainage Hydraulics: New Design Information”, 8th International Conference on Urban Storm Drainage, Sydney, Australia, pp 754-762.

109. Philip, J.R., (1957), “The Theory of Infiltration: The Infiltration Equation and Its Solution”, Soil Science, 77(2), pp 153-159.

110. Pilgrim, D.H. and Robinson, D.K., (1988), "Flood Estimation in Australia- Progress to the Present, Possibilities for the Future”, Civil Engineering Transactions (CE 30), pp 187-206.

111. Pilgrim, D.H., (1989), "Regional Methods for Estimation of Design Floods for Small to Medium Sized Drainage Basins in Australia", International Association of Hydrologic Science (IAHS)(181), pp 247-260.

112. Pisaniella, J.D., (1997), "Analysis and Modelling of private Dam Safety Assurance Policy and Flood Capability Design/ Review Procedures, PhD Thesis, School of Engineering, University of South Australia, Australia.

113. Priestley, S., Hollings, B.C. and Ferner Ltd, (1997), "Storage Model for Infiltration", 24th Hydrology and Water Resources Symposium Proceedings, pp 475-480.

114. Rahman, A., (1997), "Flood Estimation for Ungauged Catchments: A Regional Approach Using Flood and Catchment Characteristics", PhD Thesis, Department of Civil Engineering, Monash University, Australia.

115. Rao, A.R. and Han, J., (1981), "Optimal Parameter Estimation in urban Runoff Models", Second International Conference on Urban Storm Drainage, Urbana, Illinois USA, pp 384-393.

116. Robinson, D.K. and O'Loughlin, G.G., (1999), "A Critical Review of the Development of Stormwater Management Practice in New South Wales, Australia", 8^{th} International Conference on Urban Stormwater Drainage, Sydney, Australia, pp 1132-1141.

117. Sefe, F.T. and Boughton, W.C., (1982), "Variation of Model Parameter Values and Sensitivity with Type of Objective Function", Journal of Hydrology, New Zealand (21), pp 117-132.

118. Sorooshian, V. and Gupta, K., (1995), "Model Calibration" Computer Models of Watershed Hydrology, ed. V. P. Singh, Water Resources Publications, Littleton, Colo., pp 26-68.

119. Stephens, M.L. and Kuczera, G., (1999), "Testing the Time-Area Urban Runoff Model at the Allotment Scale", 8^{th} International Conference on Urban Storm Drainage, Sydney, Australia, pp 1391-1398.

120. Sueishi, T., (1963), "Runoff Estimation in Storm Sewer System Using Equivalent Roughness", Transaction of JSCE, No. 19, pp 41-54.

121. Sugawara, M., (1961), "On the Analysis of Runoff Structures about Several Japanese Rivers", Japanese Journal of Geophysics, pp 2-4.

122. Surveying and Engineering Software, (1997), CIVILCAD, User Manual, Volume 2, Chapter 8.

123. Terstriep, M.L. and Stall, J.B., (1969), "Urban Runoff by Road Research Laboratory Method", Journal of Hydraulic Division, ASCE, Vol. 95, No. Hy 6, pp 1809-1831.

124. Terstriep, M.L. and Stall, J.B., (1974), The Illinois Urban Drainage Area Simulator ILLUDAS, Bulletin 58, Illinois State Water Survey, Urbana.

125. The Institution of Engineers, Australia, (1958), Australian Rainfall and Runoff.

126. The Institution of Engineers, Australia, (1977), Australian Rainfall and Runoff.

127. The Institution of Engineers, Australia, (1987), Australian Rainfall and Runoff.

128. Tholin, A.L. and Kiefer, C.J., (1960), "The Hydrograph of Urban Runoff", Transactions, ASCE, Vol 125, Paper No 3061, pp 1308-1355.

129. U.S. Army Corps of Engineers, Hydrologic Engineering Centre, (1977), Storage, Treatment, Overflow Runoff Model (STORM), User's Manual, Davis, Canada.

130. U.S. Environmental Protection Agency (USEPA), (1992), Stormwater Management Model, Version 4, User's Manual, Environmental Research Laboratory, Office of Research and Development, Athens, Georgia.

131. U.S. Soil Conservation Service, (1985), National Engineering Hand Book, Section 4, Hydrology, U.S. Department of Agriculture, Washington, D.C.

132. U.S. Soil Conservation Service, (1986), Urban Hydrology for Small Watersheds, Technical Release No 55, U.S. Department of Agriculture, Washington, D.C.

133. Vale, D.R., Attwater, K.B. and O'Loughlin, G.G., (1986), "Application of SWMM to Two Urban Catchments in Sydney", Institution of Engineers, Australia, Hydrology and Water Resources Symposium, Brisbane, Australia, pp 268-272.

134. VicRoads, Australia, (1995), "Road Design Guidelines", Part 7, Drainage.

135. Watermark Numerical Computing (Australia), (1998), PEST (Model Independent Parameter Estimation), User's Manual.

136. Watson, M.D., (1981), “Time-Area Method of Flood Estimation for Small Catchments”, Report 7/81, Hydrological Research Unit, University of the Witwatersrand, Johanesburg.

137. Wenzel, H.G. and Voorhees, M.L., (1980), “Adoption of ILLUDAS for Continuous Simulation”, Journal of the Hydraulics Division, ASCE, Vol 106, No. HY11, pp 1795-1811.

138. White, M.J. and Cattell, F.C.R., (1992), "AUSQUAL, an Australian Water Quality Management Model", International Symposium on Urban Storm Water Management Sydney, pp 396-401.

139. WP Software, (1991), “RAFTS-XP (Runoff Analysis and Flow Training Simulation)”, Version 2.7, User Manual, Australia.

140. WP Software, (1992), “RatHGL-XP”, User Manual, Australia.

141. WP Software, (1993), SWMM-XP User’s Manual, Australia.

142. WP Software, (1994), AQUALM-XP, ACT, Australia.

143. XP Software, (2000a), XP-UDD2000, Technical Description and PowerPoint Presentation, Internet, http://www.xpsoftware.com.au/hydraulic.htm.

144. XP Software, (2000b), XP-SWMM, Technical Description and PowerPoint Presentation, Internet, http://www.xpsoftware.com.au/hydraulic.htm.

145. Zaghloul, N.A., (1981), “SWMM Model and Level of Discretization”, Journal of the Hydraulics Division, ASCE, Vol 107, No HY11, pp 1535-1545.

146. Zech, Y.; Sillen, X; Debources, C. and Hauwaert, A.V., (1994), “Rainfall-Runoff Modelling of Partly Urbanised Watersheds: Comparison between a Distributed Model using GIS and Other Models Sensitivity Analysis”, Water Science and Technology, Vol. 29, No. 1-2, pp 163-170.

APPENDIX A

CUSTOMER SURVEY QUESTIONNAIRE

QUESTIONNAIRE
Urban Stormwater Drainage Design and Analysis Practice in Victoria

Department of Civil and Building Engineering
Victoria University of Technology

NAME OF AUTHORITY: ....

1. Is your authority responsible for design and/or analysis (i.e. performance of the system under different storm conditions) of urban stormwater drainage systems?

   YES ☐ NO ☐

   If NO, please go to question 10.

2. Which of the following method(s) has your authority used?
   (Tick appropriate box(es))

   a) Rational formula (Runoff coefficient method) ☐

   b) ILSAX model ☐

   c) SWMM ☐

   d) RAFTS-XP ☐

   e) Any other method(s) (give a brief description, and reference if available)

   ____
   ____
   ____
   ____
   ____
   ____
   ____
   __

3. Specify the reason(s) for using the selected methods above.

4. For methods in 2, how do you estimate model parameters?

a) Use of pre-prepared maps or charts (only for Rational formula)
   - ARR 87 ☐
   - ARR 77 ☐

b) Use of pre-prepared maps or charts for other methods (please specify) ☐

c) Model calibration ☐

d) From a nearby catchment ☐

e) Physical field measurement ☐

f) Use of default values given in manuals ☐

g) Any other method (please specify):

5. How do you get rainfall information for design and analysis?

a) From ARR 87
   i) IFD information ☐
   ii) Temporal pattern ☐

b) Other methods (please specify)

6. What return period (average recurrence interval) do you use for?

a) design ☐
b) analysis ☐

7. Have the procedures or models adopted by you been tested using independent data sets?

YES ☐ NO ☐

If YES, are the results available to us for assessment?

YES ☐ NO ☐

8. Have you encountered any difficulties in applying model/procedure and selecting model parameters (please specify)?

YES ☐ NO ☐

9. Please comment on the adequacy of the current tools (i.e. software and guidelines)

10. Do you consider water quality control in your drainage systems (please specify)?

11. Who should we contact for further information? (Block letters, please.)

Name:

Position in Organisation:

Address:

Telephone No:

Fax:

E-mail:

Thank you for your help. Please return this form to:

Dr Chris Perera, Senior Lecturer
Department of Civil and Building Engineering
Victoria University of Technology
PO Box 14428 MMC
Melbourne, Vic 8001

Ph: (03) 9688 4729
Fax: (03) 9688 4096

Information supplied in this survey will not be released to any individual or company.

Please provide additional relevant details on a separate sheet.

APPENDIX B

CATCHMENT PLANS

KEY FOR CATCHMENT PLANS

Catchment boundary

Subcatchment boundary

Main drainage system

Secondary drainage system

Flow meter

Pluviometer

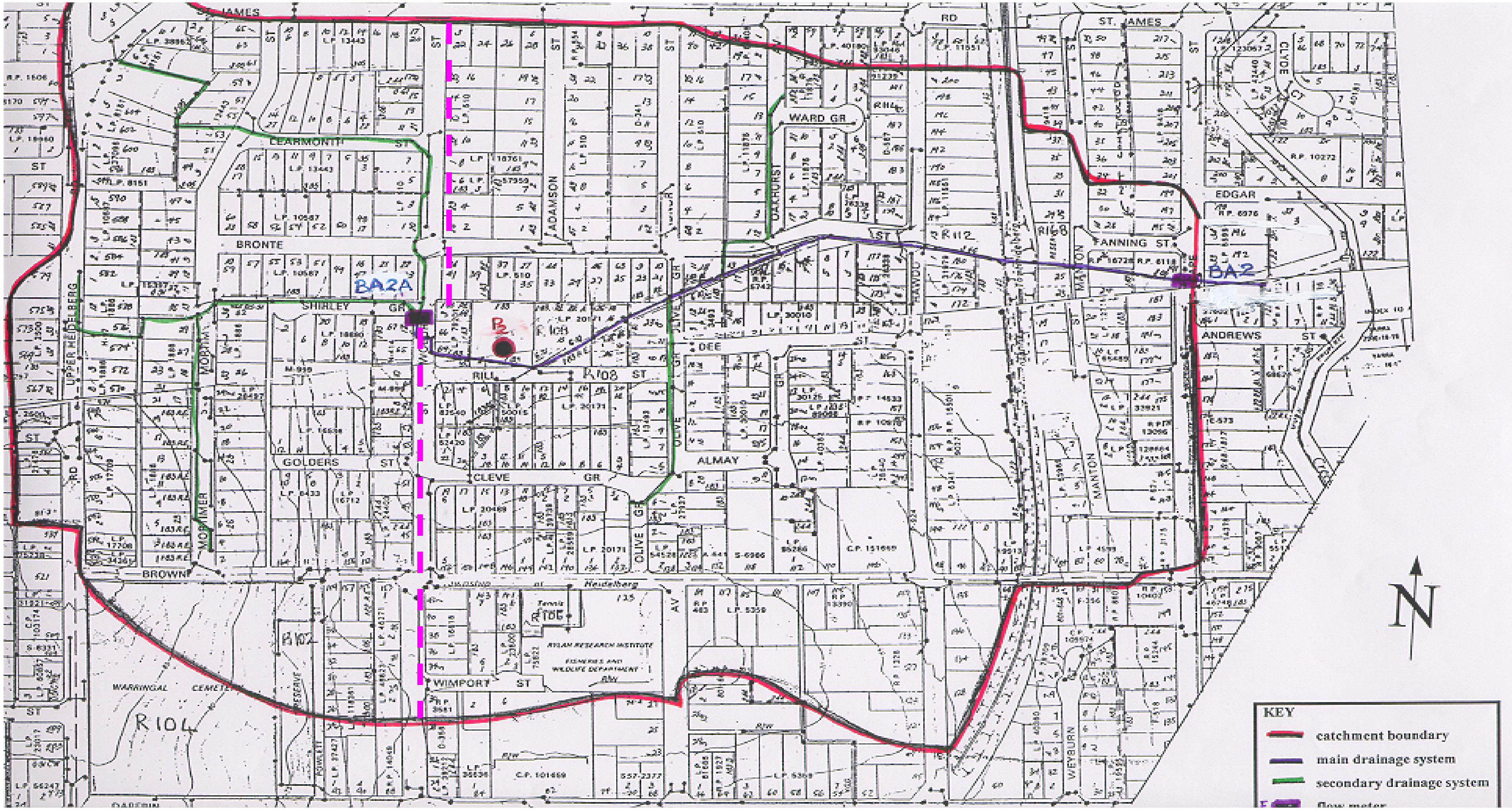


Figure B.1: Catchments BA2 and BA2A in Banyule City Council

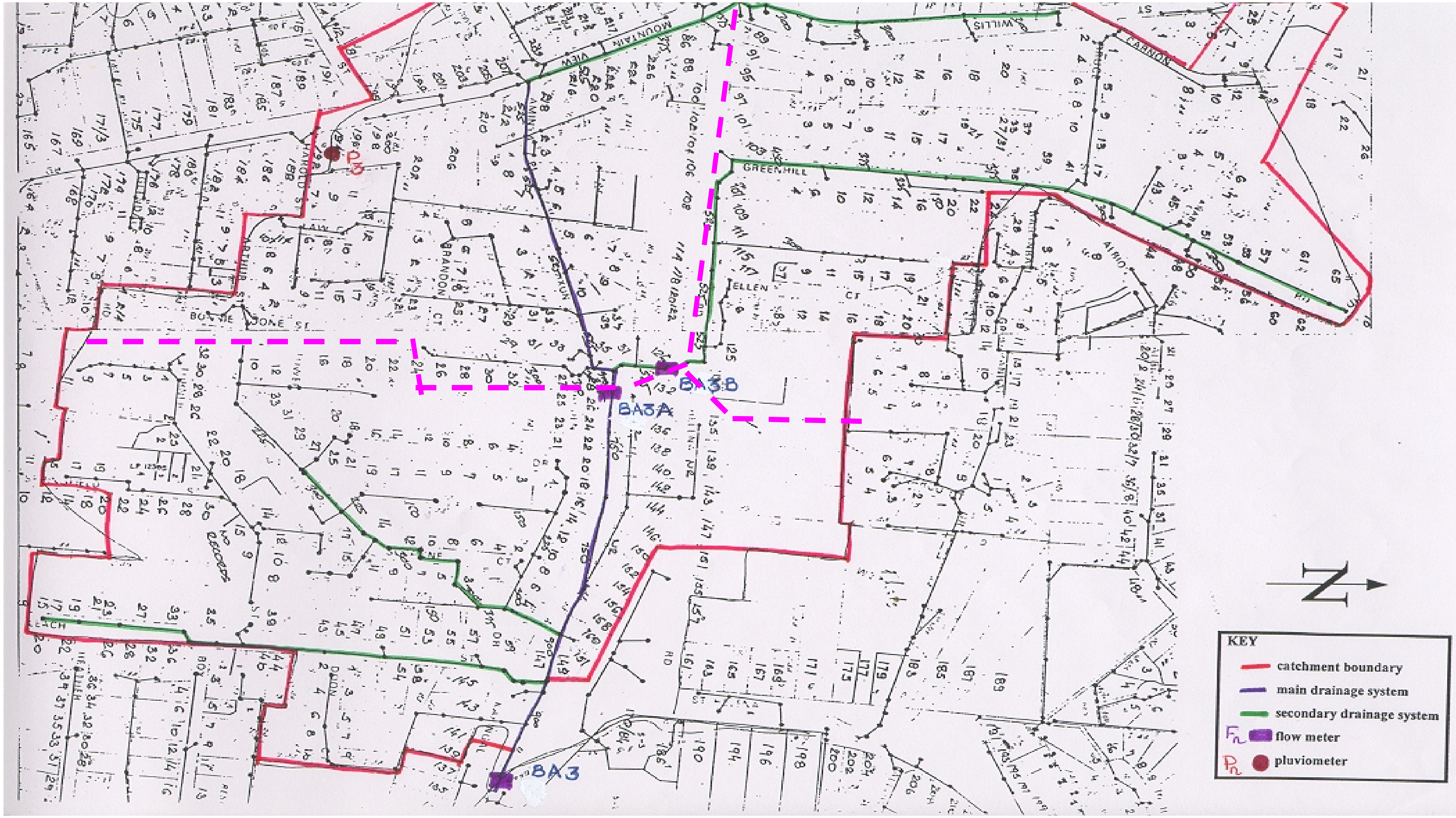


Figure B.2: Catchments BA3, BA3A and BA3B in Banyule City Council

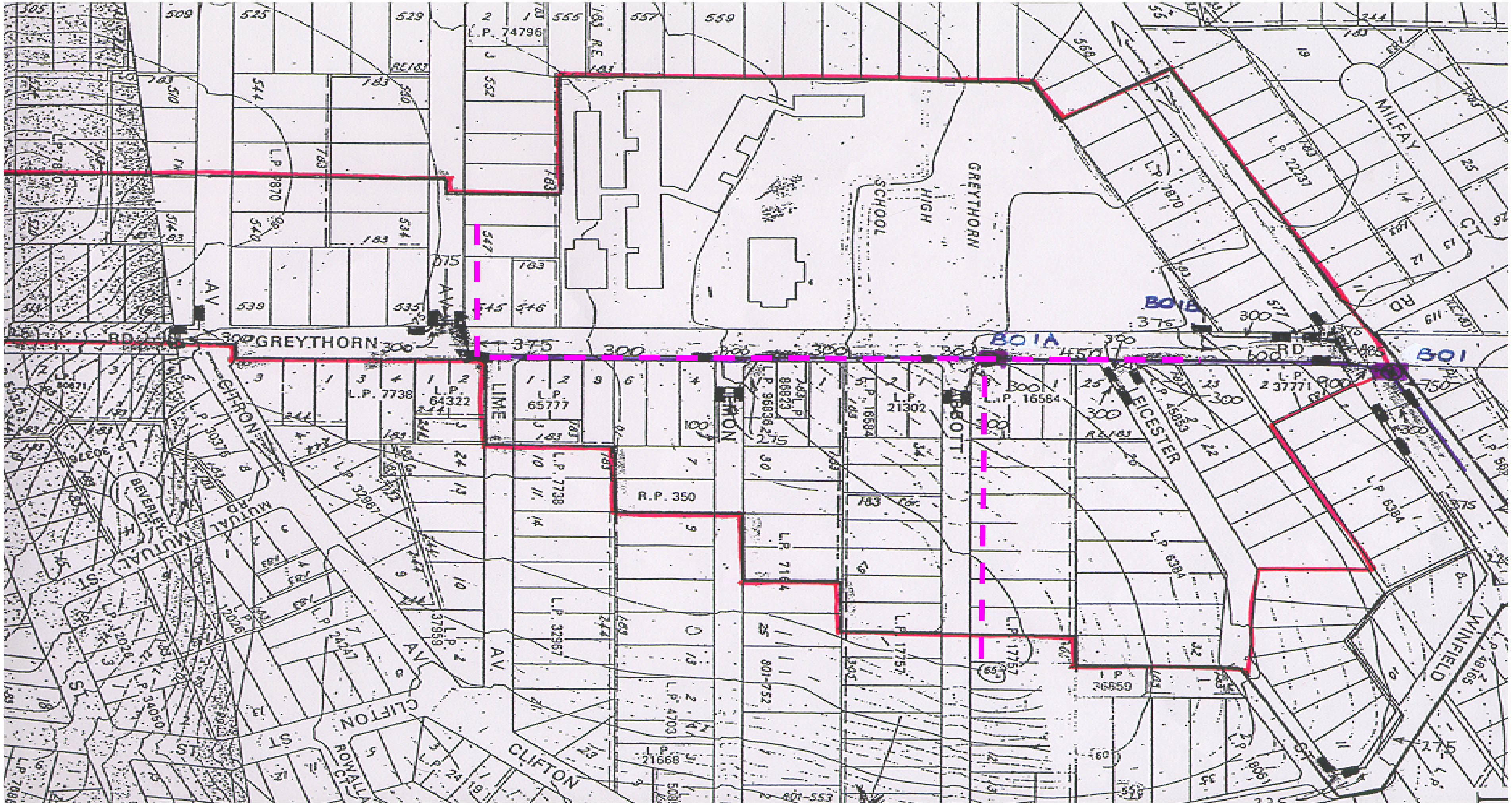


Figure B.3: Catchments BO1, BO1A and BO1B in Boroondara City Council

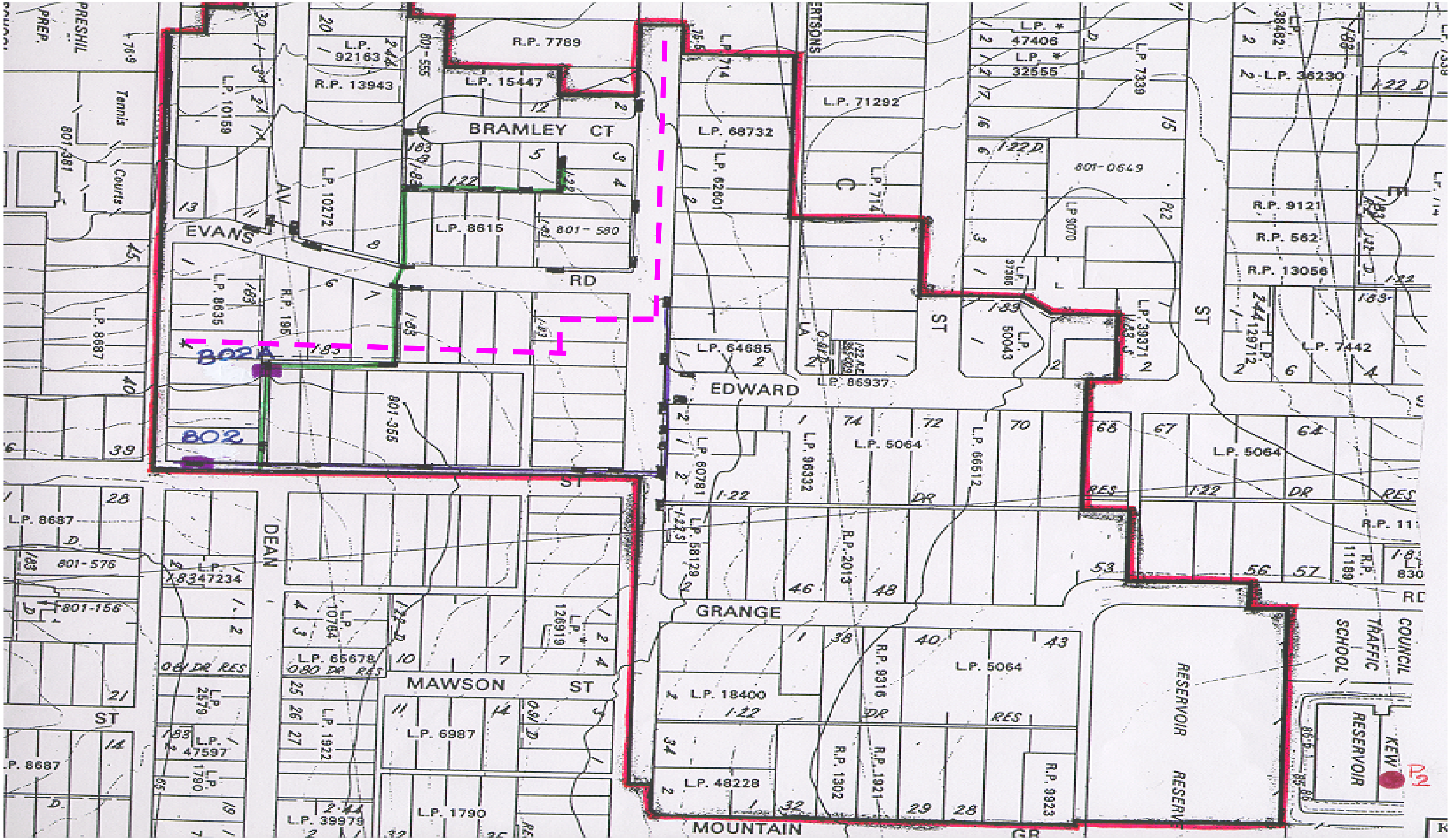


Figure B.4: Catchments BO2 and BO2A in Boroondara City Council

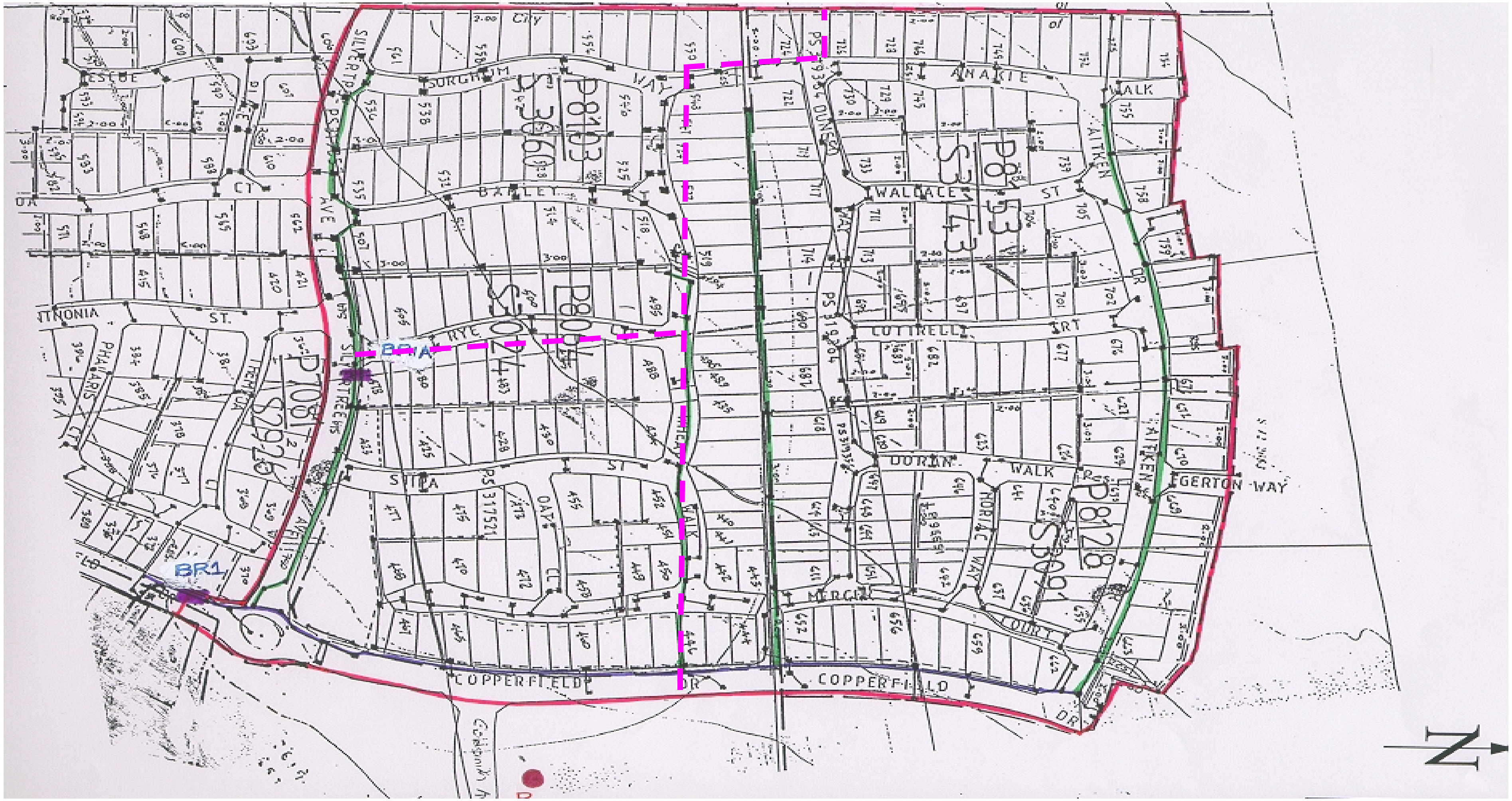


Figure B.5: Catchments BR1 and BR1A in Brimbank City Council

Figure B.6: Catchments BR2 and BR2A in Brimbank City Council

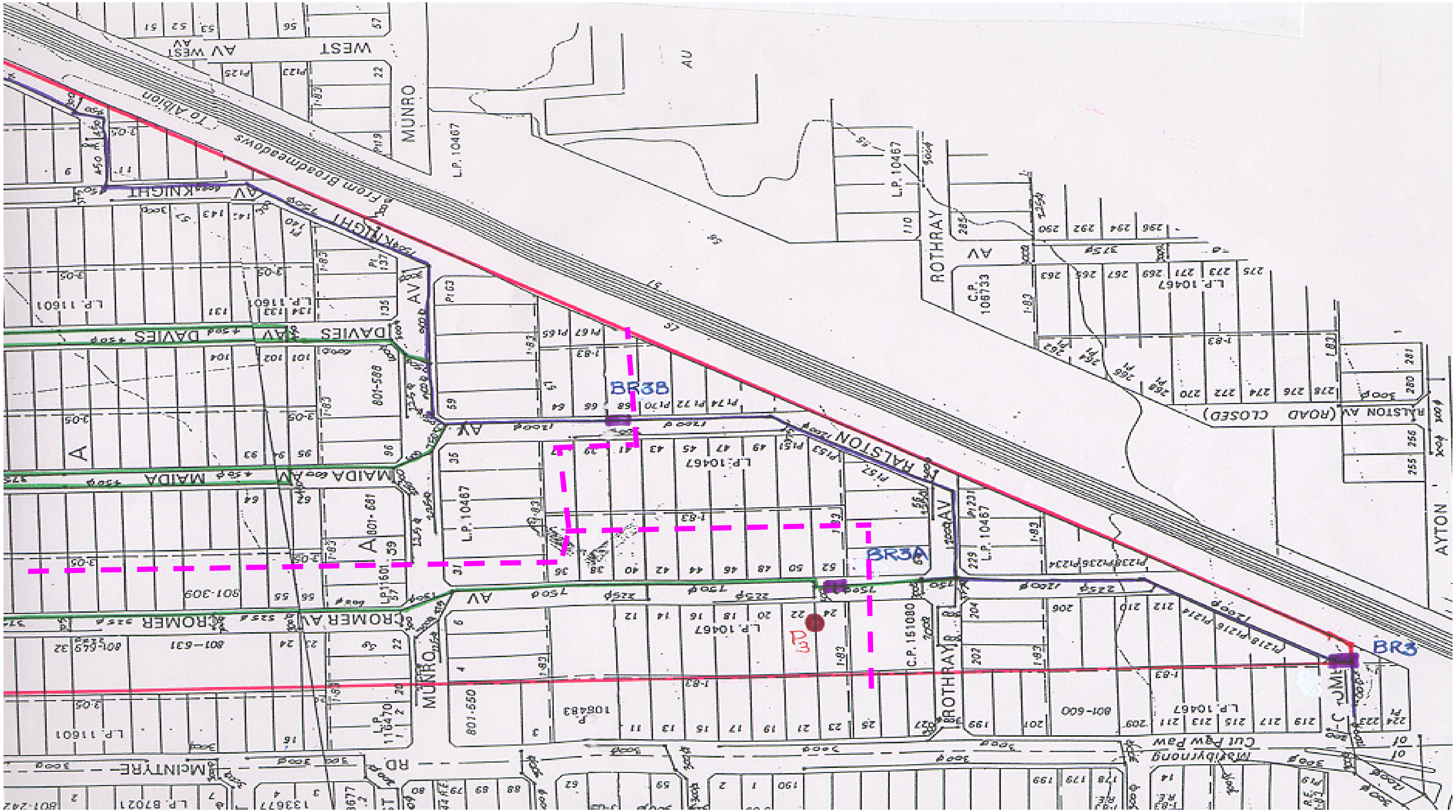


Figure B.7: Catchments BR3, BR3A and BR3B in Brimbank City Council

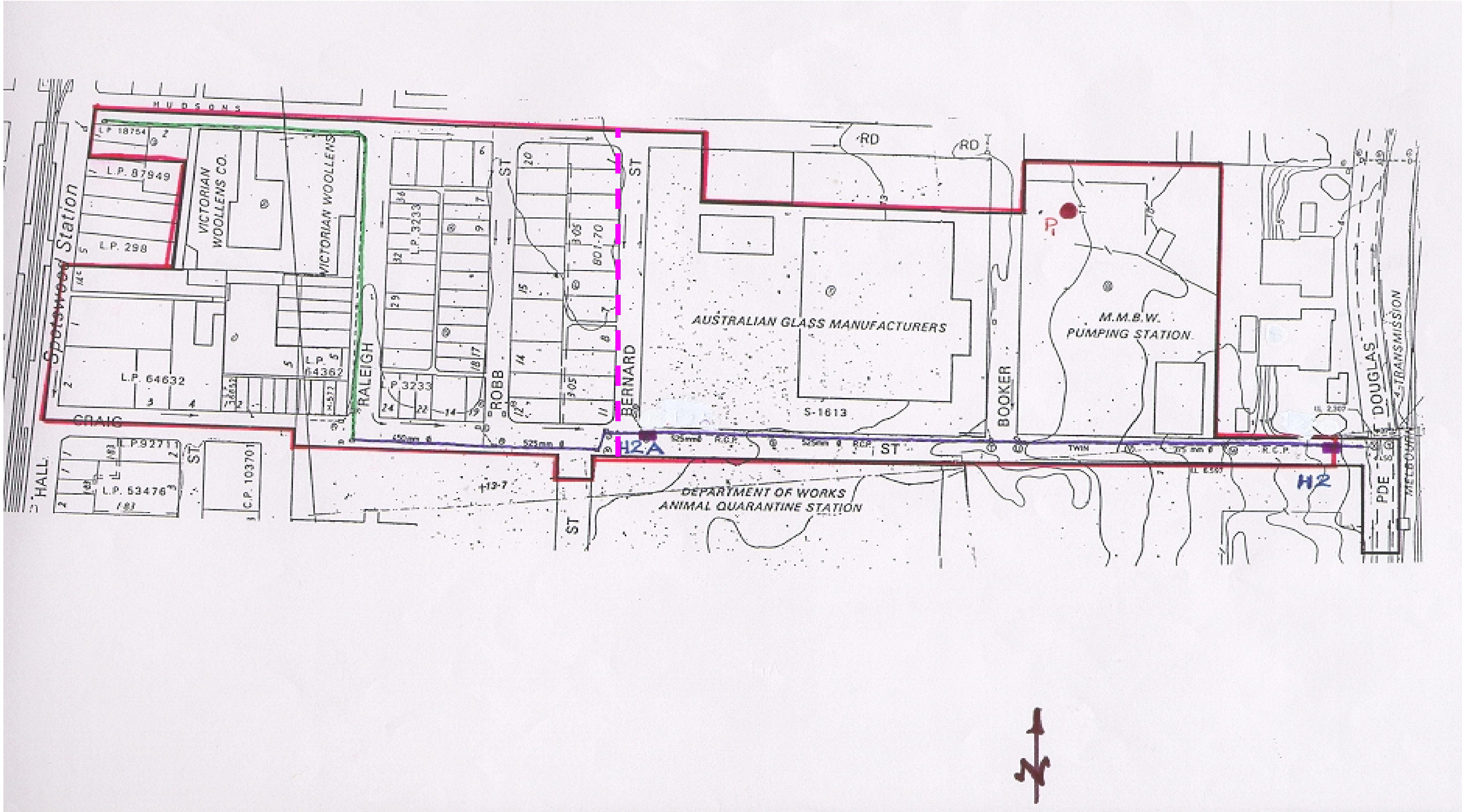


Figure B.8: Catchments H2 and H2A in Hobsons Bay City Council

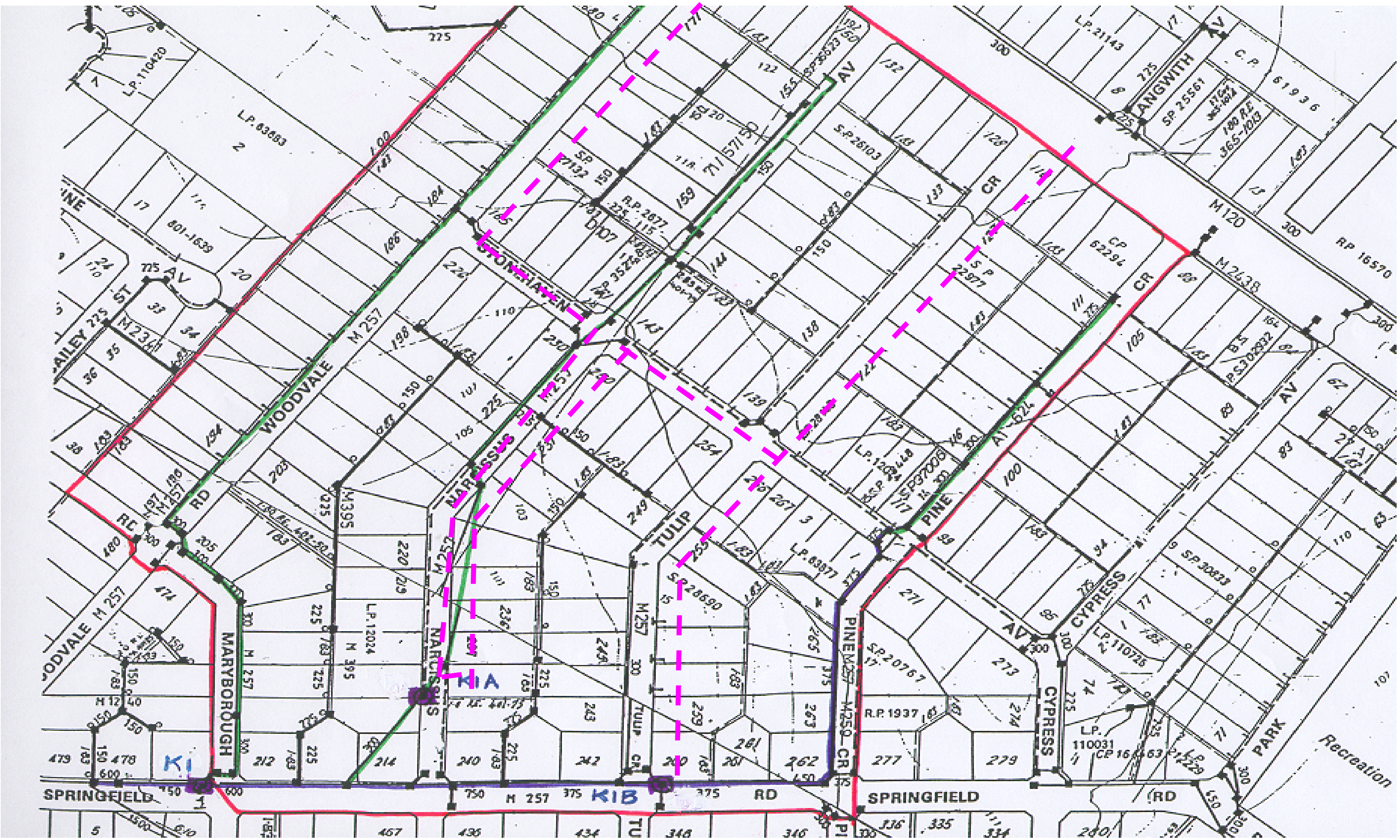


Figure B.9: Catchments K1, K1A and K1B in Knox City Council

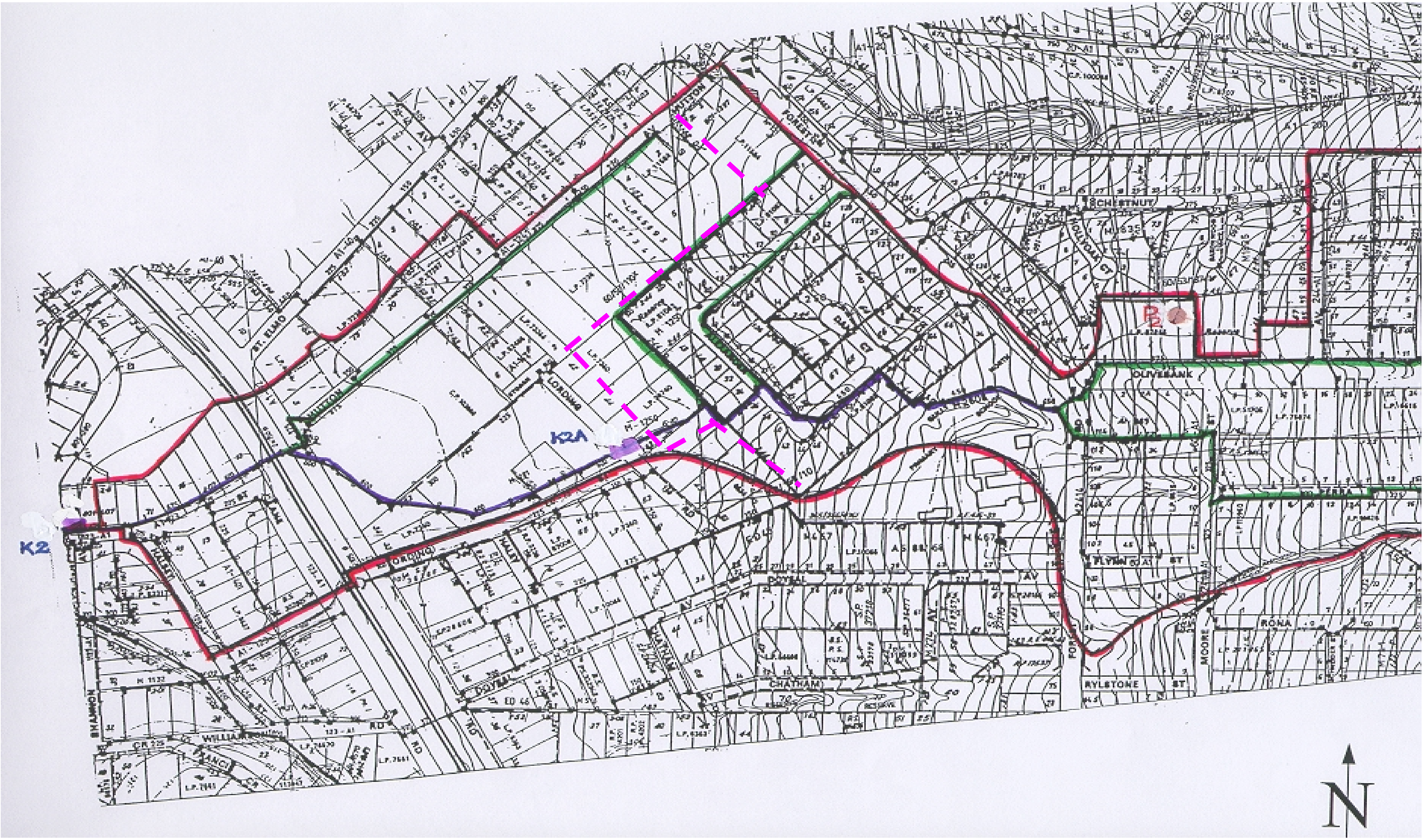


Figure B.10: Catchments K2 and K2A in Knox City Council

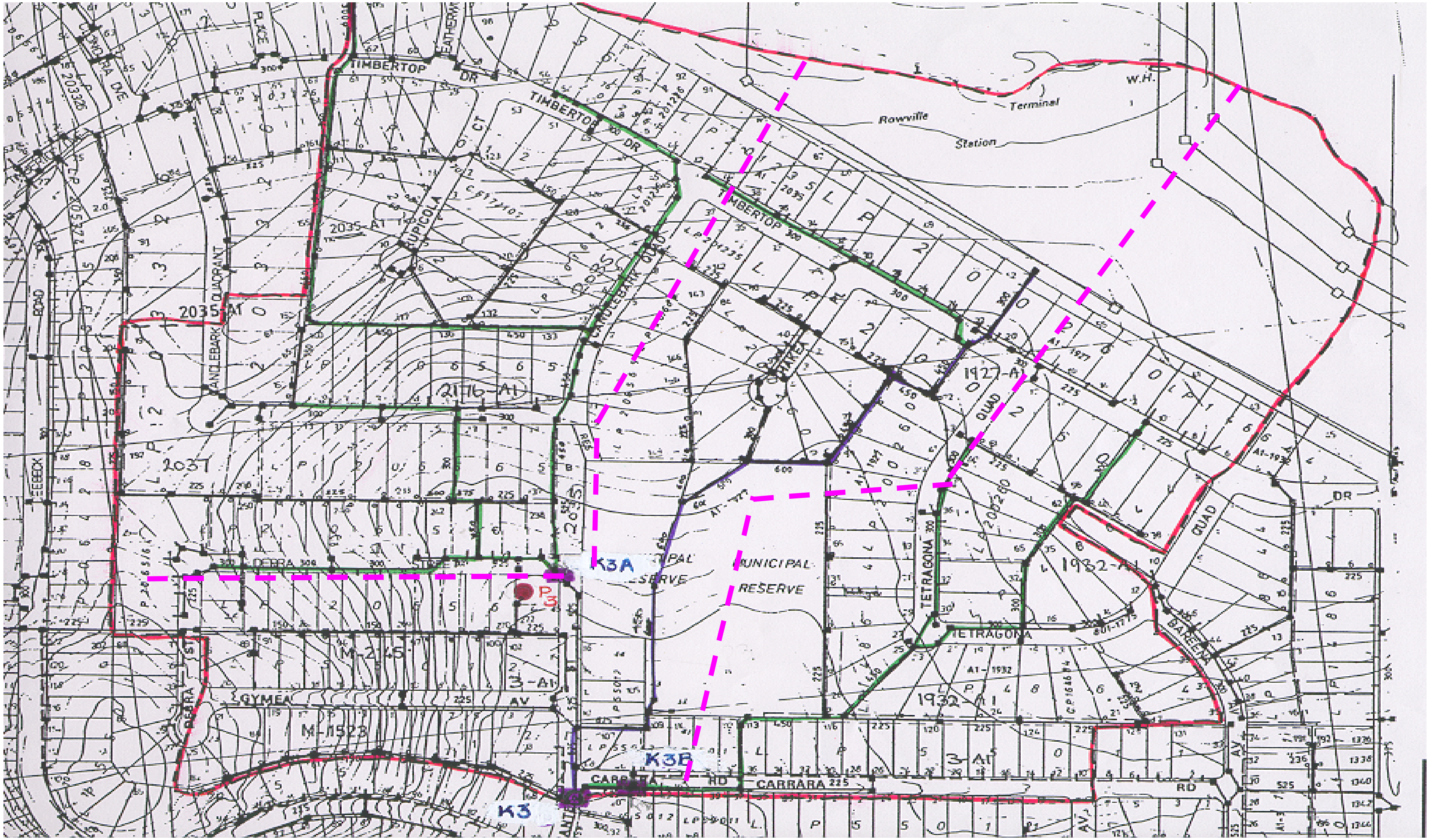


Figure B.11: Catchments K3, K3A and K3B in Knox City Council

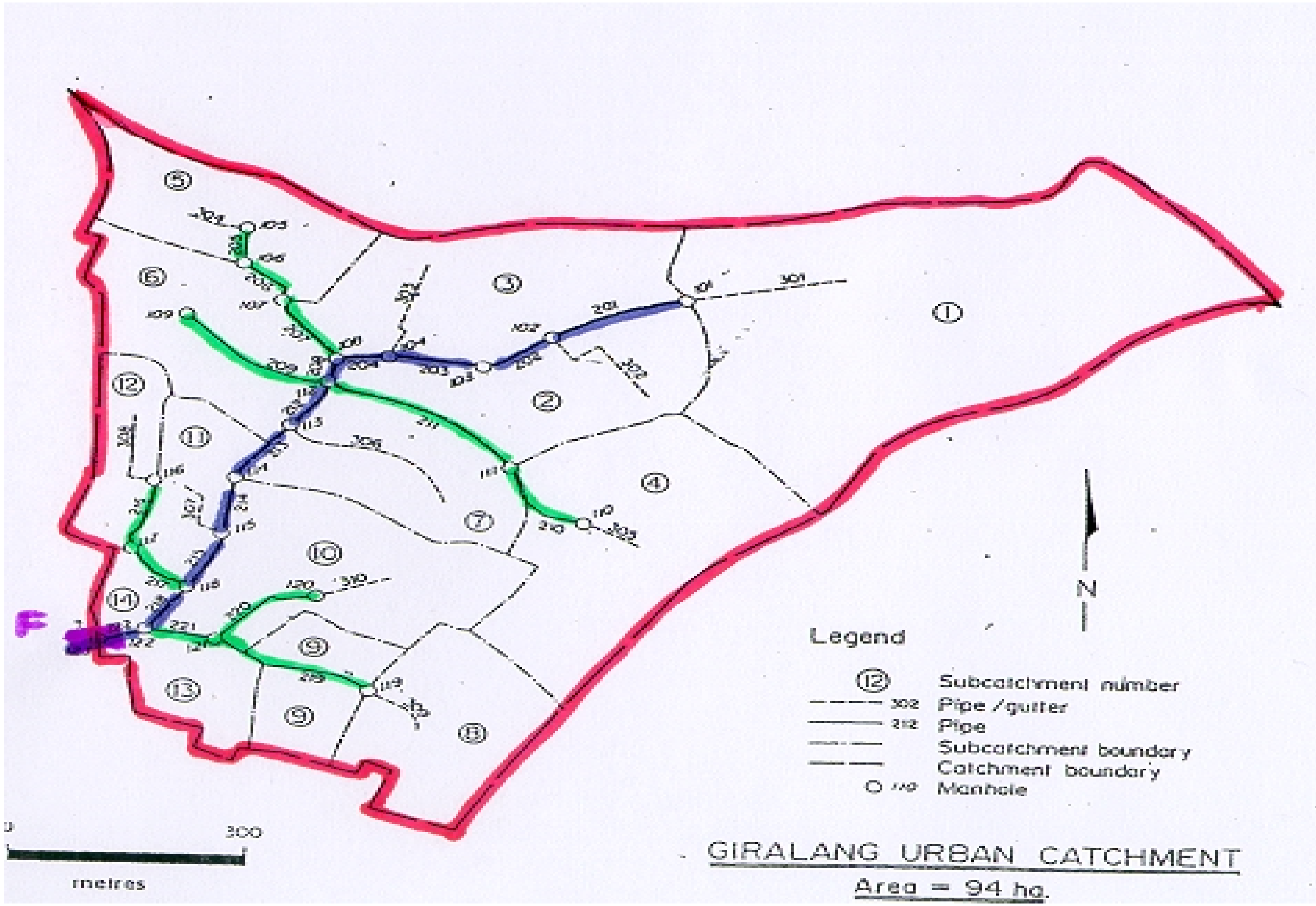


Figure B.12: Giralang (GI) Catchment in Canberra

APPENDIX C

RUNOFF DEPTH VERSUS RAINFALL DEPTH (RR) PLOTS

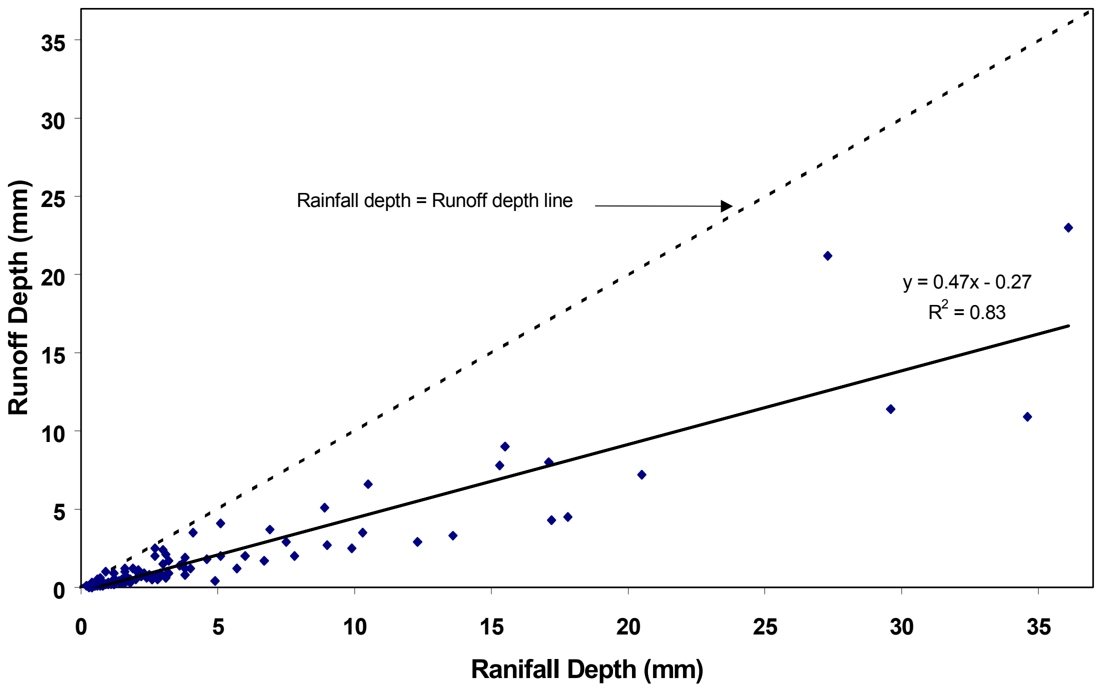


Figure C.1: RR Plot of Catchment BA2

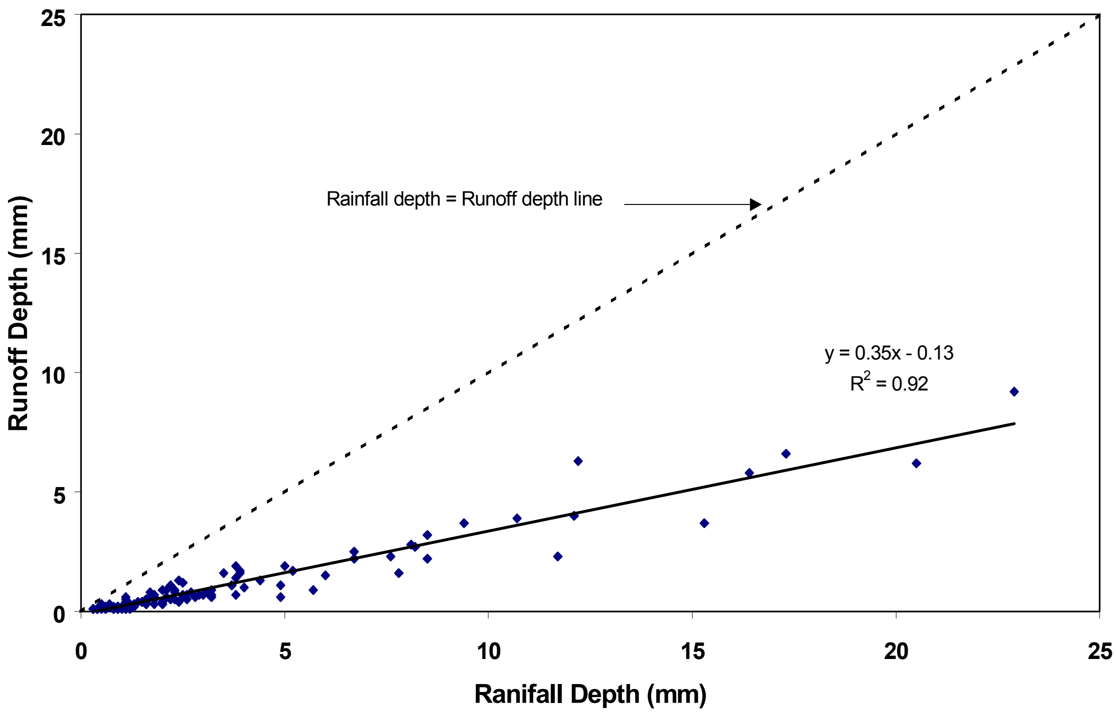


Figure C.2: RR Plot of Catchment BA2A

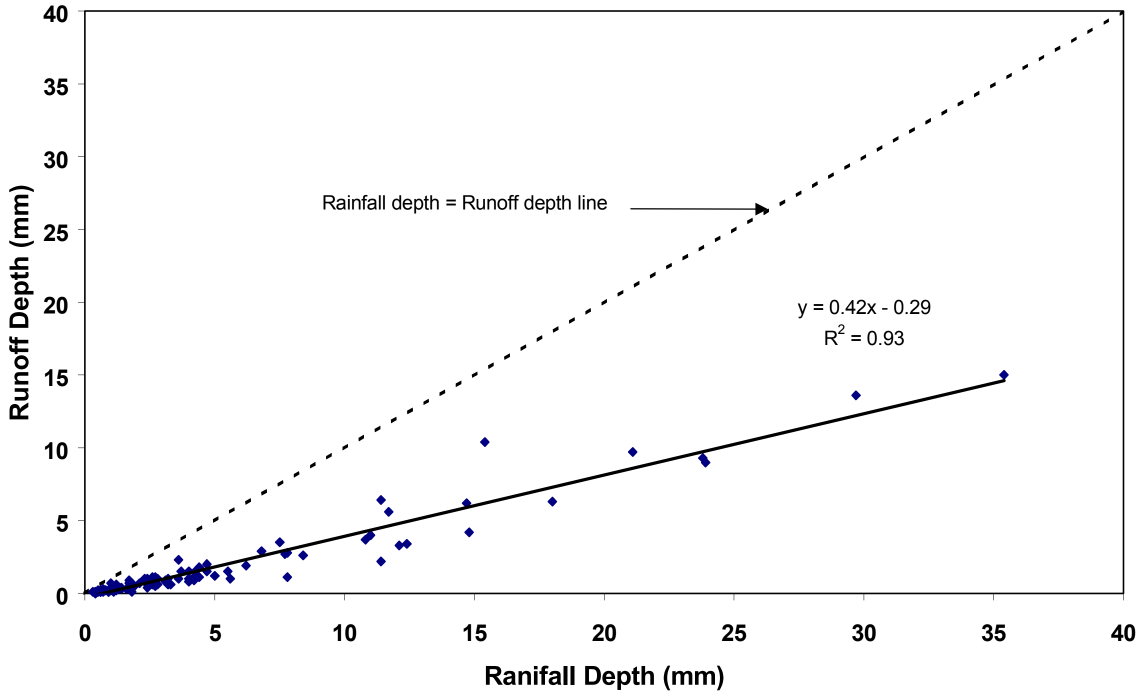


Figure C.3: RR Plot of Catchment BA3

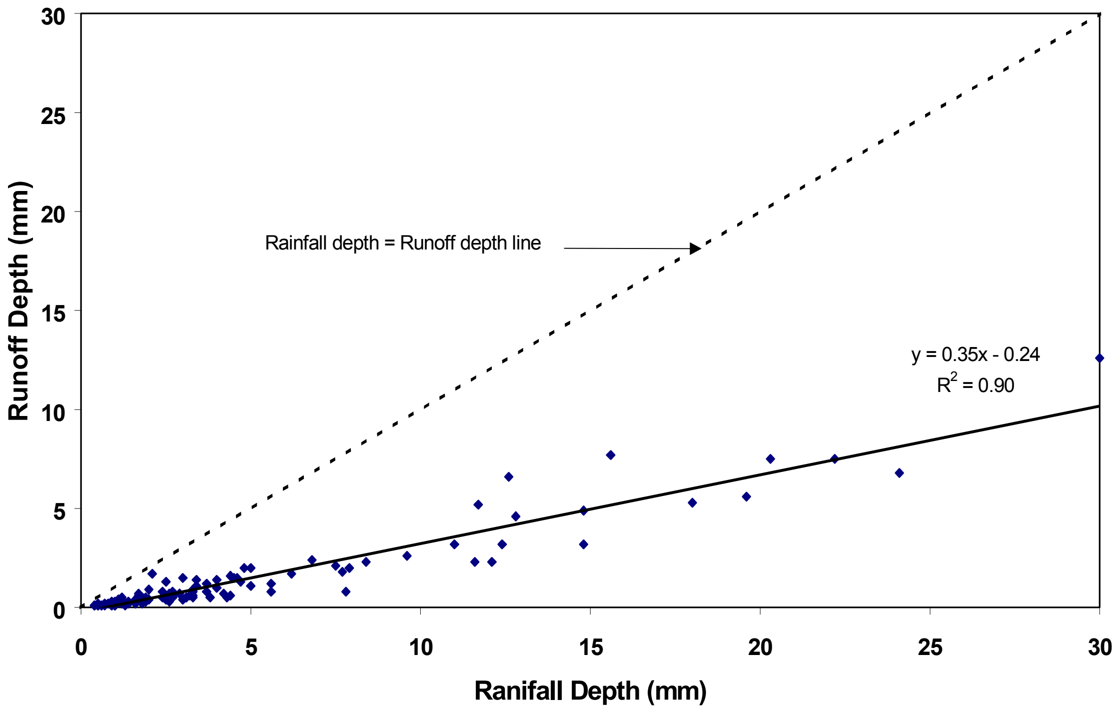


Figure C.4: RR Plot of Catchment BA3A

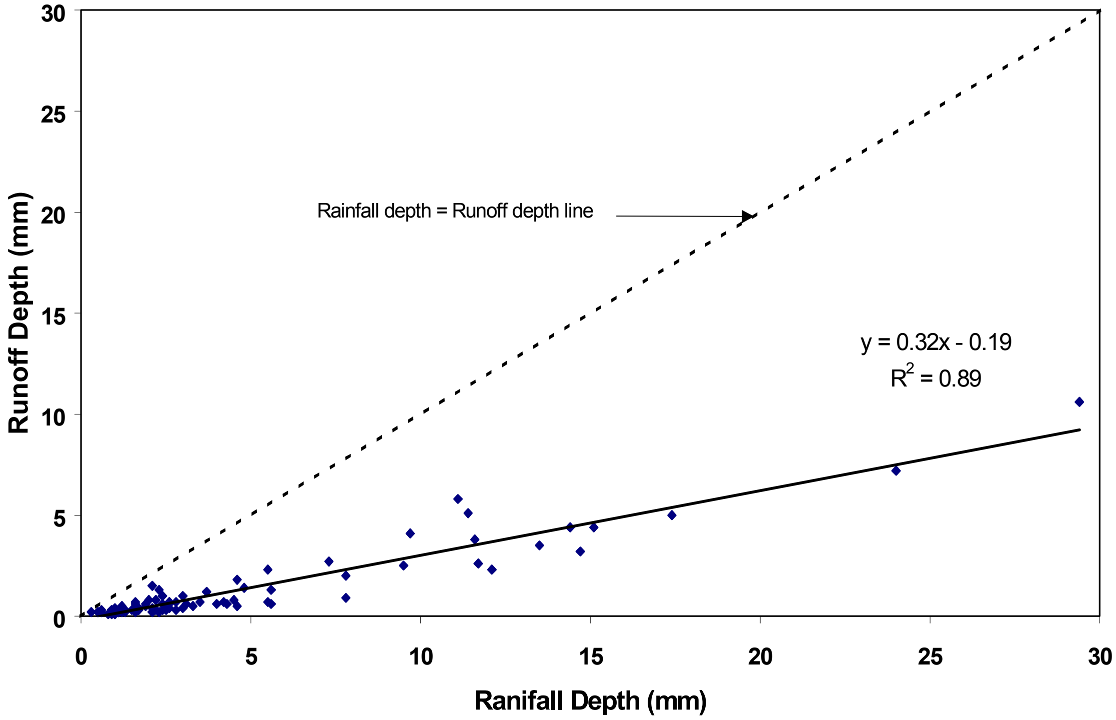


Figure C.5: RR Plot of Catchment BA3B

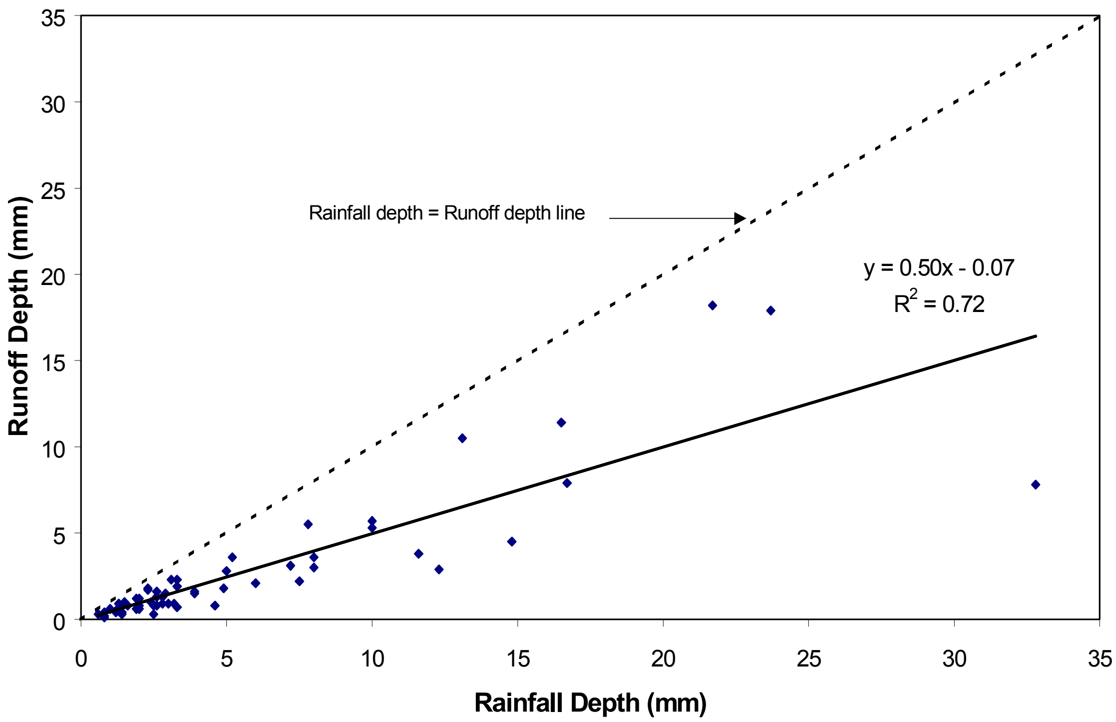


Figure C.6: RR Plot of Catchment BO1A

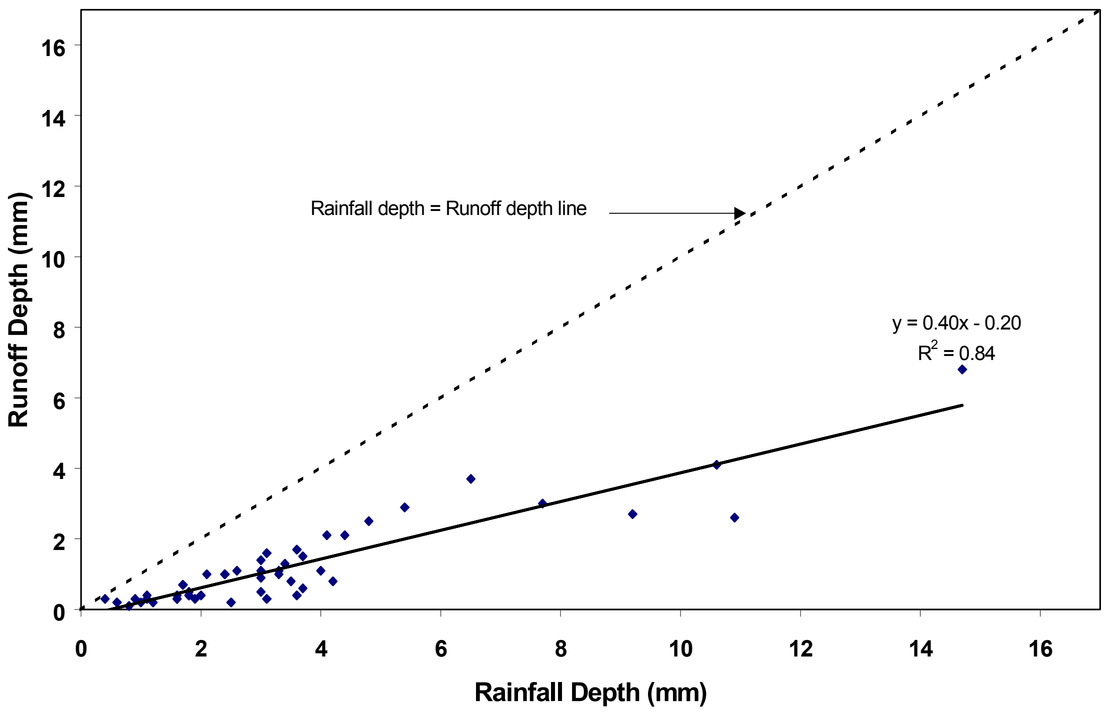


Figure C.7: RR Plot of Catchment BO2A

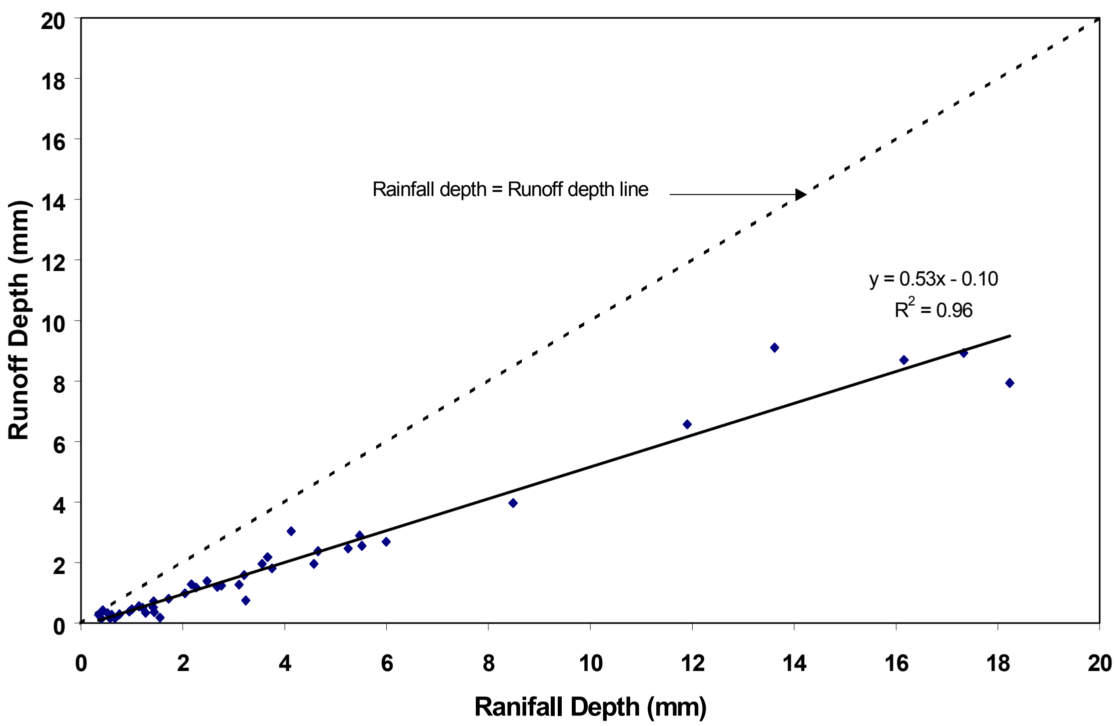


Figure C.8: RR Plot of Catchment BR1

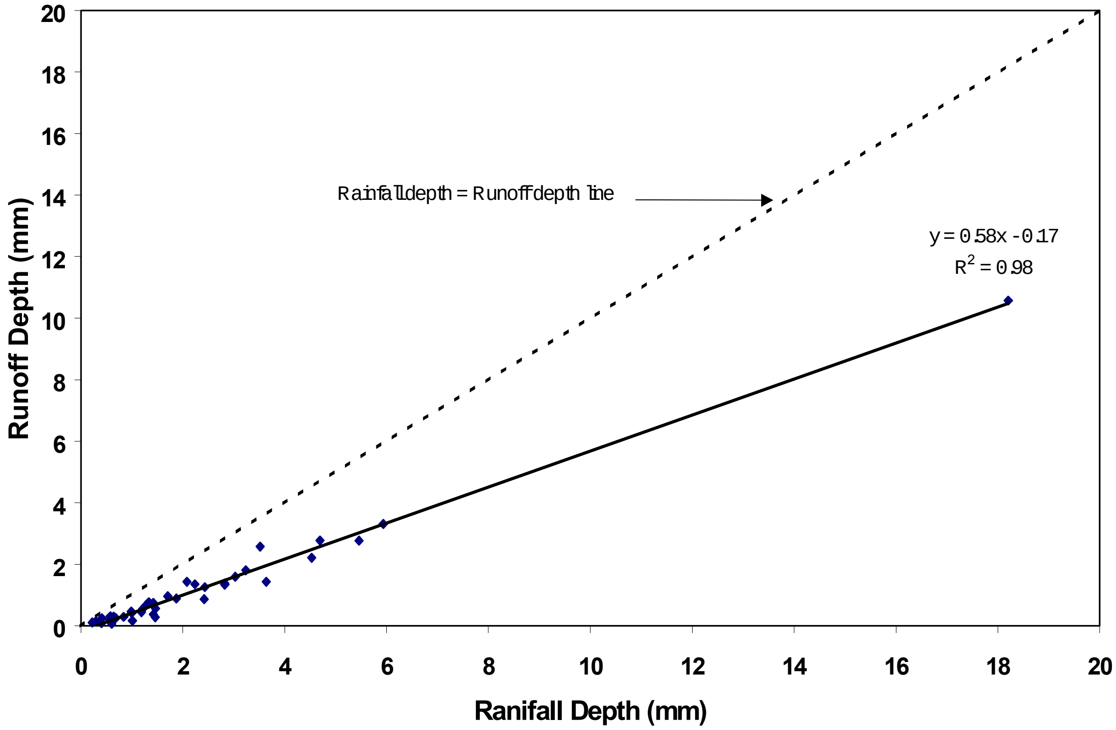


Figure C.9: RR Plot of Catchment BR1A

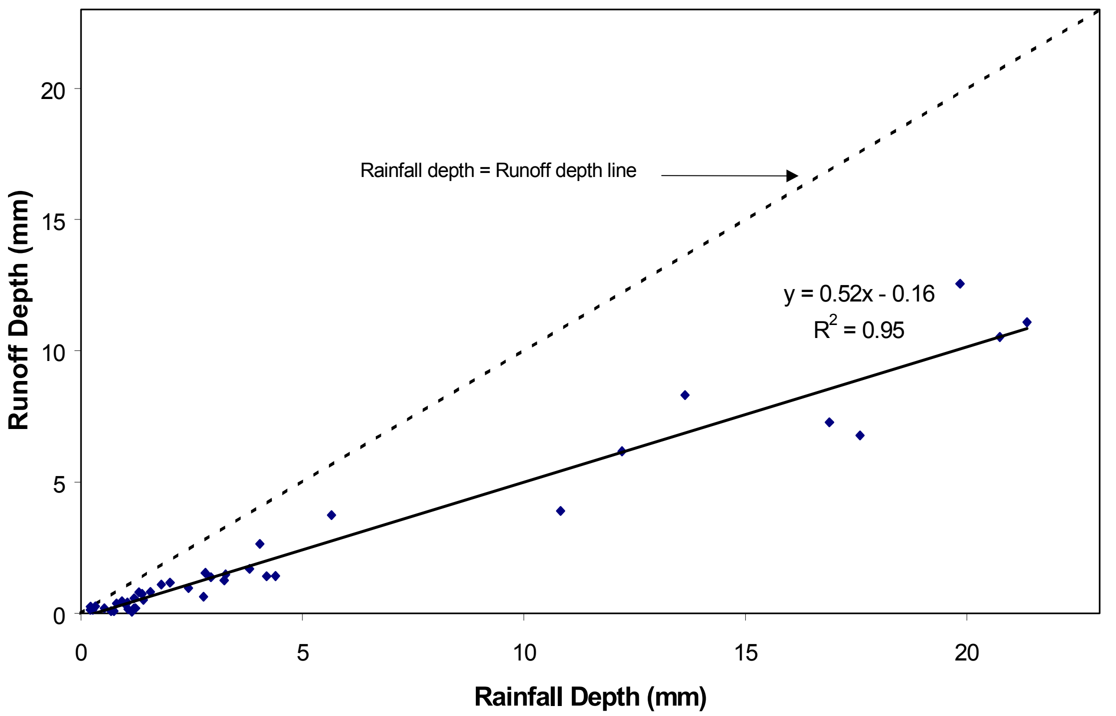


Figure C.10: RR Plot of Catchment BR2

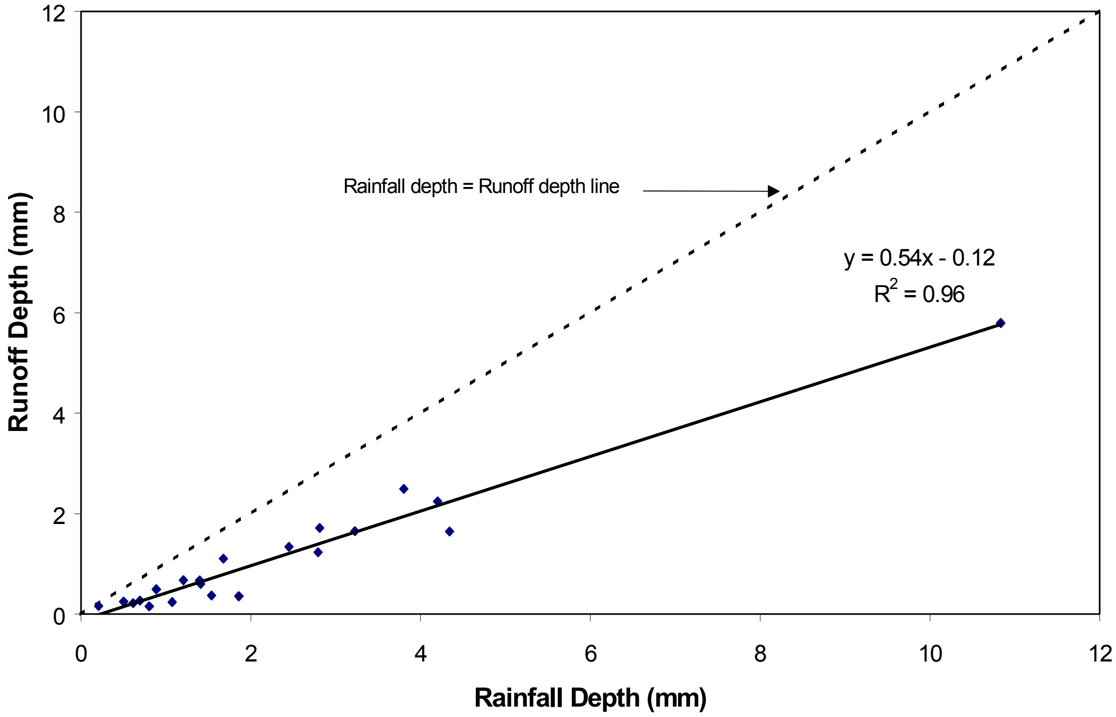


Figure C.11: RR Plot of Catchment BR2A

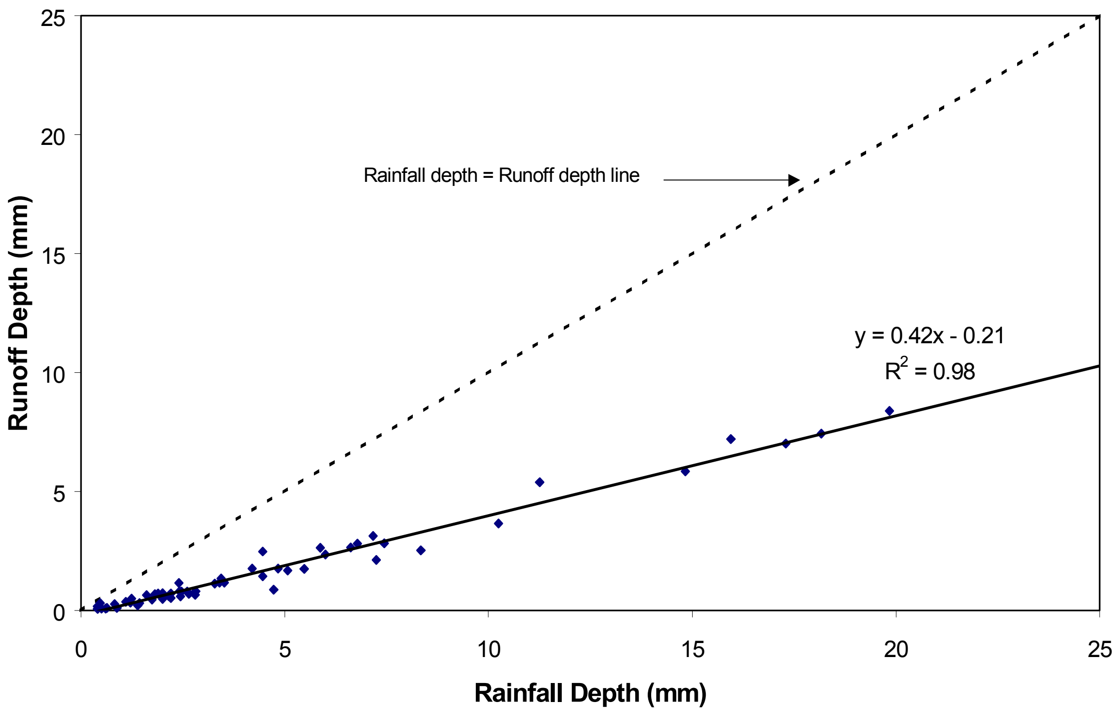


Figure C.12: RR Plot of Catchment BR3

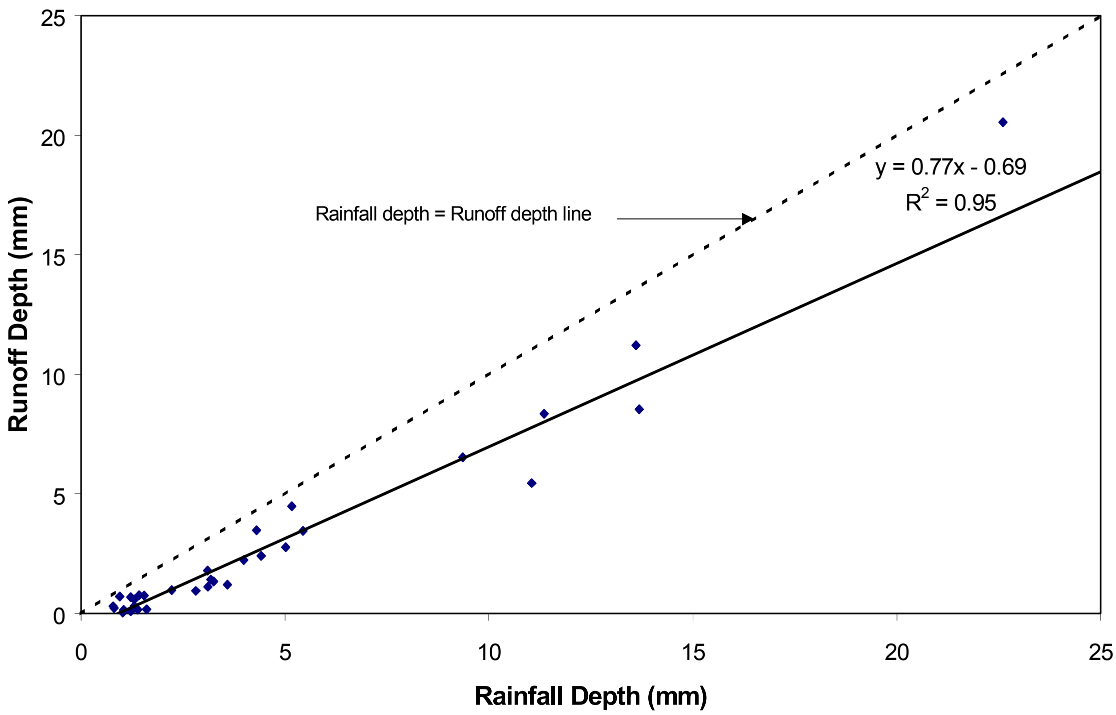


Figure C.13: RR Plot of Catchment H2

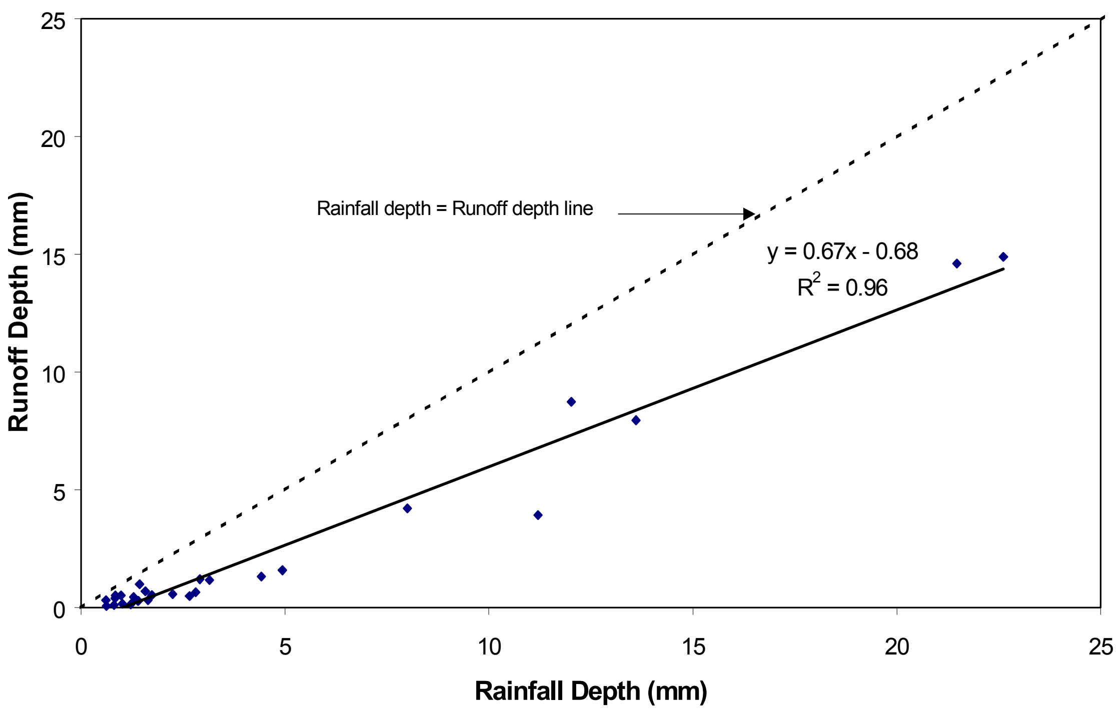


Figure C.14: RR Plot of Catchment H2A

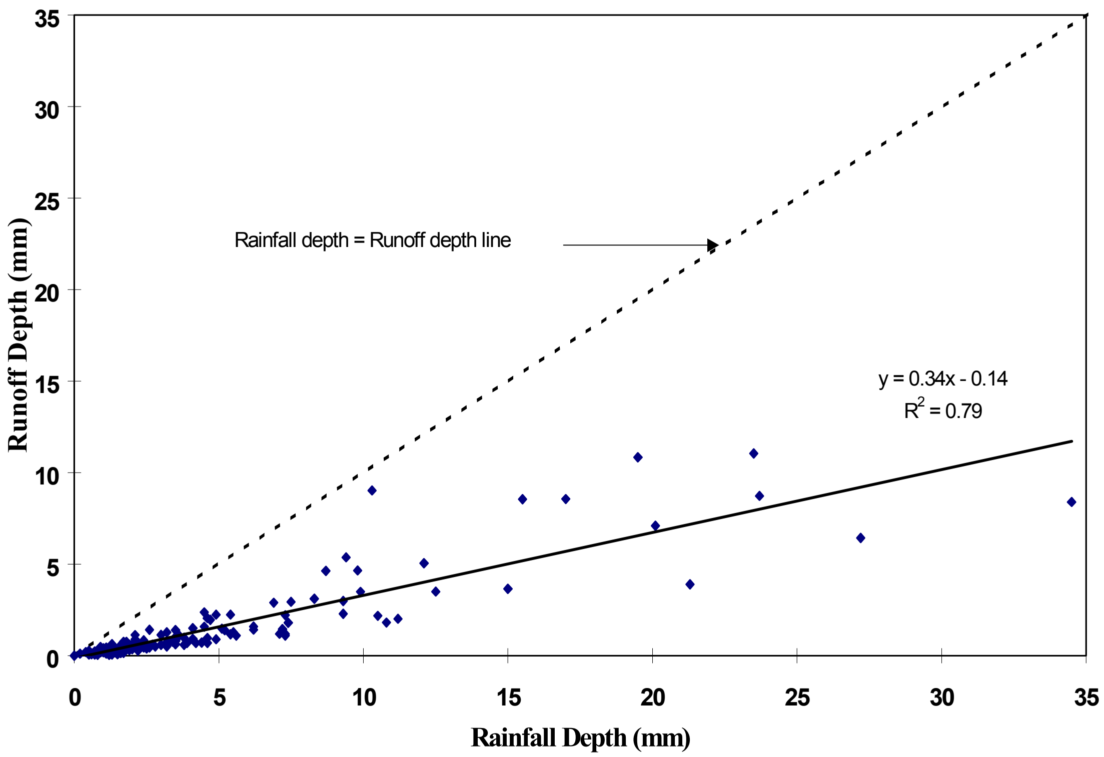


Figure C.15: RR Plot of Catchment K1

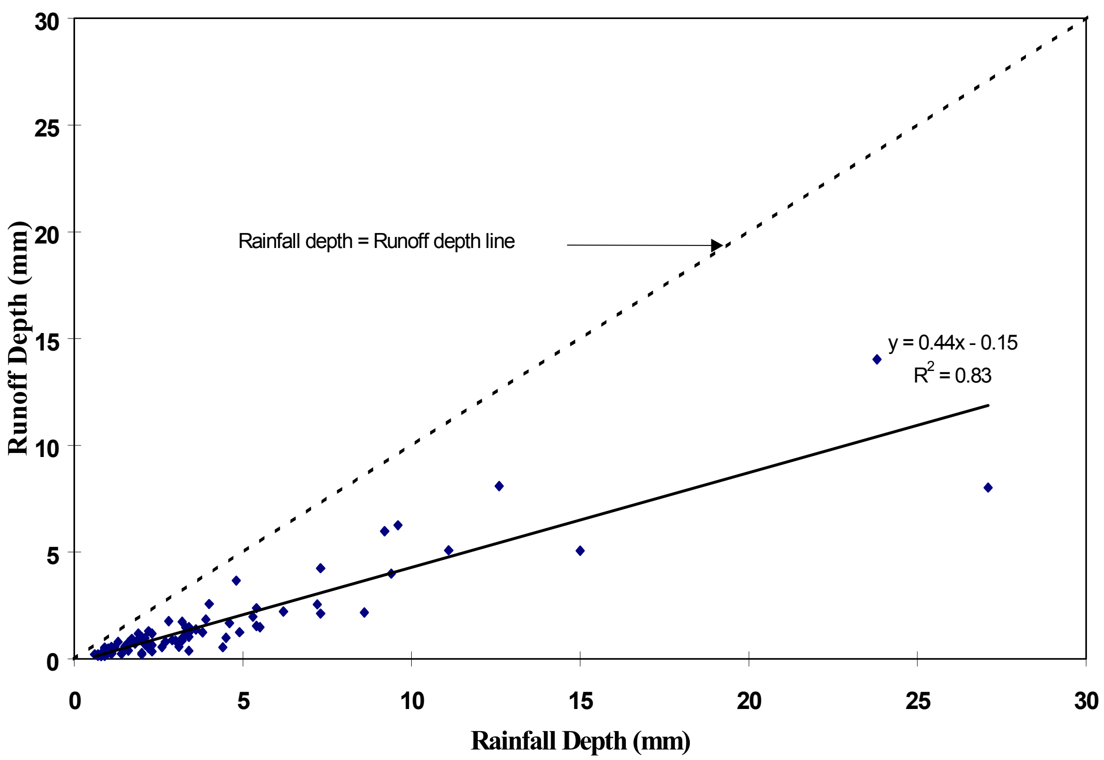


Figure C.16: RR Plot of Catchment K1A

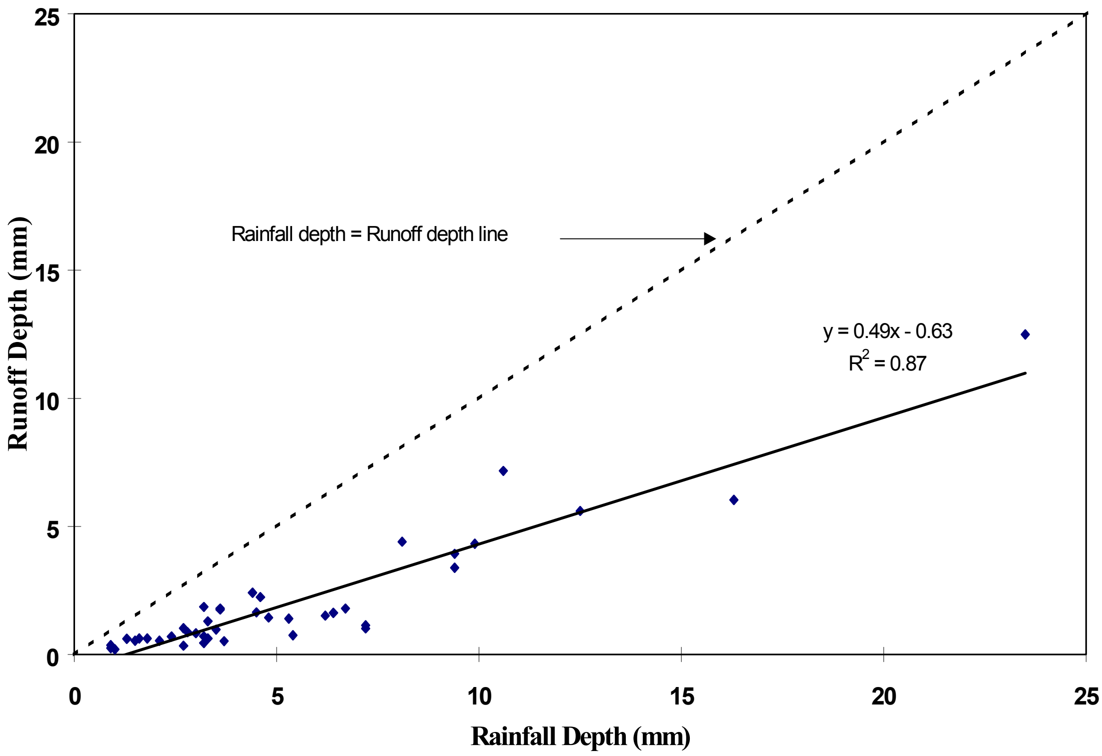


Figure C.17: RR Plot of Catchment K1B

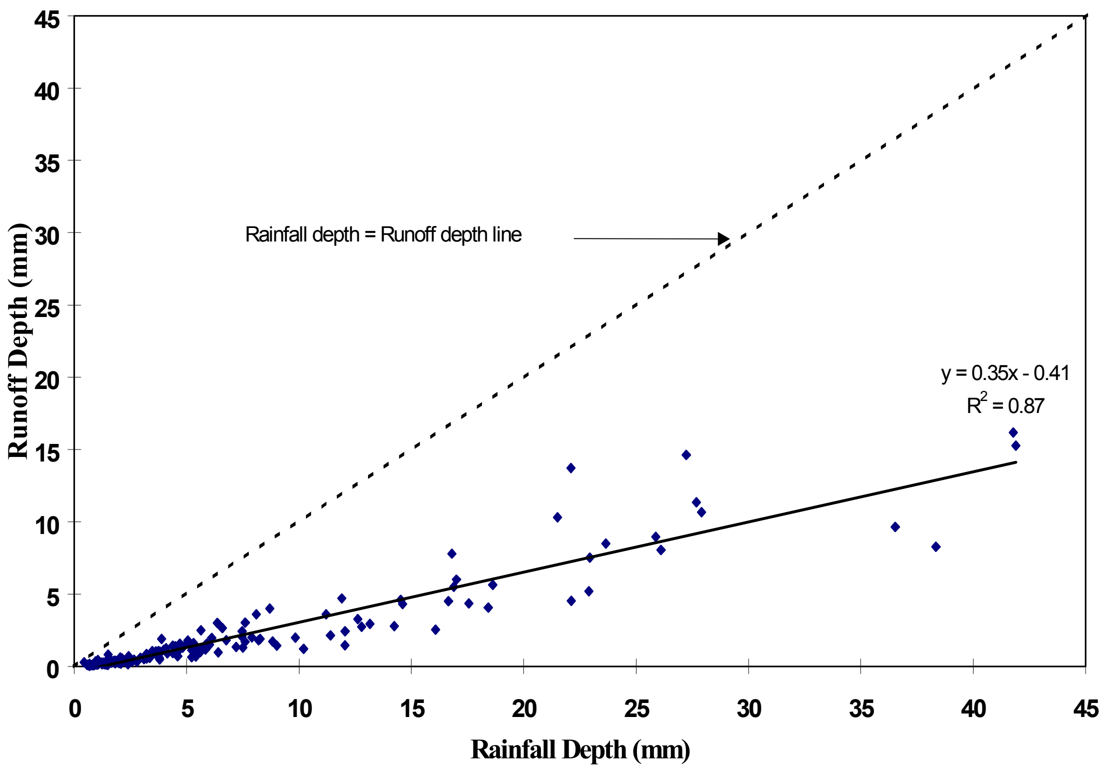


Figure C.18: RR Plot of Catchment K2

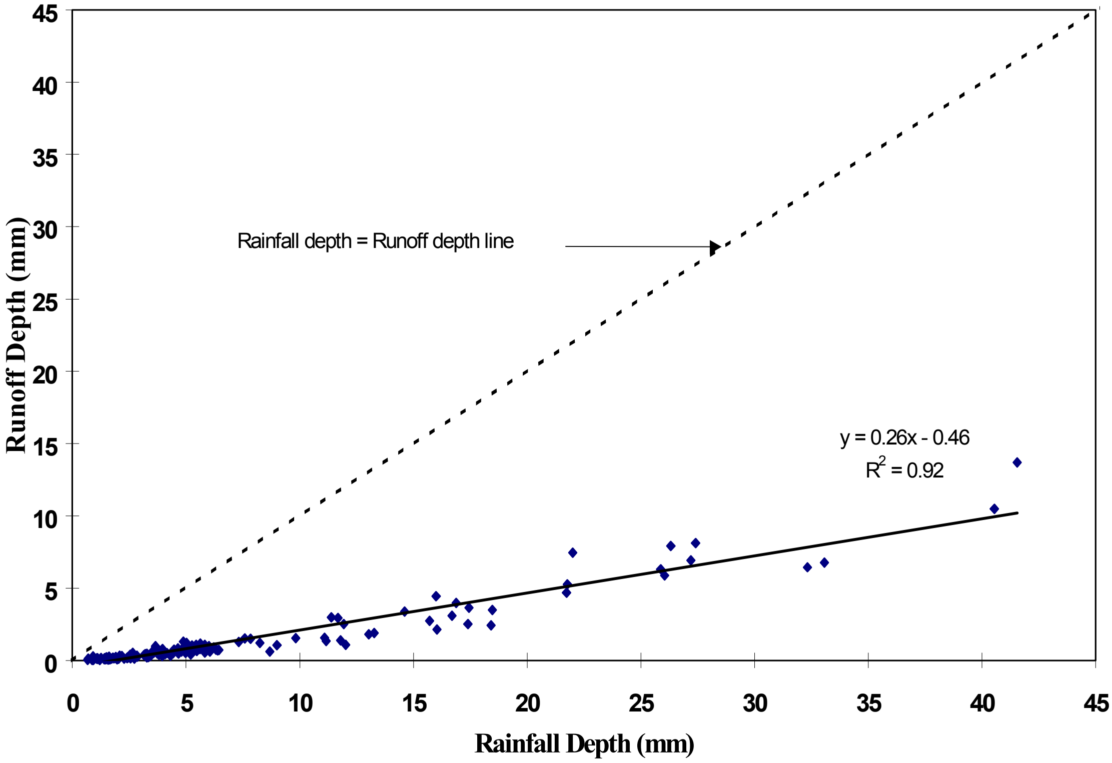


Figure C.19: RR Plot of Catchment K2A

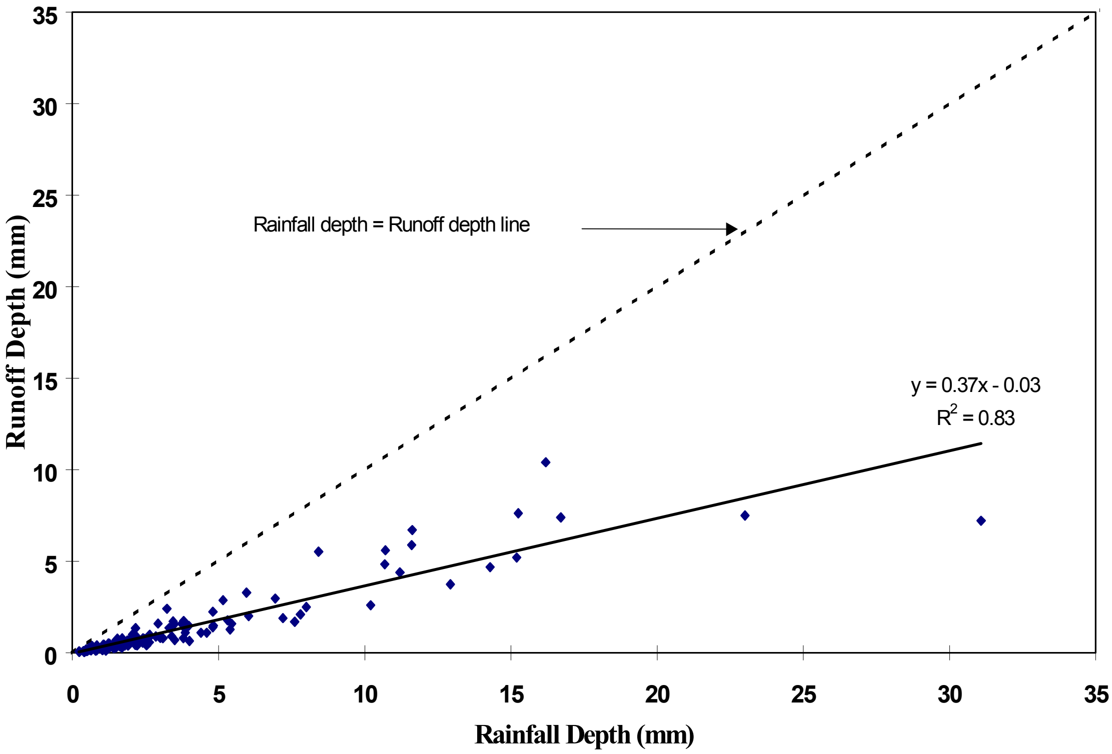


Figure C.20: RR Plot of Catchment K3

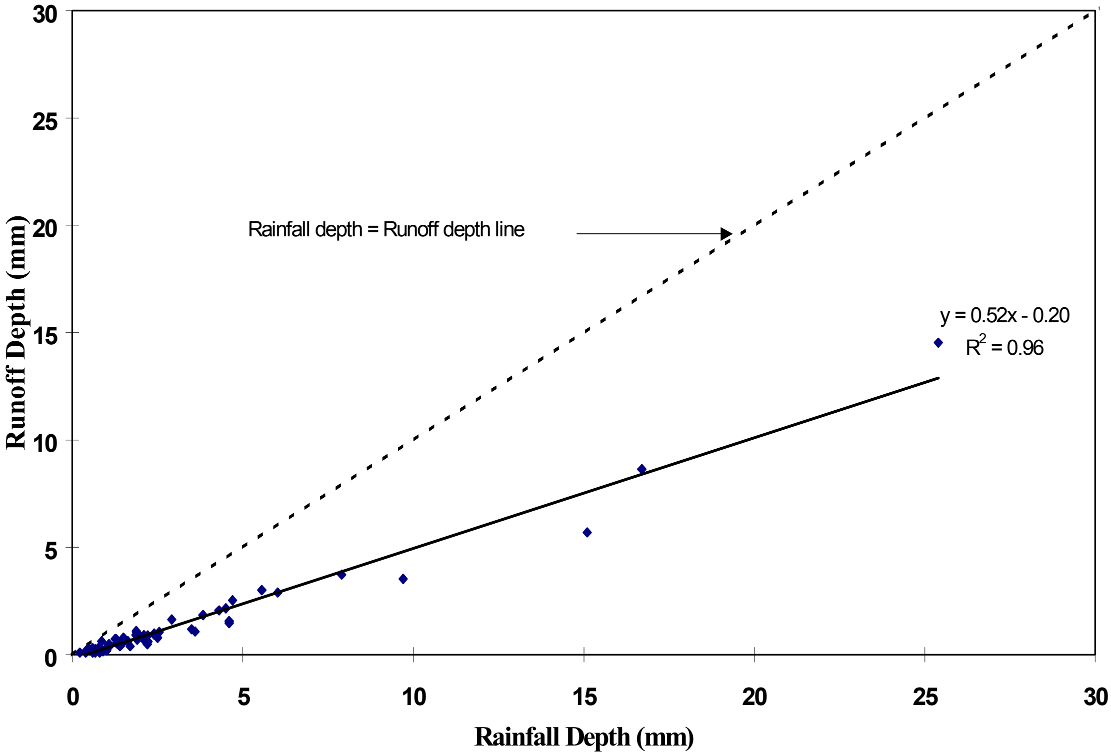


Figure C.21: RR Plot of Catchment K3A

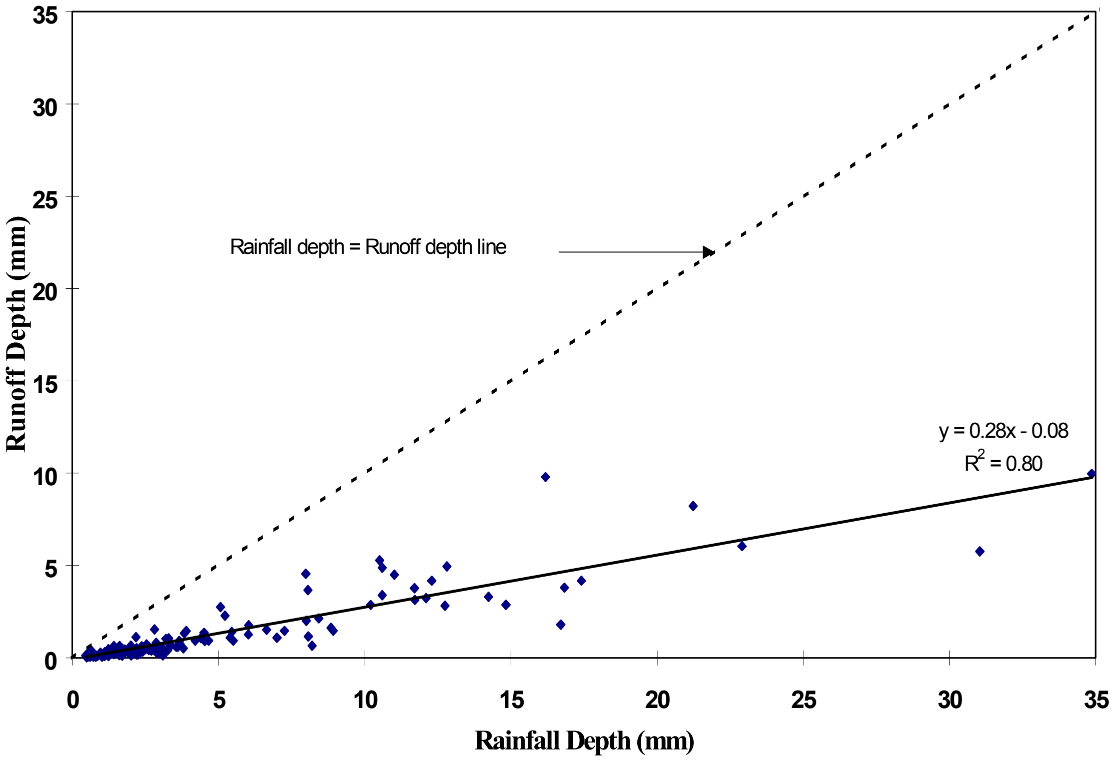


Figure C.22: RR lot of Catchment K3B

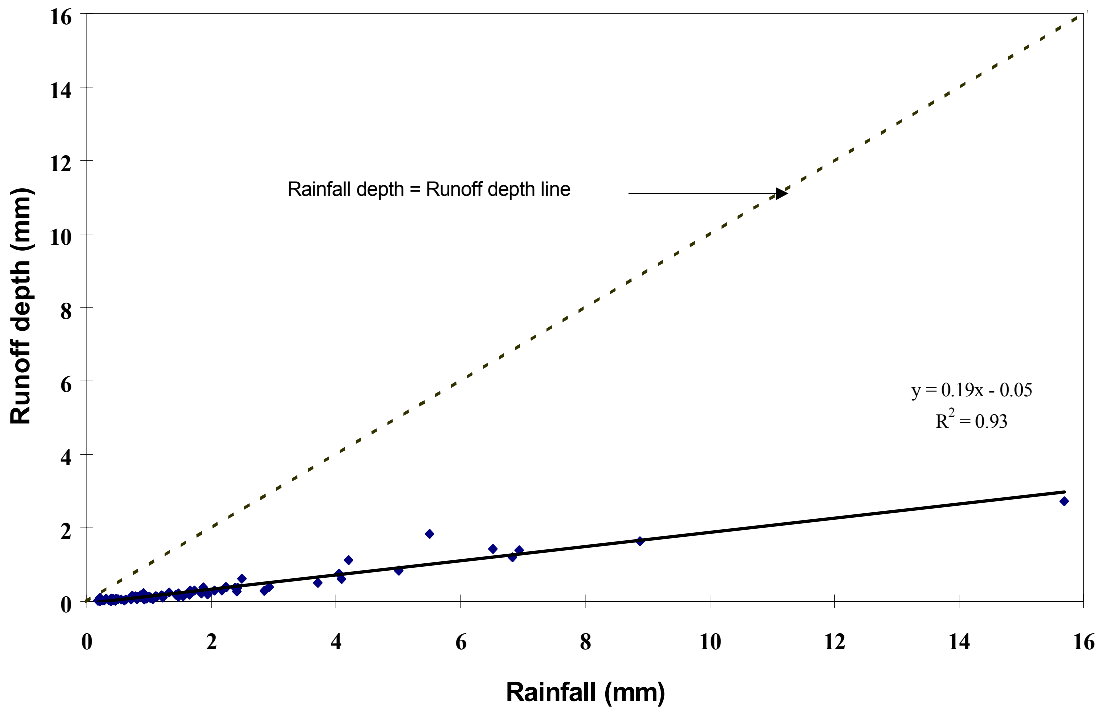


Figure C.23: RR Plot of Catchment GI

APPENDIX D

EVENTS SELECTED FOR CALIBRATION AND VERIFICATION

Table D.1: Summary of Statistics of Storm Events Selected for Modelling of Catchment BA2

Event Properties	Calibration Events				Verification Events		
Event number	C1	C2	C3	C4	V1	V2	V3
Date of occurrence	11.08.97	20.04.98	11.05.98	25.05.98	14.09.97	16.02.98	06.06.98
Total rainfall duration (min)	190	364	628	468	620	884	1002
Total rainfall depth (mm)	3.6	6.6	9.0	5.0	15.3	28.0	27.2
Maximum 2 min. intensity (mm/h)	8.0	12.0	4.0	4.0	18.0	36.0	30.0
ARI of storm event (years)	<1	<1	<1	<1	<1	<1	<1
Average intensity of most severe burst (mm/h)	6.0	8.2	3.2	3.3	10.0	28.0	22.0
Stormwater runoff volume (m^3)	671	1686	2271	1772	3464	4998	9436
Maximum discharge (m^3/s)	0.162	0.256	0.171	0.183	0.384	0.606	0.751

Table D.2: Summary of Statistics of Storm Events Selected for Modelling of Catchment BA2A

Event Properties	**Calibration Events**					**Verification Events**		
Event number	C1	C2	C3	C4	C5	V1	V2	V3
Date of occurrence	14.09.97	10.11.97	16.02.98	20.04.98	19.05.98	20.05.98	25.05.98	06.06.98
Total rainfall duration (min)	426	124	370	274	286	332	344	200
Total rainfall depth (mm)	10.6	15.3	17.2	6.7	16.4	11.8	4.9	3.8
Maximum 2 min. intensity (mm/h)	18	126	12	12	42	12	4	12
ARI of storm event (years)	<1	2	<1	<1	<1	<1	<1	<1
Average intensity of most severe burst (mm/h)	10.0	70.0	10.0	8.2	36.0	10.0	3.3	12.0
Stormwater runoff volume (m^3)	543	509	917	346	802	843	268	267
Maximum discharge (m^3/s)	0.104	0.528	0.099	0.080	0.373	0.119	0.043	0.113

Table D.3: Summary of Statistics of Storm Events Selected for Modelling of Catchment BA3

Event Properties	**Calibration Events**				**Verification Events**		
Event number	C1	C2	C3	C4	V1	V2	V3
Date of occurrence	25.01.98	20.05.98	21.06.98	28.07.98	10.09.97	14.09.97	06.06.98
Total rainfall duration (min)	412	388	616	408	320	622	640
Total rainfall depth (mm)	7.5	11.7	14.7	6.8	3.6	15.4	29.5
Maximum 2 min. intensity (mm/h)	20	30	24	6	21	18	48
ARI of storm event (years)	<1	<1	<1	<1	<1	<1	<1
Average intensity of most severe burst (mm/h)	10.5	22.0	16.0	5.3	16.0	10.0	40.0
Stormwater runoff volume (m^3)	1483	2316	2620	1226	956	4405	5461
Maximum discharge (m^3/s)	0.225	0.664	0.301	0.158	0.874	0.360	0.972

Table D.4: Summary of Statistics of Storm Events Selected for Modelling of Catchment BA3A

Event Properties	**Calibration Events**					**Verification Events**		
Event number	C1	C2	C3	C4	C5	V1	V2	V3
Date of occurrence	01.09.97	16.02.98	19.05.98	20.05.98	28.07.98	14.09.97	20.05.98	06.06.98
Total rainfall duration (min)	638	276	282	658	512	782	494	912
Total rainfall depth (mm)	4.5	12.8	22.2	14.8	6.8	15.6	11.6	30.0
Maximum 2 min. intensity (mm/h)	2.4	48.0	42.0	30.0	6.0	18.0	30.0	48.0
ARI of storm event (years)	<1	<1	<1	<1	<1	<1	<1	<1
Average intensity of most severe burst (mm/h)	2.2	40.0	30.2	22.0	5.3	10.0	22.0	40.0
Stormwater runoff volume (m^3)	435	1345	1377	1462	728	2166	1546	3652
Maximum discharge (m^3/s)	0.066	0.489	0.491	0.322	0.101	0.235	0.360	0.671

Table D.5: Summary of Statistics of Storm Events Selected for Modelling of Catchment BA3B

Event Properties	**Calibration Events**					**Verification Events**		
Event number	C1	C2	C3	C4	C5	V1	V2	V3
Date of occurrence	16.02.98	16.02.98	26.04.98	19.05.98	29.07.98	14.01.97	20.05.98	20.05.98
Total rainfall duration (min)	334	194	136	226	246	270	116	86
Total rainfall depth (mm)	17.4	11.6	13.5	15.1	5.5	7.3	4.6	2.3
Maximum 2 min. intensity (mm/h)	18	48	24	42	6	9	18	12
ARI of storm event (years)	<1	<1	<1	<1	<1	<1	<1	<1
Average intensity of most severe burst (mm/h)	16.0	40.0	22.0	30.2	5.3	6.0	14.0	7.3
Stormwater runoff volume (m^3)	522	400	361	448	237	277	180	115
Maximum discharge (m^3/s)	0.110	0.172	0.145	0.174	0.034	0.054	0.104	0.086

Table D.6: Summary of Statistics of Storm Events Selected for Modelling of Catchment BO1A

Event Properties	**Calibration Events**				**Verification Events**		
Event number	C1	C2	C3	C4	V1	V2	V3
Date of occurrence	13.06.97	07.02.98	11.05.98	19.05.98	16.02.98	06.06.98	21.06.98
Total rainfall duration (min)	306	340	222	272	338	530	164
Total rainfall depth (mm)	10	17	5	17	24	22	13
Maximum 2 min. intensity (mm/h)	12	24	6	30	12	18	18
ARI of storm event (years)	<1	<1	<1	<1	<1	<1	<1
Average intensity of most severe burst (mm/h)	9	13	5	24	11	12	12
Stormwater runoff volume (m^3)	178	247	113	332	213	566	147
Maximum discharge (m^3/s)	0.035	0.042	0.026	0.072	0.048	0.046	0.048

Table D.7: Summary of Statistics of Storm Events Selected for Modelling of Catchment BO2A

Event Properties	Calibration Events					Verification Events		
Event number	C1	C2	C3	C4	C5	V1	V2	V3
Date of occurrence	29.05.97	31.10.97	25.01.98	12.04.98	20.04.98	14.11.97	25.01.98	26.04.98
Total rainfall duration (min)	130	148	192	136	298	368	100	150
Total rainfall depth (mm)	5.7	4.8	4.8	2.4	6.0	5.6	1.1	5.1
Maximum 2 min. intensity (mm/h)	30	24	12	3	8	6	12	12
ARI of storm event (years)	<1	<1	<1	<1	<1	<1	<1	<1
Average intensity of most severe burst (mm/h)	24.0	18.0	6.5	2.3	6.5	5.6	6.7	10.7
Stormwater runoff volume (m^3)	131	137	154	82	228	257	230	213
Maximum discharge (m^3/s)	0.101	0.097	0.032	0.020	0.050	0.047	0.285	0.065

Table D.8: Summary of Statistics of Storm Events Selected for Modelling of Catchment BR1

Event Properties	**Calibration Events**				**Verification Events**		
Event number	C1	C2	C3	C4	V1	V2	V3
Date of occurrence	10.11.97	14.01.98	25.01.98	16.02.98	14.11.97	20.04.98	26.04.98
Total rainfall duration (min)	188	174	106	352	688	186	310
Total rainfall depth (mm)	17.3	4.7	3.6	16.2	13.6	3.8	3.7
Maximum 2 min. intensity (mm/h)	54	18	18	24	9	9	5
ARI of storm event (years)	<1	<1	<1	<1	<1	<1	<1
Average intensity of most severe burst (mm/h)	37.0	16.0	8.4	15.3	7.4	7.0	4.5
Stormwater runoff volume (m^3)	673	184	151	664	689	140	168
Maximum discharge (m^3/s)	0.417	0.159	0.097	0.180	0.094	0.064	0.043

Table D.9: Summary of Statistics of Storm Events Selected for Modelling of Catchment BR1A

Event Properties	Calibration Events				Verification Events		
Event number	C1	C2	C3	C4	V1	V2	V3
Date of occurrence	10.11.97	25.01.98	07.02.98	16.02.98	14.11.97	13.01.98	25.01.98
Total rainfall duration (min)	152	192	372	96	226	72	92
Total rainfall depth (mm)	17.1	5.5	18.2	5.9	6.6	8.4	3.5
Maximum 2 min. intensity (mm/h)	54	13	18	24	9	84	18
ARI of storm event (years)	<1	<1	<1	<1	<1	2	<1
Average intensity of most severe burst (mm/h)	37.0	7.0	13.0	16.0	7.4	68.0	8.4
Stormwater runoff volume (m^3)	362	126	394	124	171	197	96
Maximum discharge (m^3/s)	0.288	0.031	0.059	0.089	0.057	0.403	0.077

Table D.10: Summary of Statistics of Storm Events Selected for Modelling of Catchment BR2

Event Properties	**Calibration Events**					**Verification Events**		
Event number	C1	C2	C3	C4	C5	V1	V2	V3
Date of occurrence	10.11.97	13.01.98	07.02.98	16.02.98	26.04.98	30.10.97	25.01.98	20.04.98
Total rainfall duration (min)	290	164	486	648	434	888	988	340
Total rainfall depth (mm)	17.6	10.8	16.9	21.4	5.7	12.2	20.7	4.0
Maximum 2 min. intensity (mm/h)	78	48	12	24	7	9	48	15
ARI of storm event (years)	2	<1	<1	<1	<1	<1	<1	<1
Average intensity of most severe burst (mm/h)	60.0	36.2	11.0	15.0	4.2	7.0	32.0	8.3
Stormwater runoff volume (m^3)	2602	1498	2801	4268	1417	2134	3066	1003
Maximum discharge (m^3/s)	1.287	0.717	0.279	0.454	0.164	0.252	0.341	0.273

Table D.11: Summary of Statistics of Storm Events Selected for Modelling of Catchment BR2A

Event Properties	**Calibration Events**					**Verification Events**		
Event number	C1	C2	C3	C4	C5	V1	V2	V3
Date of occurrence	13.01.98	14.01.98	07.02.98	26.02.98	20.04.98	16.02.98	26.04.98	19.05.98
Total rainfall duration (min)	200	212	498	326	296	728	418	388
Total rainfall depth (mm)	10.8	3.8	16.9	4.2	4.0	21.4	5.7	13.0
Maximum 2 min. intensity (mm/h)	48	24	12	24	15	24	7	18
ARI of storm event (years)	<1	<1	<1	<1	<1	<1	<1	<1
Average intensity of most severe burst (mm/h)	36.3	11.0	11.0	16.2	8.3	15.0	4.3	16.0
Stormwater runoff volume (m^3)	580	236	1194	203	315	1774	495	1111
Maximum discharge (m^3/s)	0.283	0.051	0.018	0.059	0.101	0.176	0.058	0.273

Table D.12: Summary of Statistics of Storm Events Selected for Modelling of Catchment BR3

Event Properties	Calibration Events					Verification Events		
Event number	C1	C2	C3	C4	C5	V1	V2	V3
Date of occurrence	14.09.97	19.09.97	13.01.98	25.01.98	26.04.98	30.10.97	10.11.97	25.01.98
Total rainfall duration (min)	586	442	128	112	342	656	172	98
Total rainfall depth (mm)	14.8	5.5	10.2	6.0	7.4	11.3	19.8	7.2
Maximum 2 min. intensity (mm/h)	6	6	60	36	9	9	66	51
ARI of storm event (years)	<1	<1	<1	<1	<1	<1	<1	<1
Average intensity of most severe burst (mm/h)	6.0	5.0	28.0	26.6	7.0	7.0	50.0	27.7
Stormwater runoff volume (m^3)	1165	343	724	451	559	1069	1607	599
Maximum discharge (m^3/s)	0.094	0.066	0.415	0.388	0.082	0.147	0.998	0.273

Table D.13: Summary of Statistics of Storm Events Selected for Modelling of Catchment H2

Event Properties	**Calibration Events**					**Verification Events**		
Event number	C1	C2	C3	C4	C5	V1	V2	V3
Date of occurrence	11.08.97	30.10.97	31.10.97	10.11.97	25.01.98	11.08.97	30.10.97	27.11.97
Total rainfall duration (min)	226	148	268	342	306	140	304	124
Total rainfall depth (mm)	5.4	3.2	5.2	25.7	4.3	1.9	4.2	6.0
Maximum 2 min. intensity (mm/h)	7	12	8	60	6	3	14	30
ARI of storm event (years)	<1	<1	<1	1	<1	<1	<1	<1
Average intensity of most severe burst (mm/h)	4.5	10.2	6.0	50.0	4.0	3.0	8.0	26
Stormwater runoff volume (m^3)	330	200	622	2116	497	155	900	629
Maximum discharge (m^3/s)	0.076	0.077	0.101	0.762	0.061	0.055	0.109	0.275

Table D.14: Summary of Statistics of Storm Events Selected for Modelling of Catchment H2A

Event Properties	**Calibration Events**				**Verification Events**		
Event number	C1	C2	C3	C4	V1	V2	V3
Date of occurrence	07.08.97	07.02.98	16.02.98	16.02.98	01.09.97	25.01.98	
Total rainfall duration (min)	138	398	128	350	198	114	100
Total rainfall depth (mm)	10.8	13.6	8.0	12.0	11.2	1.9	3.3
Maximum 2 min. intensity (mm/h)	36	24	18	12	36	12	12
ARI of storm event (years)	<1	<1	<1	<1	<1	<1	<1
Average intensity of most severe burst (mm/h)	36.0	12.0	18	9.0	24.0	7.0	11.0
Stormwater runoff volume (m^3)	281	574	301	626	476	254	131
Maximum discharge (m^3/s)	0.209	0.092	0.167	0.108	0.268	0.288	.082

Table D.15: Summary of Statistics of Storm Events Selected for Modelling of Catchment K1

Event Properties	**Calibration Events**					**Verification Events**		
Event number	C1	C2	C3	C4	C5	V1	V2	V3
Date of occurrence	03.11.96	06.05.97	11.08.97	31.10.97	10.11.97	08.05.97	01.11.97	11.11.97
Total rainfall duration (min)	468	440	282	176	220	170	104	216
Total rainfall depth (mm)	34.5	20.1	27.2	9.3	12.5	6.9	4.1	8.3
Maximum 2 min. intensity (mm/h)	90.0	66.0	6.0	24.0	36	12	9	36
ARI of storm event (years)	2	<1	<1	<1	<1	<1	<1	<1
Average intensity of most severe burst (mm/h)	67.5	38.0	5.7	17.3	28.0	9.0	6.5	28.0
Stormwater runoff volume (m^3)	1716	1538	354	583	766	448	171	680
Maximum discharge (m^3/s)	0.496	0.356	0.086	0.300	0.292	0.135	0.110	0.446

Table D.16: Summary of Statistics of Storm Events Selected for Modelling of Catchment K1A

Event Properties	Calibration Events					Verification Events		
Event number	C1	C2	C3	C4	C5	V1	V2	V3
Date of occurrence	13.06.97	25.06.97	31.10.97	10.11.97	19.04.98	14.09.97	11.11.97	19.05.98
Total rainfall duration (min)	136	408	134	196	534	652	222	160
Total rainfall depth (mm)	10.0	11.1	9.2	12.6	9.4	16.3	8.3	9.6
Maximum 2 min. intensity (mm/h)	12	12	24	36	9	12	36	24
ARI of storm event (years)	<1	<1	<1	<1	<1	<1	<1	<1
Average intensity of most severe burst (mm/h)	10.0	10.0	17.0	28.0	8.0	8.5	28.0	16.1
Stormwater runoff volume (m^3)	216	289	325	465	230	830	432	276
Maximum discharge (m^3/s)	0.071	0.065	0.165	0.151	0.086	0.092	0.174	0.110

Table D.17: Summary of Statistics of Storm Events Selected for Modelling of Catchment K1B

Event Properties	**Calibration Events**					**Verification Events**	
Event number	C1	C2	C3	C4	C5	V1	V2
Date of occurrence	27.09.97	16.10.97	06.11.97	10.11.97	11.11.97	31.10.97	01.11.97
Total rainfall duration (min)	152	162	176	194	82	358	346
Total rainfall depth (mm)	3.2	4.6	3.4	12.5	8.6	9.2	4.1
Maximum 2 min. intensity (mm/h)	8	24	12	36	36	24	9
ARI of storm event (years)	<1	<1	<1	<1	<1	<1	<1
Average intensity of most severe burst (mm/h)	6.4	14.4	10.0	28.0	28.0	17.3	6.5
Stormwater runoff volume (m^3)	81	100	102	238	193	334	130
Maximum discharge (m^3/s)	0.032	0.058	0.048	0.099	0.129	0.125	0.042

Table D.18: Summary of Statistics of Storm Events Selected for Modelling of Catchment K2

Event Properties	**Calibration Events**			**Verification Events**		
Event number	C1	C2	C3	V1	V2	V3
Date of occurrence	25.12.96	31.10.97	25.01.98	22.01.97	16.02.98	20.05.98
Total rainfall duration (min)	418	386	618	352	1114	1214
Total rainfall depth (mm)	8.7	11.9	27.9	21.5	41.8	27.7
Maximum 2 min. intensity (mm/h)	11.3	15.0	24.0	15.0	30.0	15.0
ARI of storm event (years)	<1	<1	<1	<1	<1	<1
Average intensity of most severe burst (mm/h)	9.3	14.1	20.0	14.1	22.0	10.1
Stormwater runoff volume (m^3)	878	1416	2490	2254	3723	2467
Maximum discharge (m^3/s)	0.123	0.348	0.252	0.292	0.391	0.155

Table D.19: Summary of Statistics of Storm Events Selected for Modelling of Catchment K2A

Event Properties	Calibration Events					Verification Events		
Event number	C1	C2	C3	C4	C5	V1	V2	V3
Date of occurrence	31.10.97	11.11.97	26.01.98	23.03.98	19.04.98	14.09.97	06.06.98	24.06.98
Total rainfall duration (min)	160	138	168	206	500	470	638	402
Total rainfall depth (mm)	11.7	16.9	4.9	11.4	14.6	26.3	41.6	22.0
Maximum 2 min. intensity (mm/h)	18	78	14	12	12	18	30	24
ARI of storm event (years)	<1	<1	<1	<1	<1	<1	<1	<1
Average intensity of most severe burst (mm/h)	14.0	54.0	8.0	9.0	12.0	12.0	24.0	22.0
Stormwater runoff volume (m^3)	410	649	203	423	451	1201	2236	1012
Maximum discharge (m^3/s)	0.148	0.419	0.062	0.070	0.109	0.129	0.196	0.158

Table D.20: Summary of Statistics of Storm Events Selected for Modelling of Catchment K3

Event Properties	Calibration Events				Verification Events		
Event number	C1	C2	C3	C4	V1	V2	V3
Date of occurrence	29.05.97	13.06.97	01.11.97	.8.07.98	14.09.97	13.11.97	09.07.98
Total rainfall duration (min)	414	506	306	216	986	1196	348
Total rainfall depth (mm)	11.2	15.2	4.8	3.7	11.6	15.3	3.6
Maximum 2 min. intensity (mm/h)	12	30	15	30	6	6	9
ARI of storm event (years)	<1	<1	<1	<1	<1	<1	<1
Average intensity of most severe burst (mm/h)	9.3	22.0	8.5	14.7	5.0	5.2	6.4
Stormwater runoff volume (m^3)	1675	2146	515	657	2702	2078	776
Maximum discharge (m^3/s)	0.318	0.344	0.237	0.317	0.251	0.156	0.301

Table D.21: Summary of Statistics of Storm Events Selected for Modelling of Catchment K3A

Event Properties	Calibration Events					Verification Events		
Event number	C1	C2	C3	C4	C5	V1	V2	V3
Date of occurrence	21.04.97	14.05.97	08.05.97	14.09.97	10.11.97	29.05.97	30.10.97	31.10.97
Total rainfall duration (min)	272	384	418	550	220	616	178	440
Total rainfall depth (mm)	5.2	7.2	8.2	11.6	12.8	11.3	5.4	14.3
Maximum 2 min. intensity (mm/h)	8	12	18	6	60	12	4	42
ARI of storm event (years)	<1	<1	<1	<1	<1	<1	<1	<1
Average intensity of most severe burst (mm/h)	5.0	8.8	12.0	5.0	34.0	9.3	3.5	28.0
Stormwater runoff volume (m^3)	444	522	739	766	726	1261	265	1087
Maximum discharge (m^3/s)	0.076	0.102	0.139	0.073	0.489	0.160	0.054	0.660

Table D.22: Summary of Statistics of Storm Events Selected for Modelling of Catchment K3B

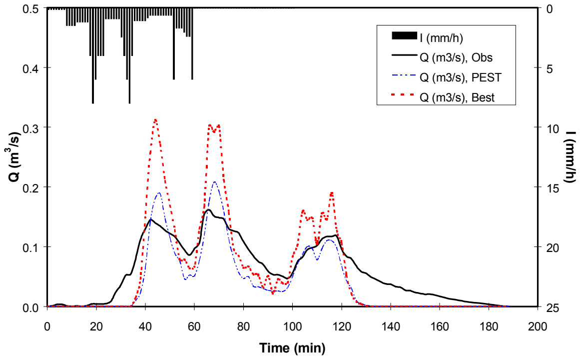
Event Properties	Calibration Events				Verification Events		
Event number	C1	C2	C3	C4	V1	V2	V3
Date of occurrence	06.05.97	13.06.97	07.08.97	25.01.98	29.05.97	14.09.97	06.06.98
Total rainfall duration (min)	410	74	120	208	138	102	544
Total rainfall depth (mm)	22.9	12.1	10.2	11.7	10.6	11.7	21.2
Maximum 2 min. intensity (mm/h)	30	12	24	18	12	6	24
ARI of storm event (years)	<1	<1	<1	<1	<1	<1	<1
Average intensity of most severe burst (mm/h)	22.5	7.3	24.0	14.0	9.0	5.0	19.0
Stormwater runoff volume (m^3)	547	55	251	306	206	67	1130
Maximum discharge (m^3/s)	0.144	0.066	0.145	0.145	0.114	0.058	0.185

Table D.23: Summary of Statistics of Storm Events Selected for Modelling of Catchment GI

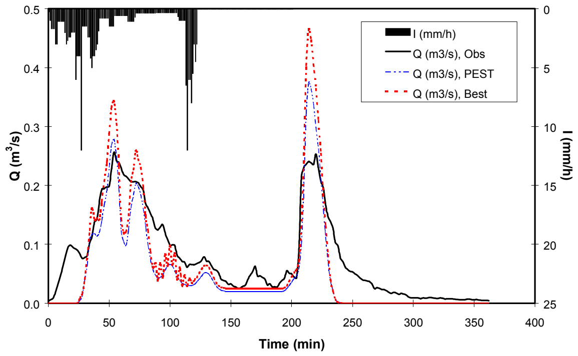
Event Properties	**Calibration Events**									**Verification Events**		
	'Small'				'Large'							
Event number	CS1	CS2	CS3	CS4	CL1	CL2	CL3	CL4	CL5	V1	V2	V3
Date of occurrence	15.02.77	12.09.77	12.01.78	24.11.79	06.11.89	03.01.93	05.02.81	03.03.92	25.03.84	27.01.78	12.02.81	3.392
Total rainfall duration (min)	200	35	30	15	1885	320	575	1195	695	210	250	370
Total rainfall depth (mm)	15.7	4.2	8.9	5.5	44.5	75.2	69.8	98.0	45.8	33.5	43.7	68.6
Maximum 2 min. intensity (mm/h)	52.0	12.0	50.0	38.0	144.0	97.2	60.6	136.2	94.8	19.2	39.1	120.1
ARI of storm event (years)	<1	<1	<1	<1	40	10	40	35	5	1	1	10
Average intensity of Most Severe burst (mm/h)	52.0	12.0	50.0	38.0	144.0	97.2	92.0	136.2	94.8	28.2	39.1	120.1
Stormwater runoff volume (m^3)	2568	1056	1540	1728	15030	26706	28050	22961	9299	9517	12097	22961
Maximum discharge (m^3/s)	1.441	0.725	1.767	2.339	8.173	7.123	9.660	10.703	6.206	4.646	2.806	10.703

APPENDIX E

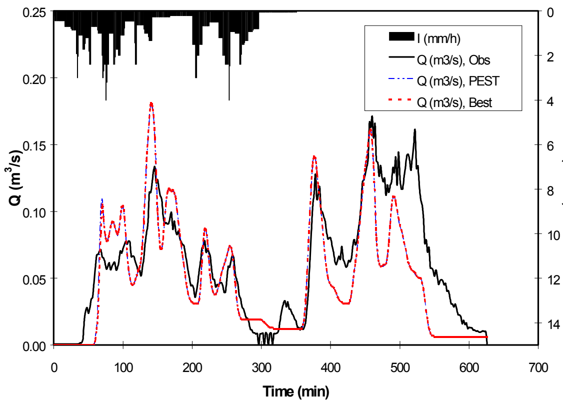
CALIBRATION PLOTS



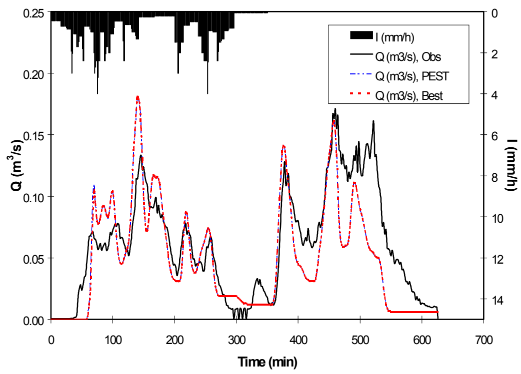
(a) Event C1



(b) Event C2

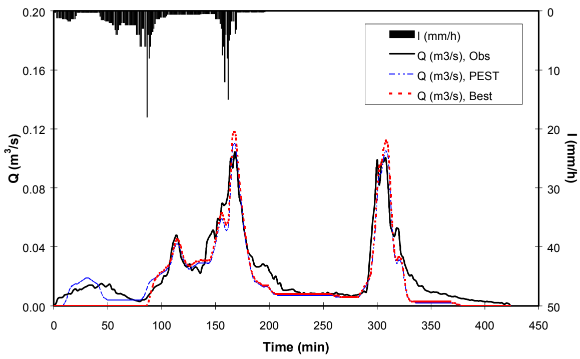


(c) Event C3

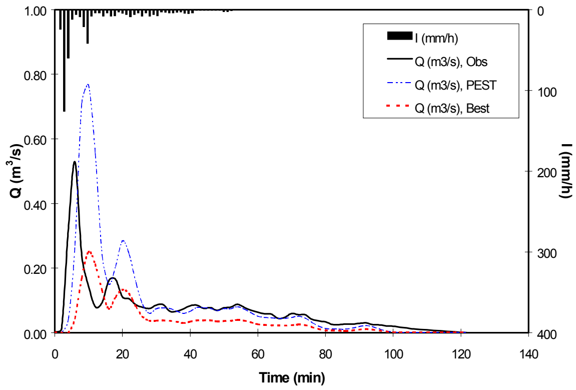


(d) Event C4

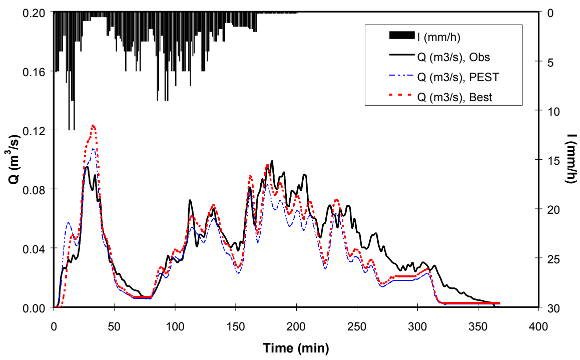
Figure E1: Calibration Plots for Catchment BA2



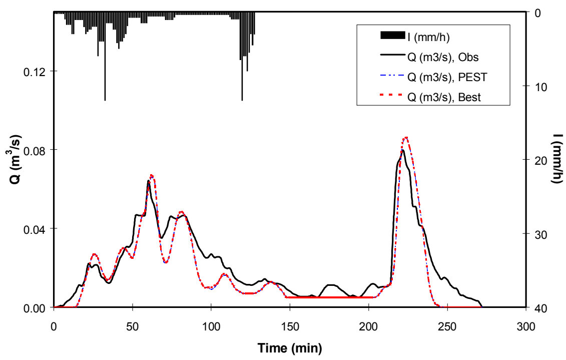
(a) Event C1



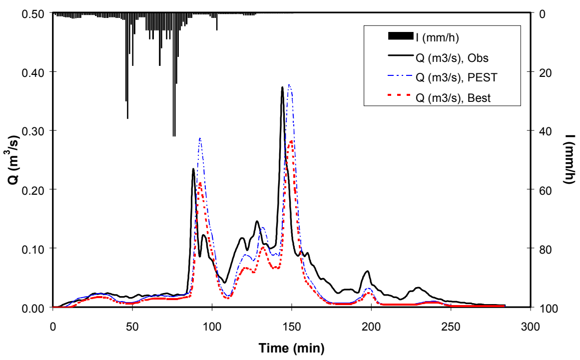
(b) Event C2



(c) Event C3

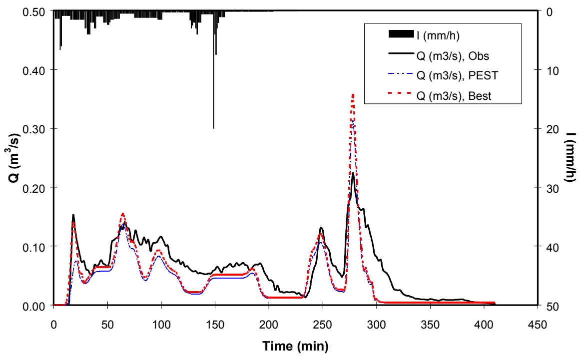


(d) Event C4

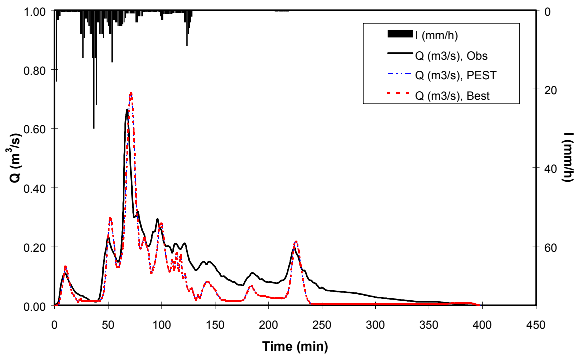


(e) Event C5

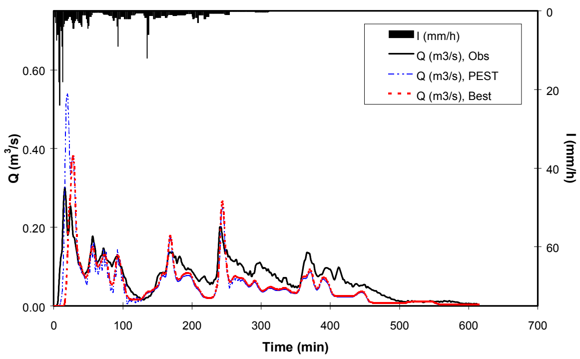
Figure E2: Calibration Plots for Catchment BA2A



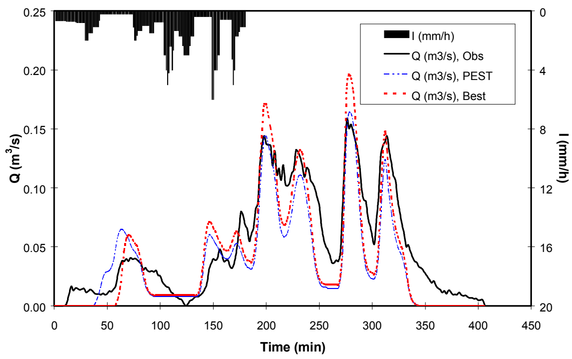
(a) Event C1



(b) Event C2

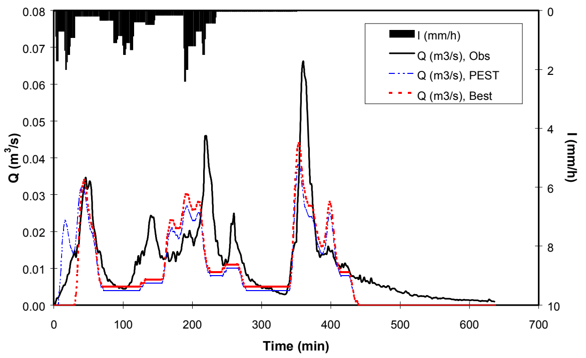


(c) Event C3

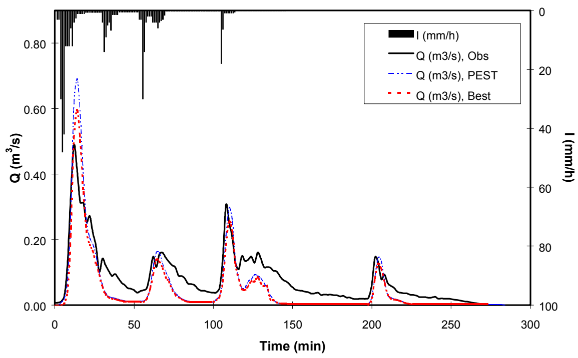


(d) Event C4

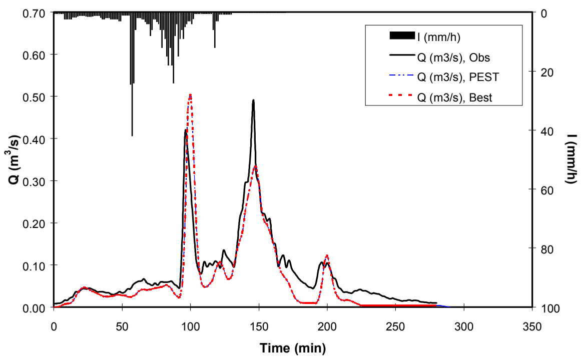
Figure E3: Calibration Plots for Catchment BA3



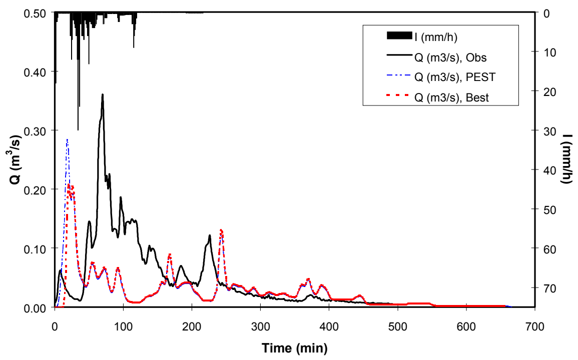
(a) Event C1



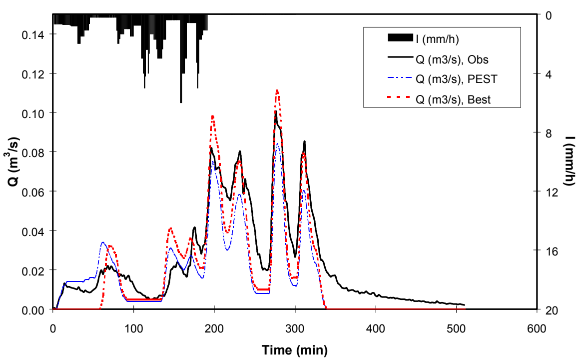
(b) Event C2



(c) Event C3

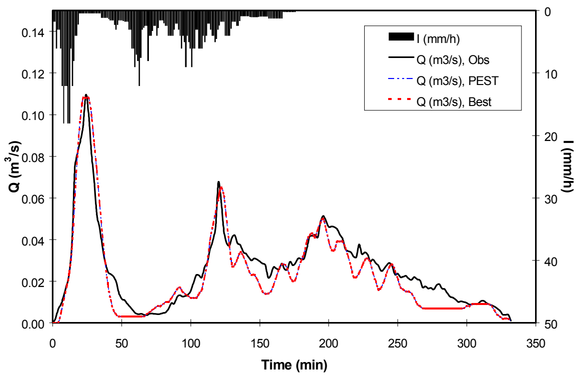


(d) Event C4

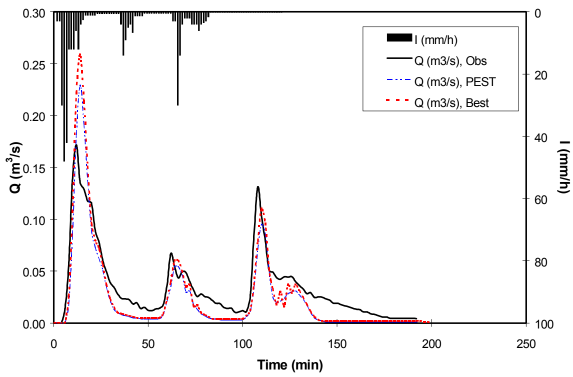


(e) Event C5

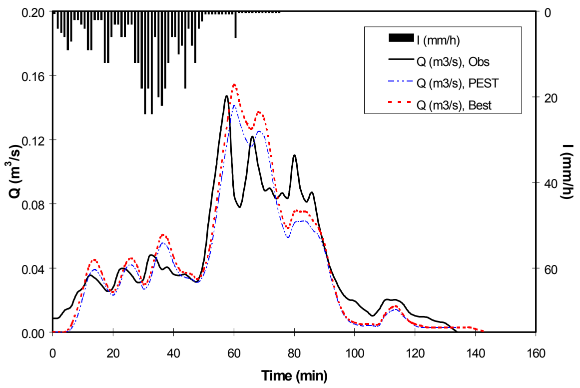
Figure E4: Calibration Plots for Catchment BA3A



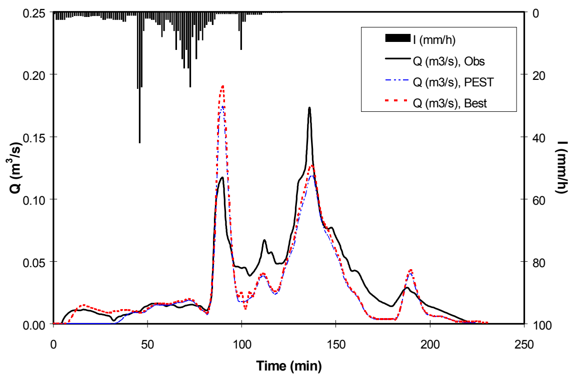
(a) Event C1



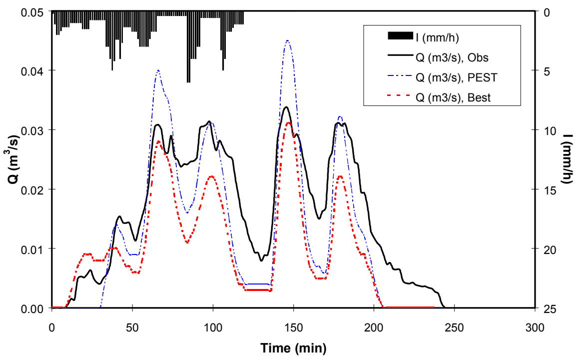
(b) Event C2



(c) Event C3

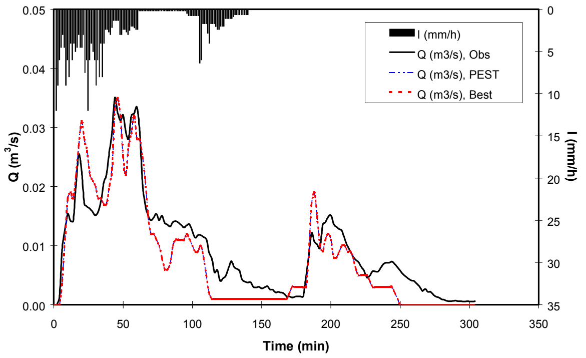


(d) Event C4

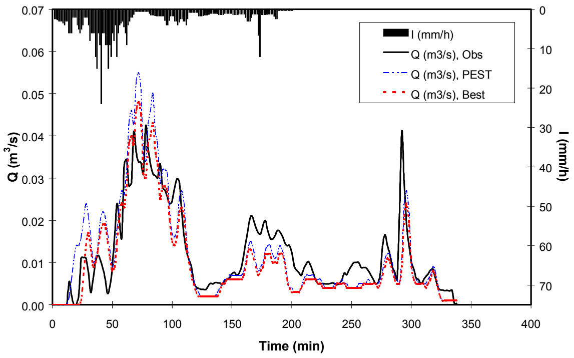


(e) Event C5

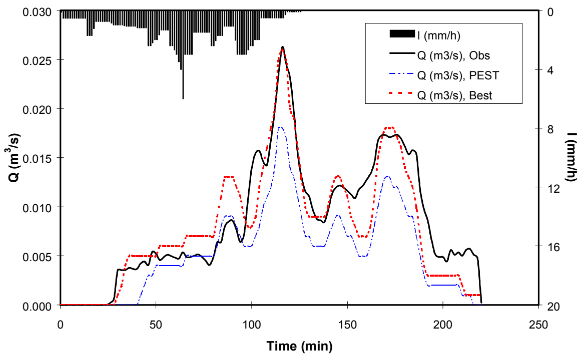
Figure E5: Calibration Plots for Catchment BA3B



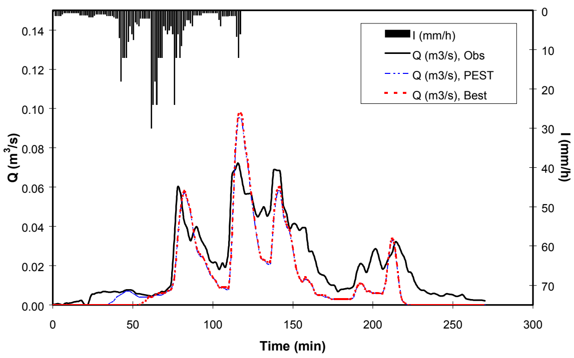
(a) Event C1



(b) Event C2

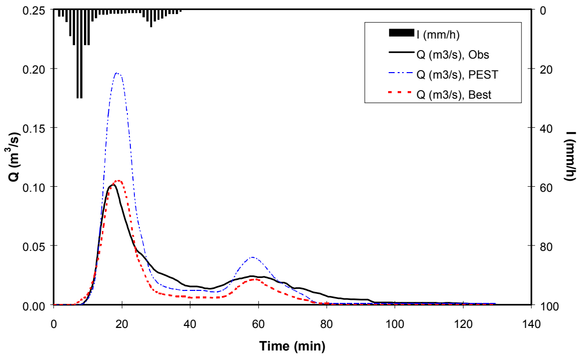


(c) Event C3

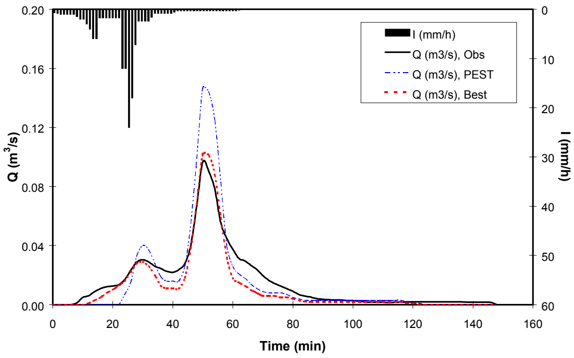


(d) Event C4

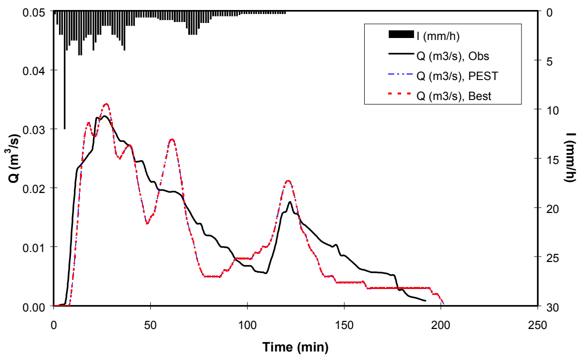
Figure E6: Calibration Plots for Catchment BO1A



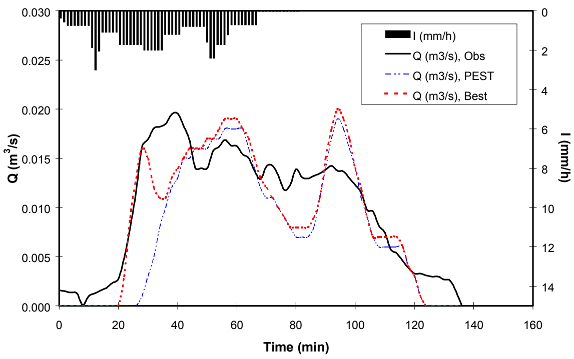
(a) Event C1



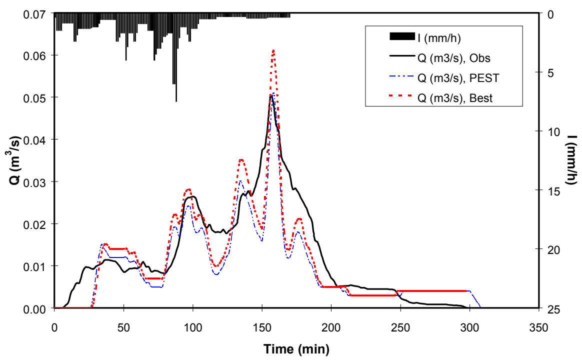
(b) Event C2



(c) Event C3

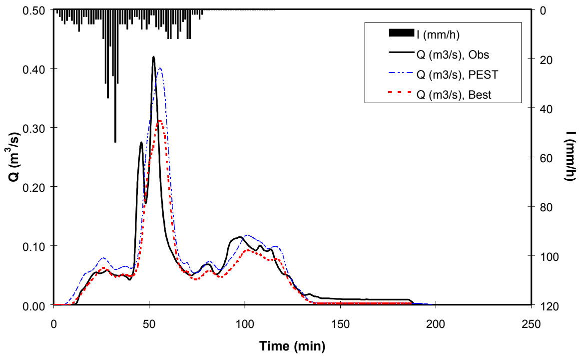


(d) Event C4

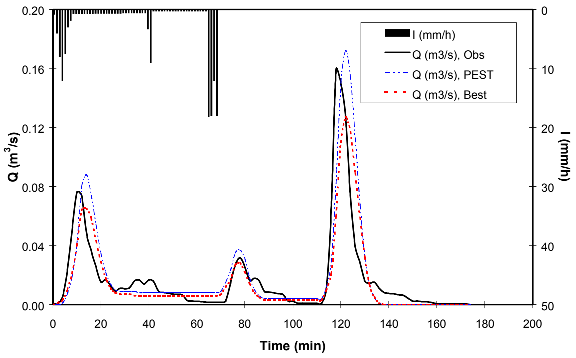


(e) Event C5

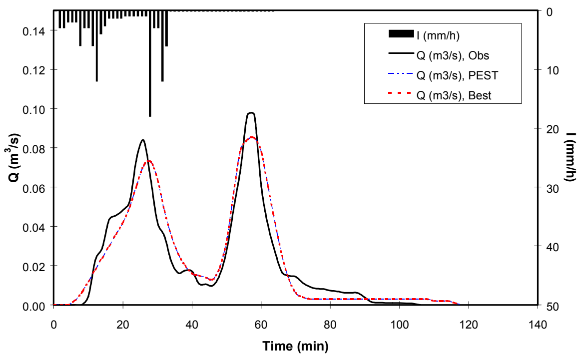
Figure E7: Calibration Plots for Catchment BO2A



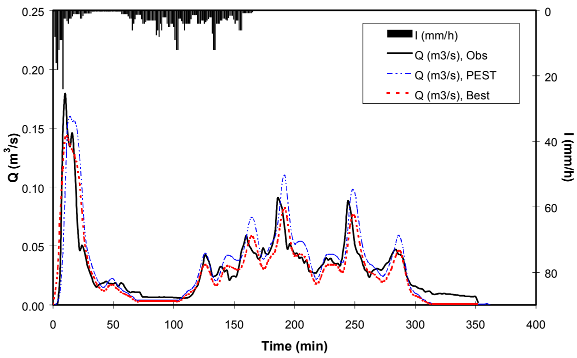
(a) Event C1



(b) Event C2

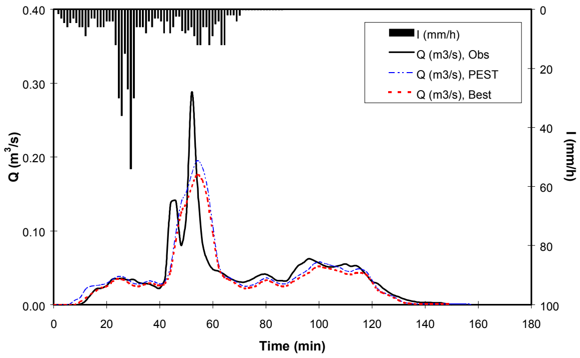


(c) Event C3

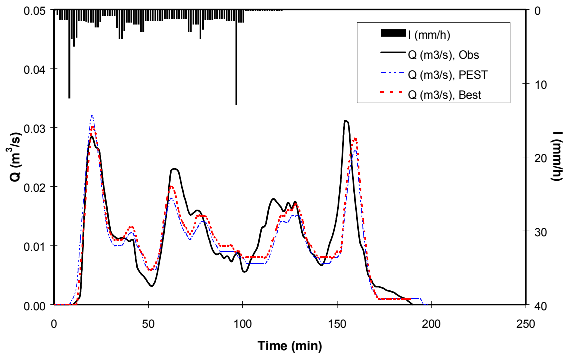


(d) Event C4

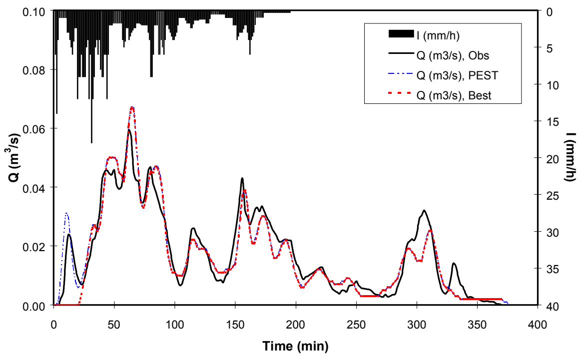
Figure E8: Calibration Plots for Catchment BR1



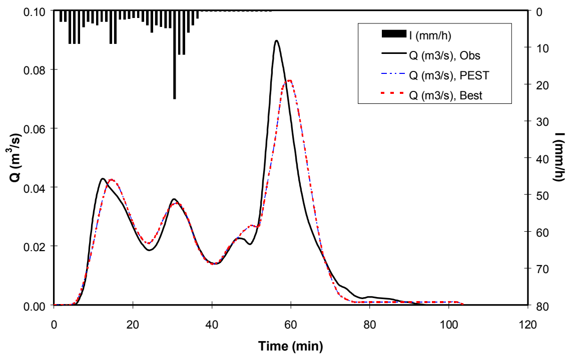
(a) Event C1



(b) Event C2

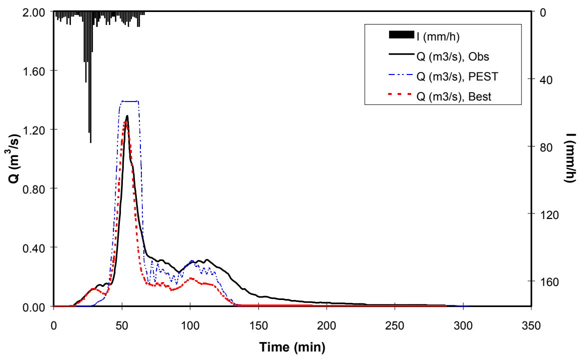


(c) Event C3

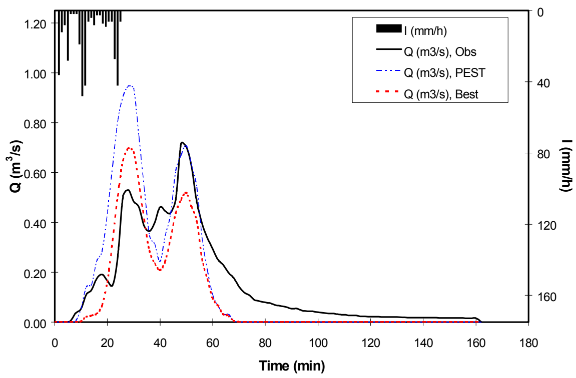


(d) Event C4

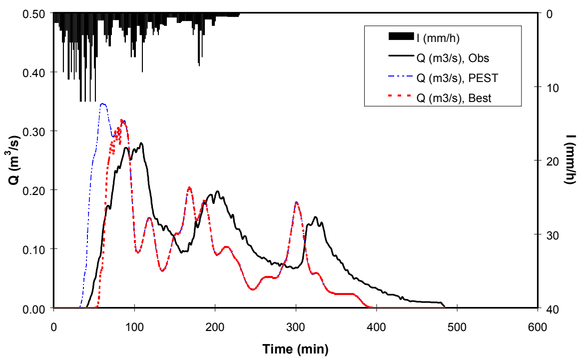
Figure E9: Calibration Plots for Catchment BR1A



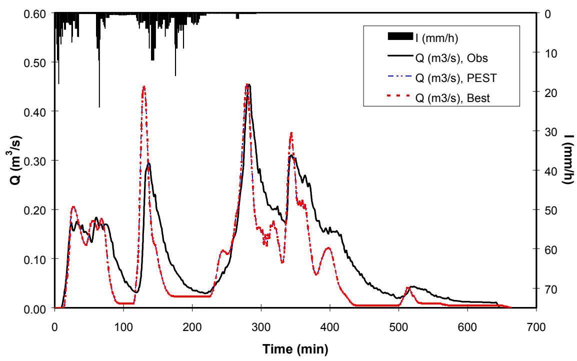
(a) Event C1



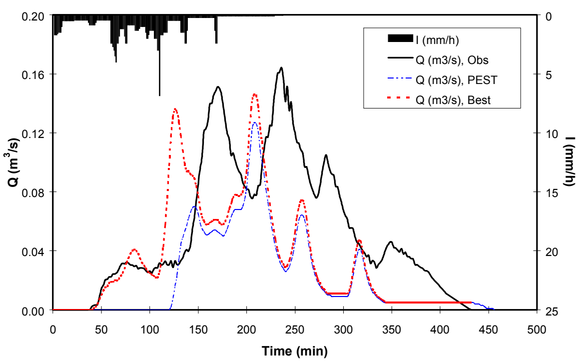
(b) Event C2



(c) Event C3

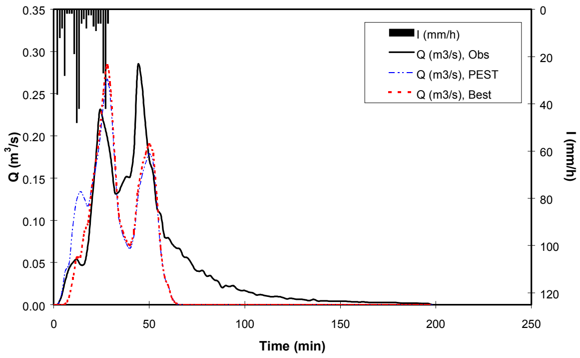


(d) Event C4

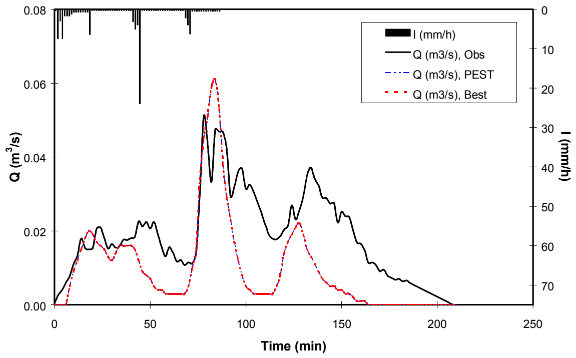


(e) Event C5

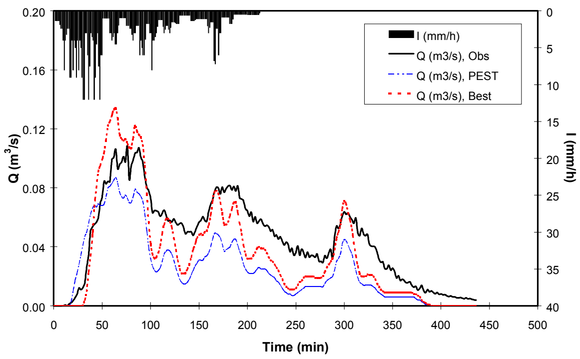
Figure E10: Calibration Plots for Catchment BR2



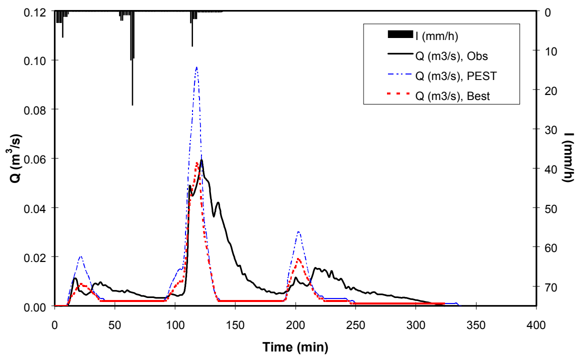
(a) Event C1



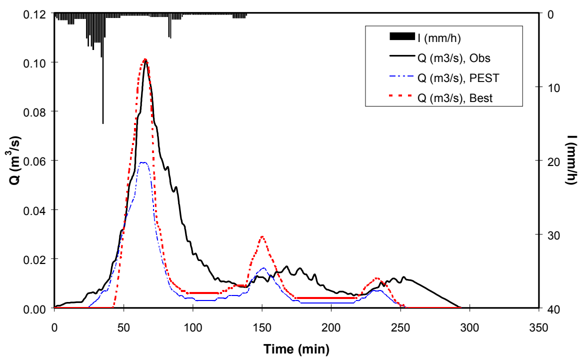
(b) Event C2



(c) Event C3

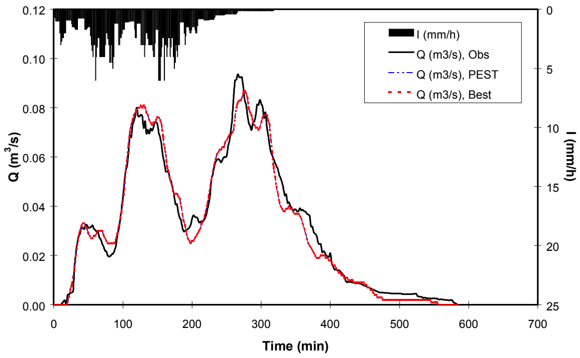


(d) Event C4

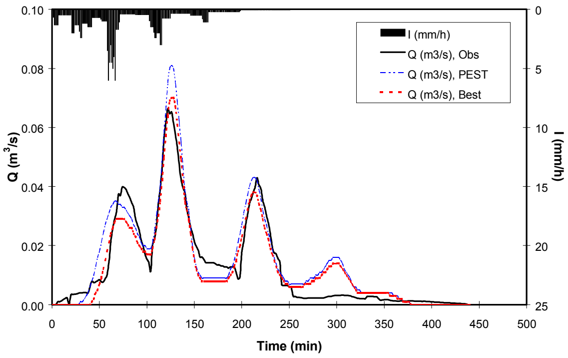


(e) Event C5

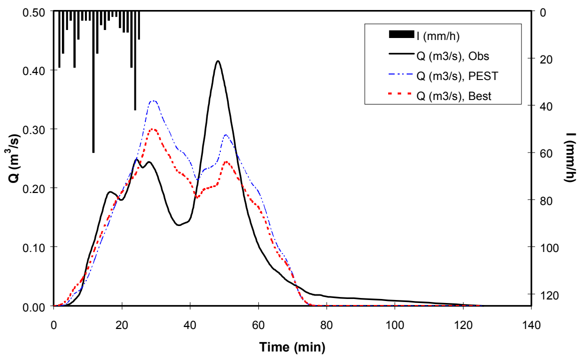
Figure E11: Calibration Plots for Catchment BR2A



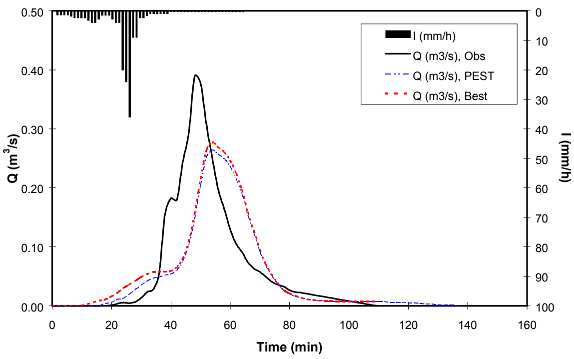
(a) Event C1



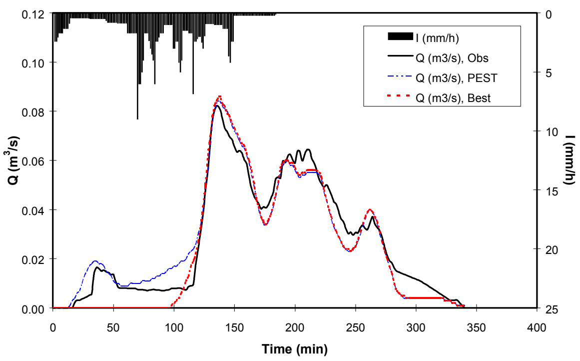
(b) Event C2



(c) Event C3

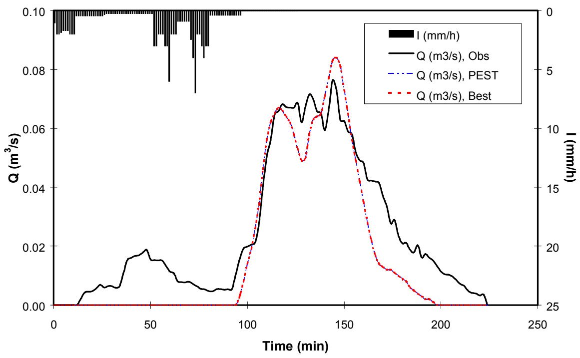


(d) Event C4

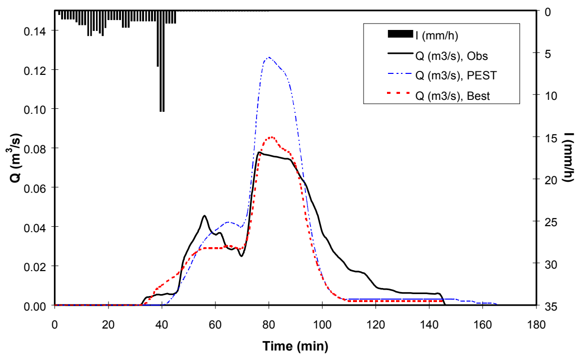


(e) Event C5

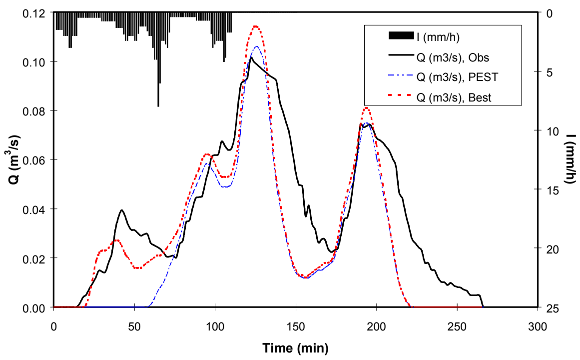
Figure E12: Calibration Plots for Catchment BR3



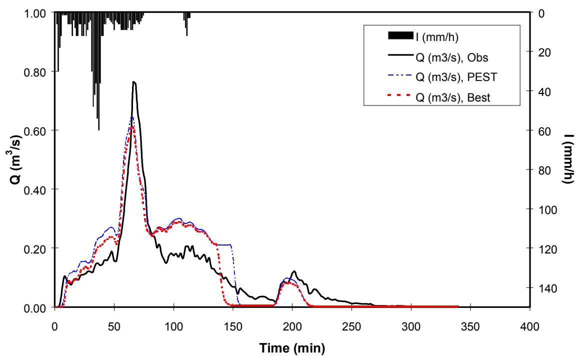
(a) Event C1



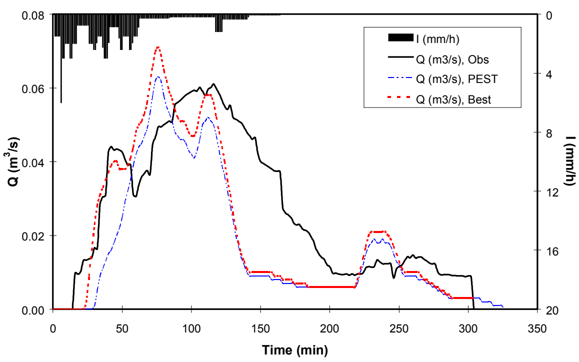
(b) Event C2



(c) Event C3

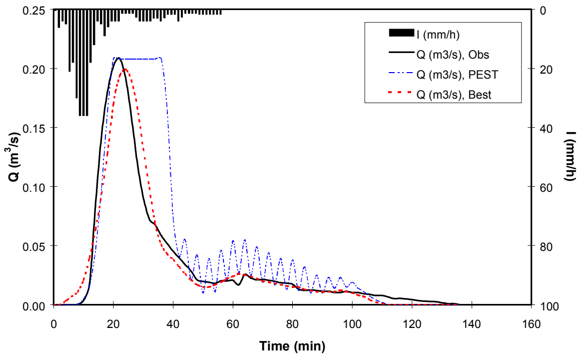


(d) Event C4

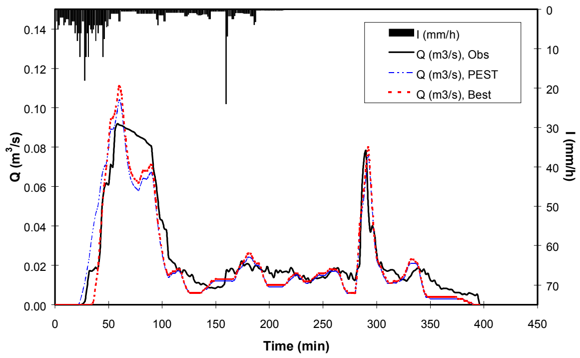


(e) Event C5

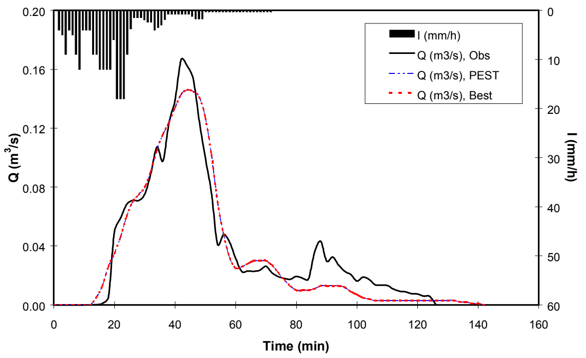
Figure E13: Calibration Plots for Catchment H2



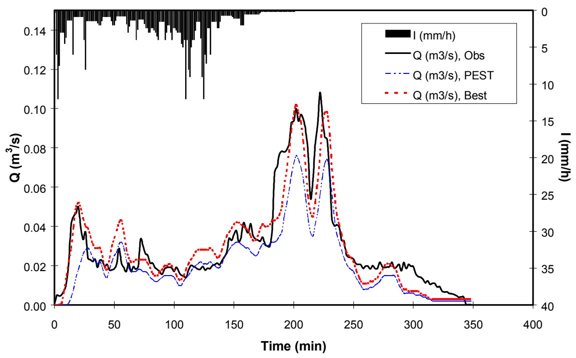
(a) Event C1



(b) Event C2

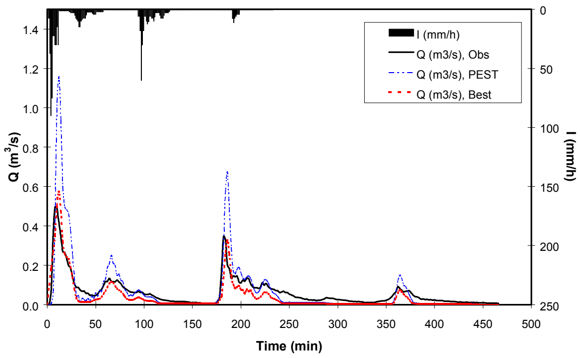


(c) Event C3

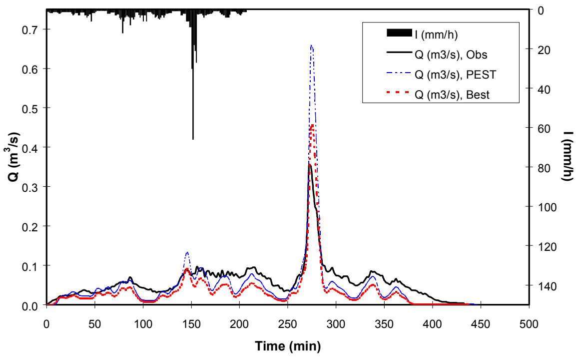


(d) Event C4

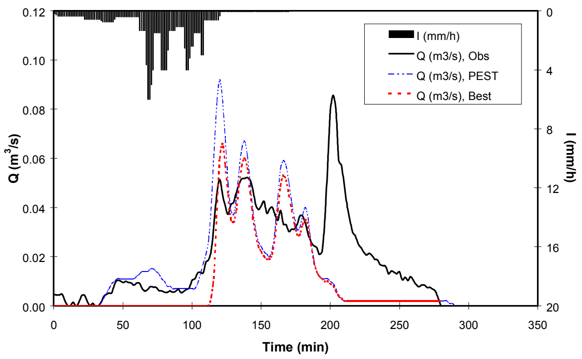
Figure E14: Calibration Plots for Catchment H2A



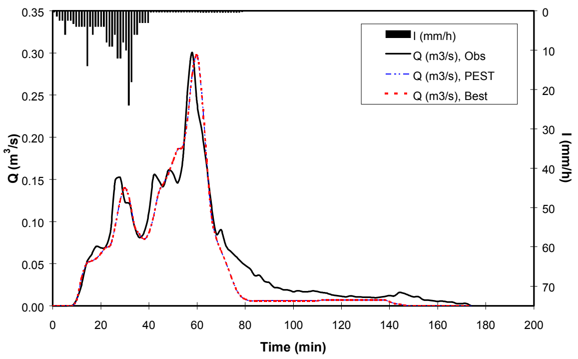
(a) Event C1



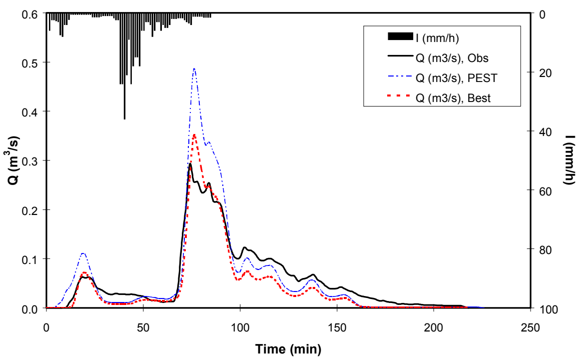
(b) Event C2



(c) Event C3

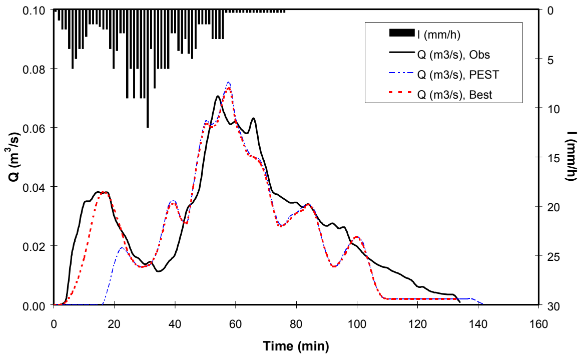


(d) Event C4

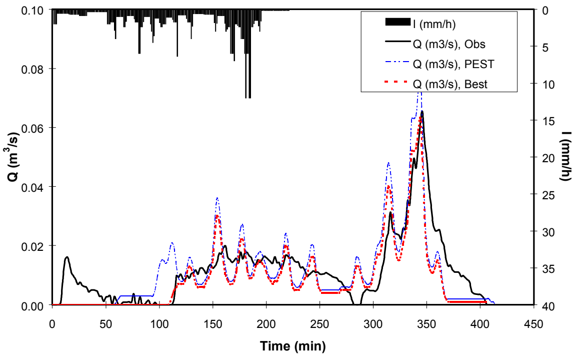


(e) Event C5

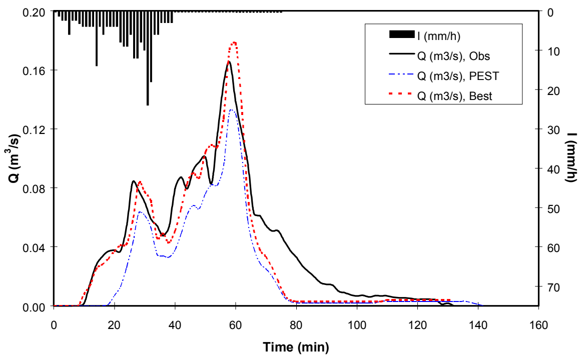
Figure E15: Calibration Plots for Catchment K1



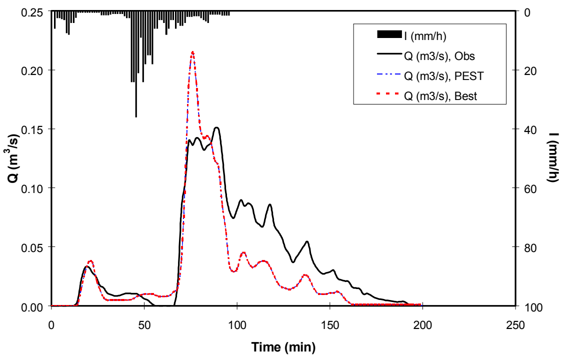
(a) Event C1



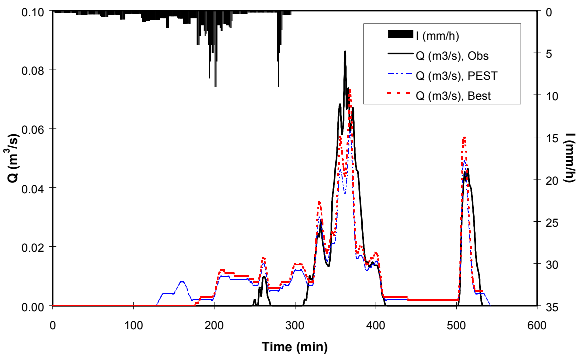
(b) Event C2



(c) Event C3

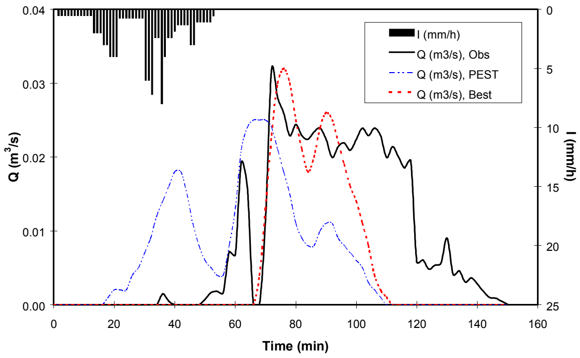


(d) Event C4

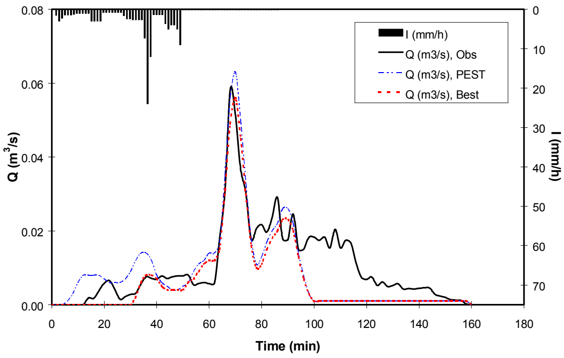


(e) Event C5

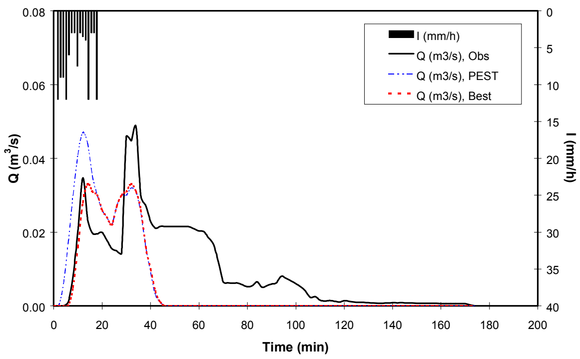
Figure E16: Calibration Plots for Catchment K1A



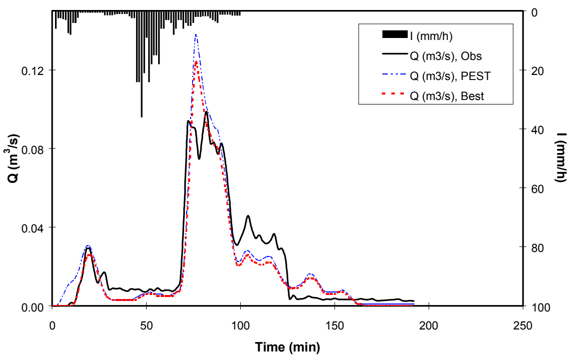
(a) Event C1



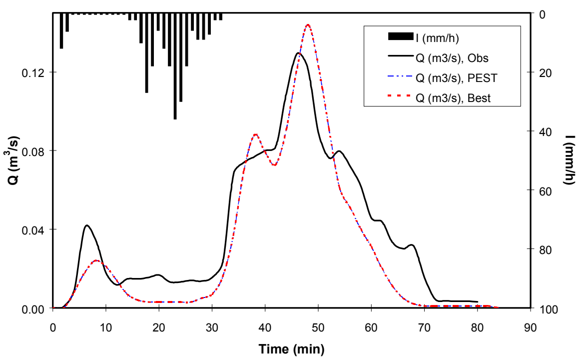
(b) Event C2



(c) Event C3

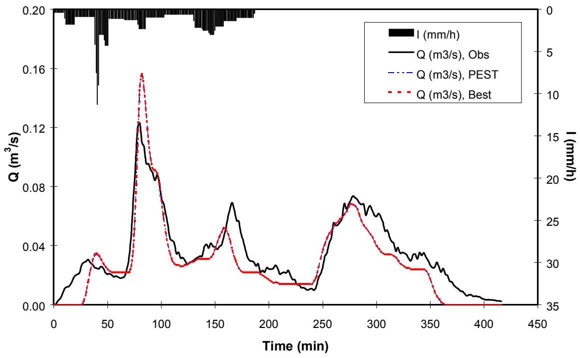


(d) Event C4

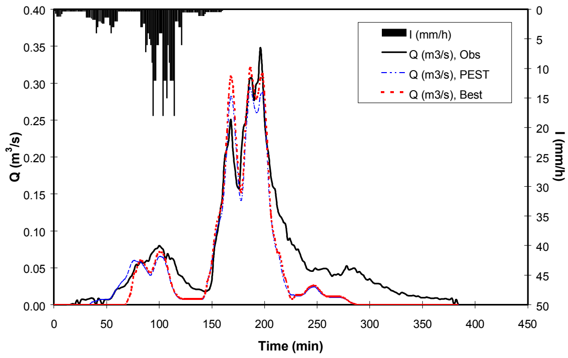


(e) Event C5

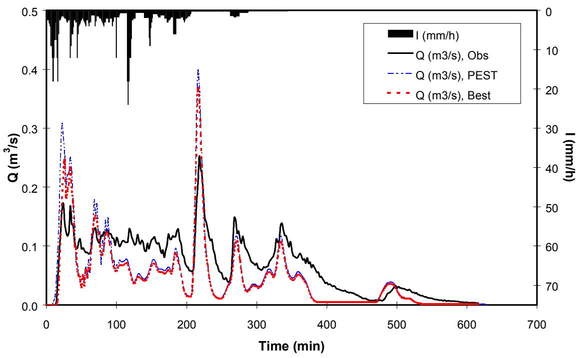
Figure E17: Calibration Plots for Catchment K1B



(a) Event C1

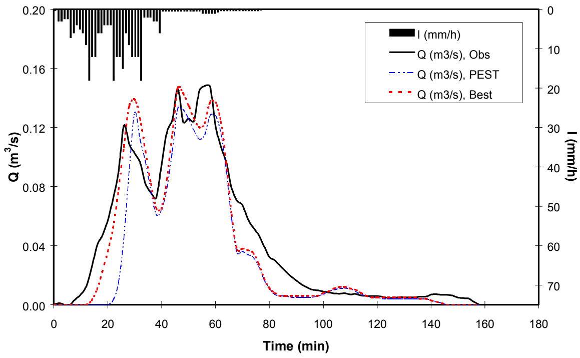


(b) Event C2

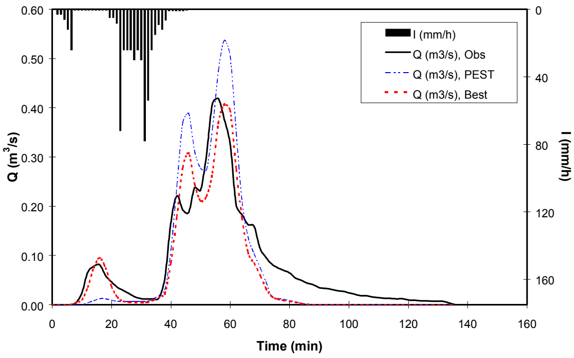


(c) Event C3

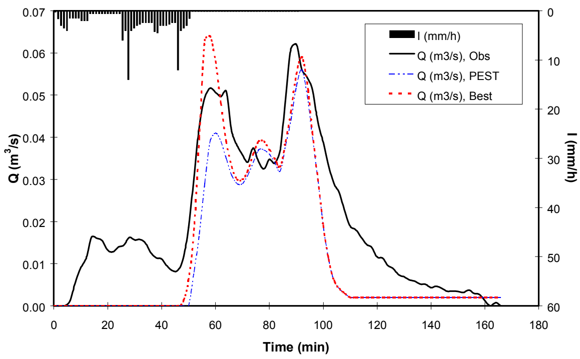
Figure E18: Calibration Plots for Catchment K2



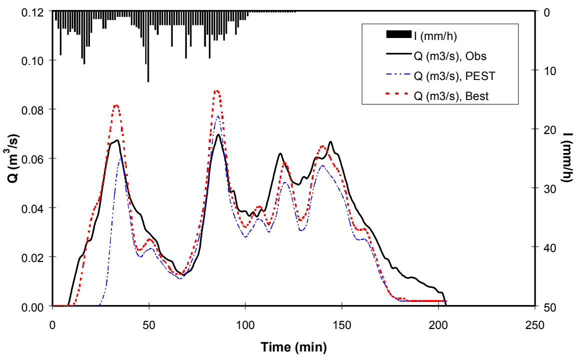
(a) Event C1



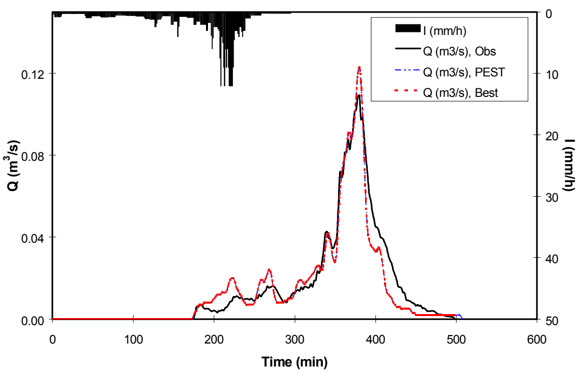
(b) Event C2



(c) Event C3

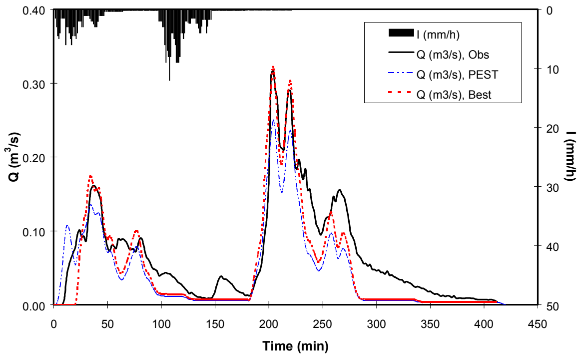


(d) Event C4

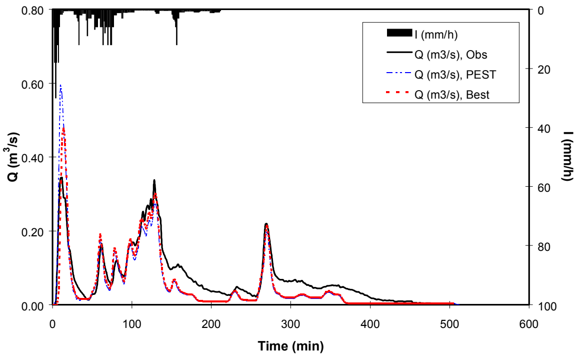


(e) Event C5

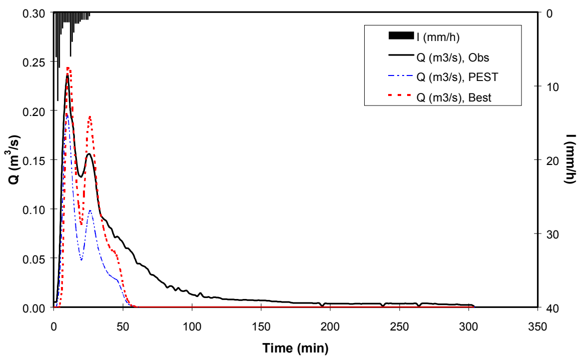
Figure E19: Calibration Plots for Catchment K2A



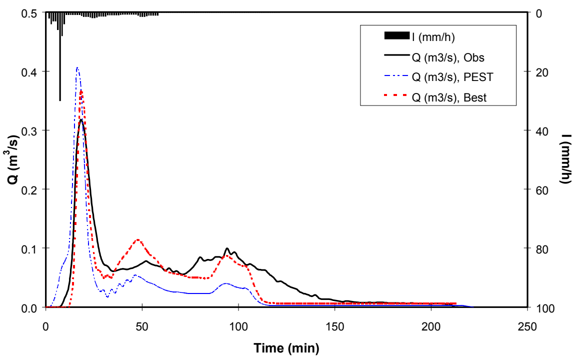
(a) Event C1



(b) Event C2

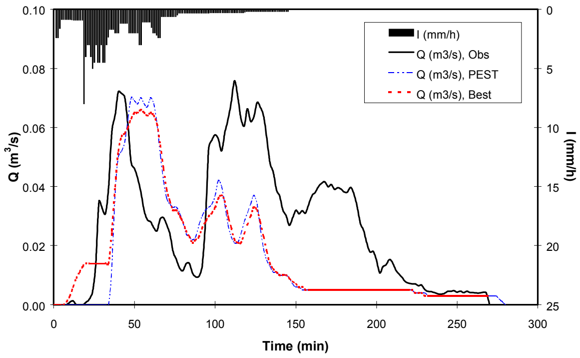


(c) Event C3

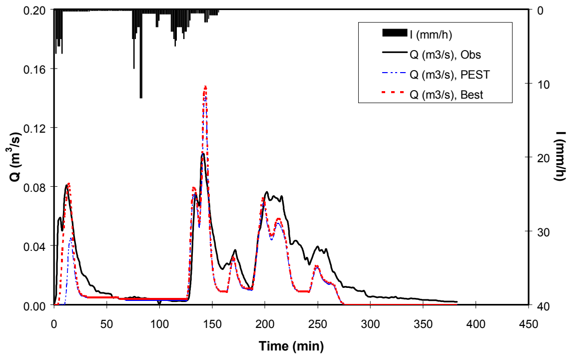


(d) Event C4

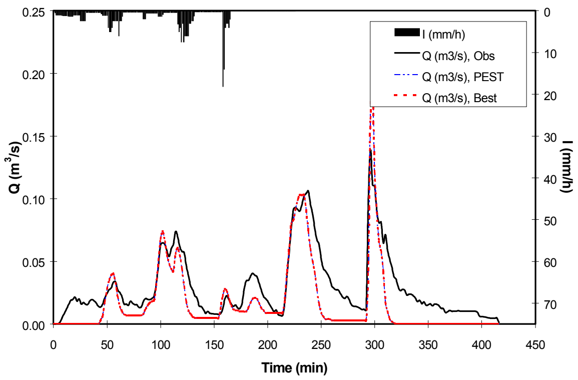
Figure E20: Calibration Plots for Catchment K3



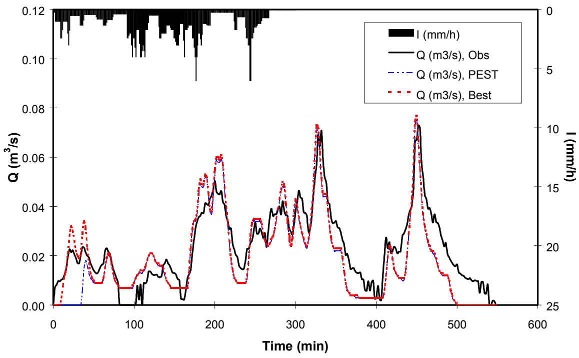
(a) Event C1



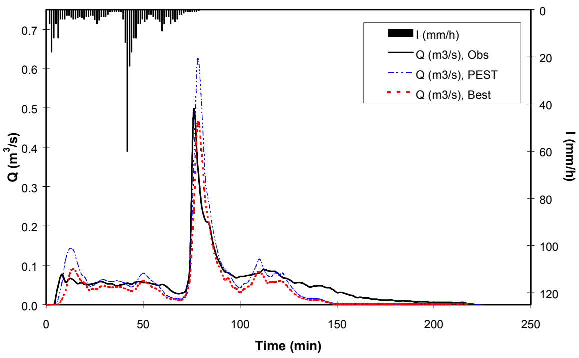
(b) Event C2



(c) Event C3

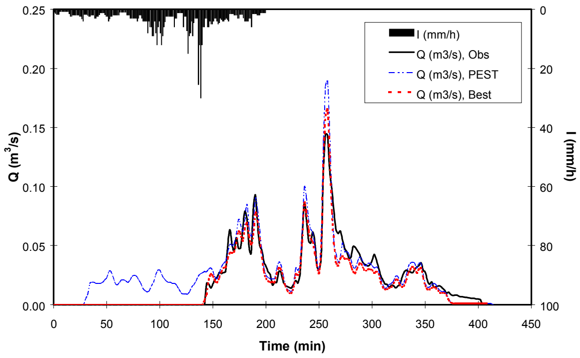


(d) Event C4

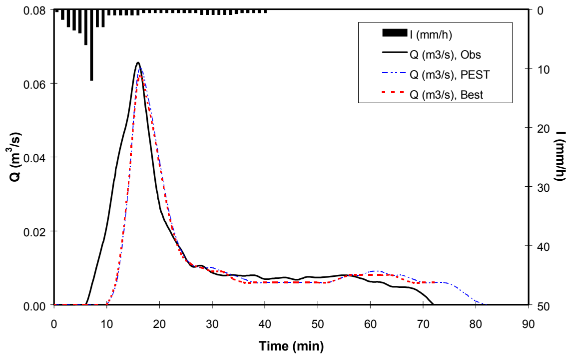


(e) Event C5

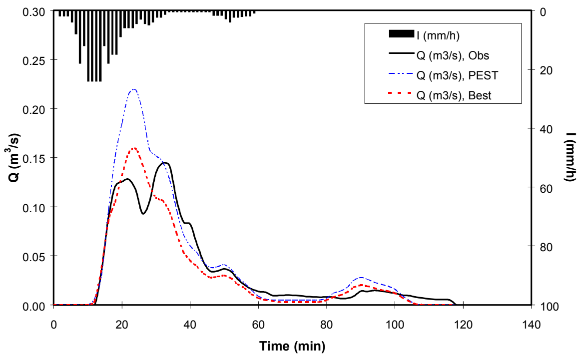
Figure E21: Calibration Plots for Catchment K3A



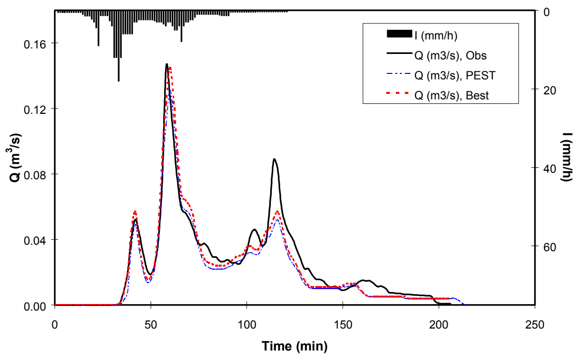
(a) Event C1



(b) Event C2

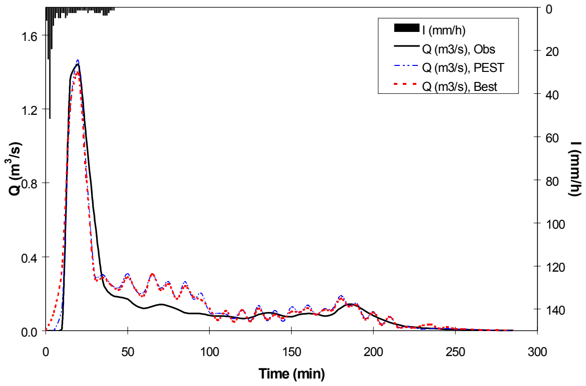


(c) Event C3

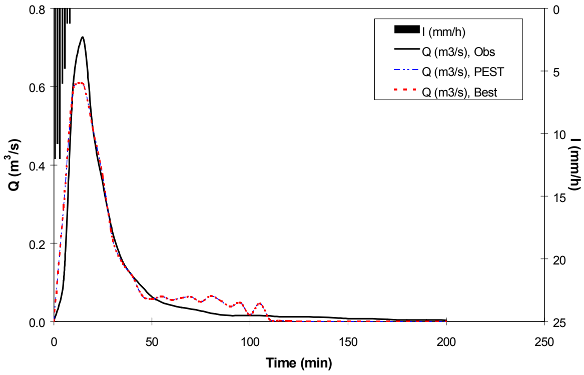


(d) Event C4

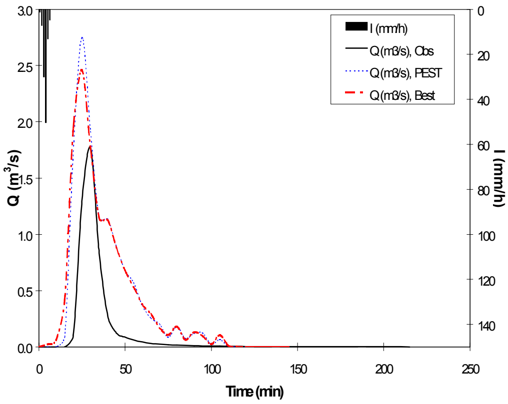
Figure E22: Calibration Plots for Catchment K3B



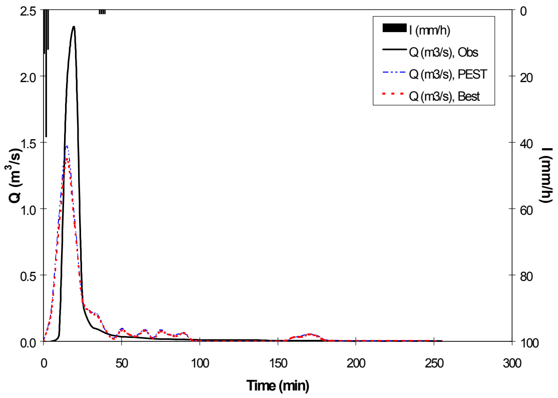
(a) Event CS1



(b) Event CS2

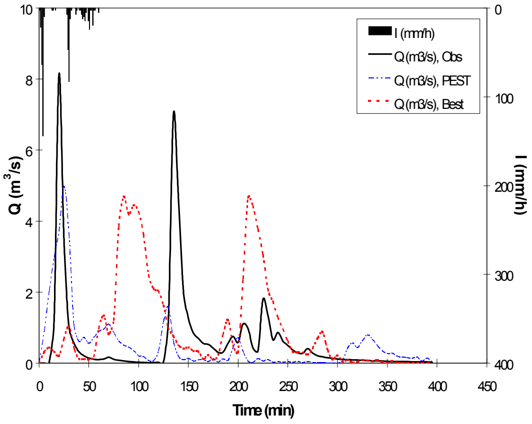


(c) Event CS3

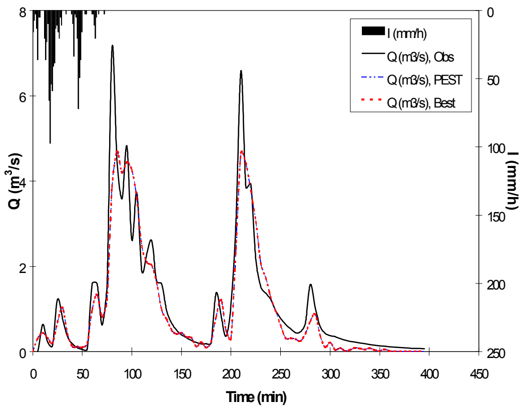


(d) Event CS4

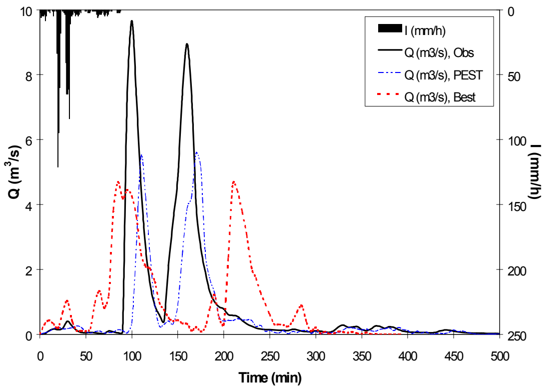
Figure E23: Calibration Plots for Giralang Catchment for 'Small' Events



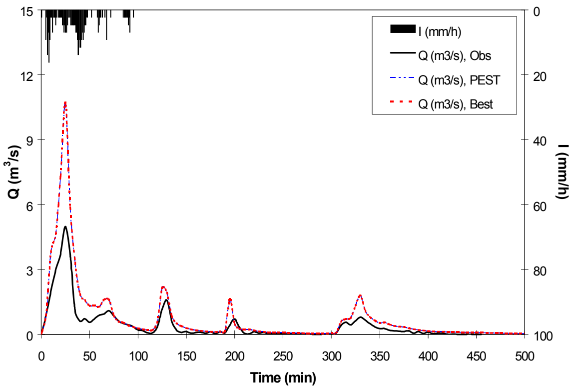
(a) Event CL1



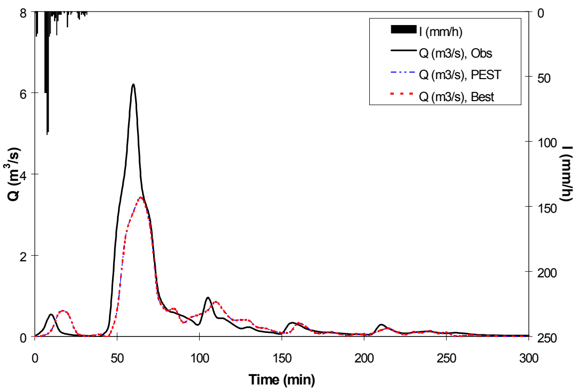
(b) Event CL2



(c) Event CL3



(d) Event CL4

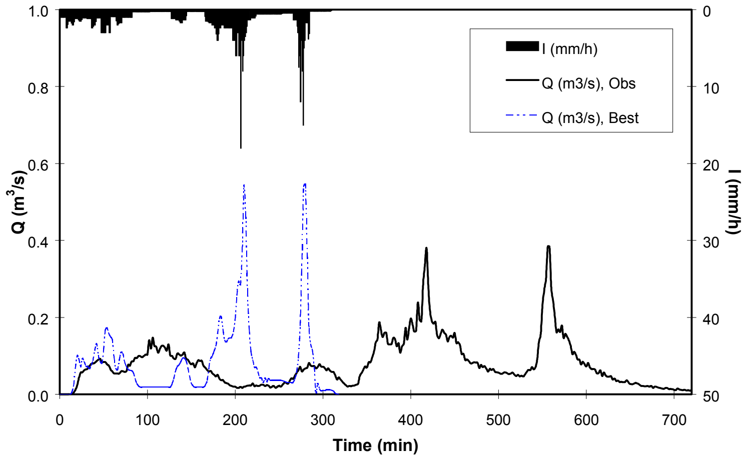


(e) Event CL5

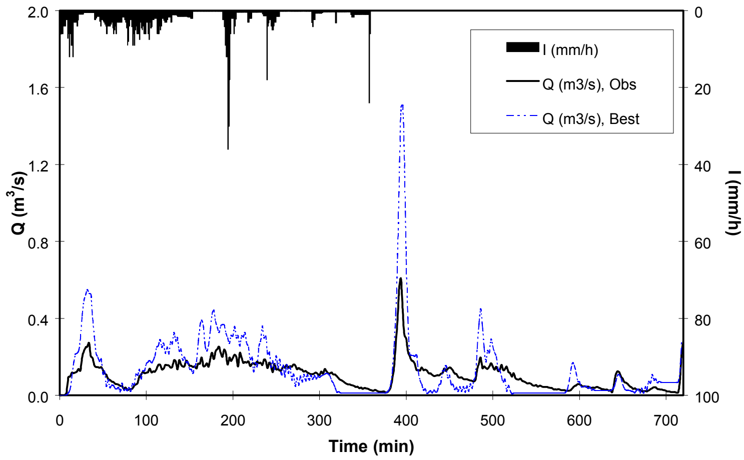
Figure E24: Calibration Plots for Giralang Catchment for ‘Large’ Events

APPENDIX F

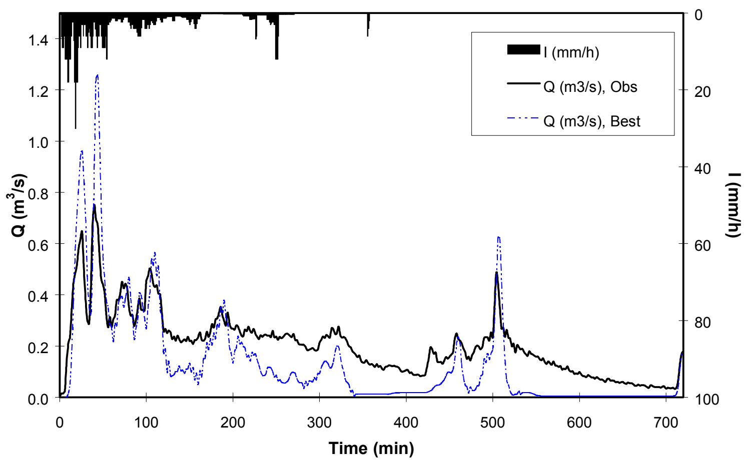
VERIFICATION PLOTS



(a) Event V1

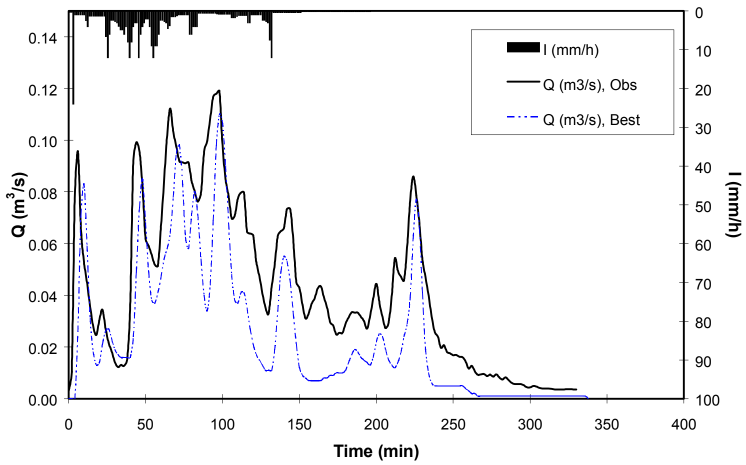


(b) Event V2

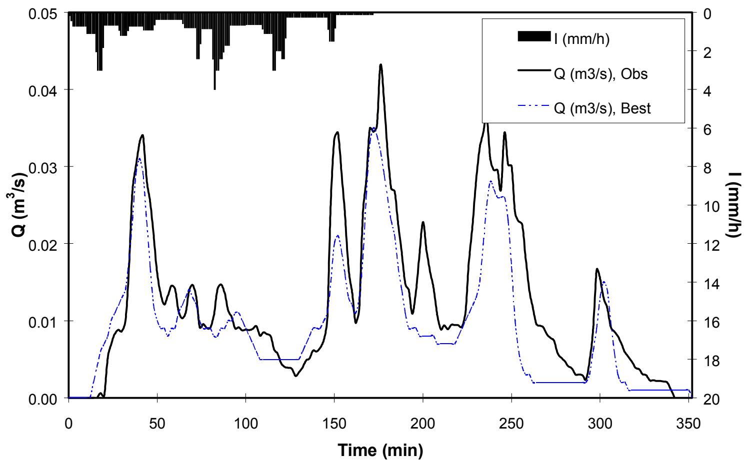


(c) Event V3

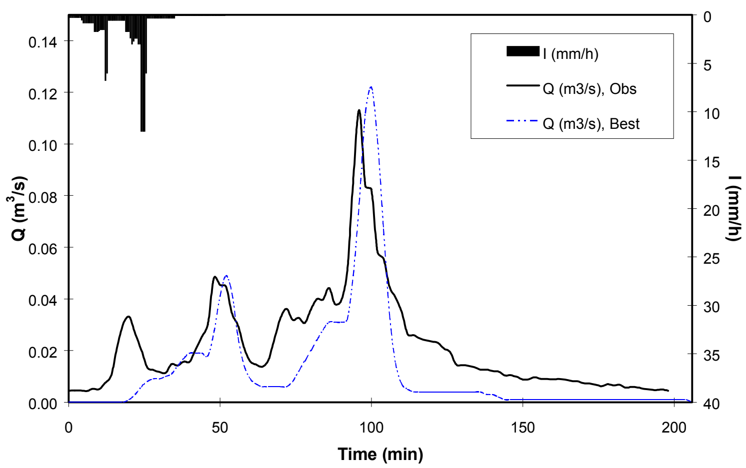
Figure F1: Verification Plots for Catchment BA2



(c) Event V1

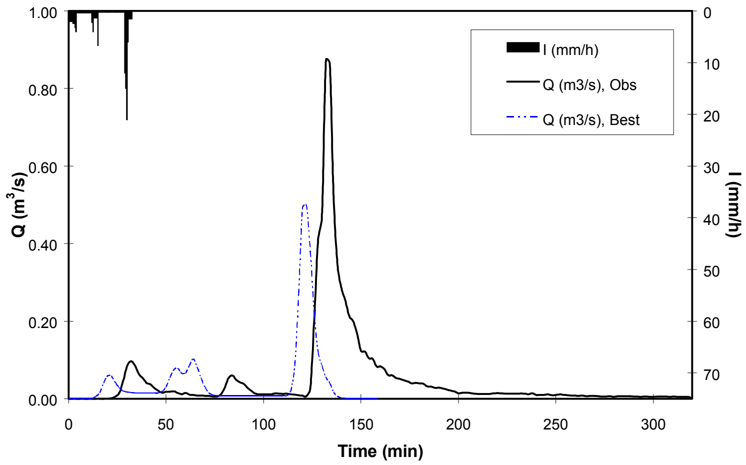


(d) Event V2

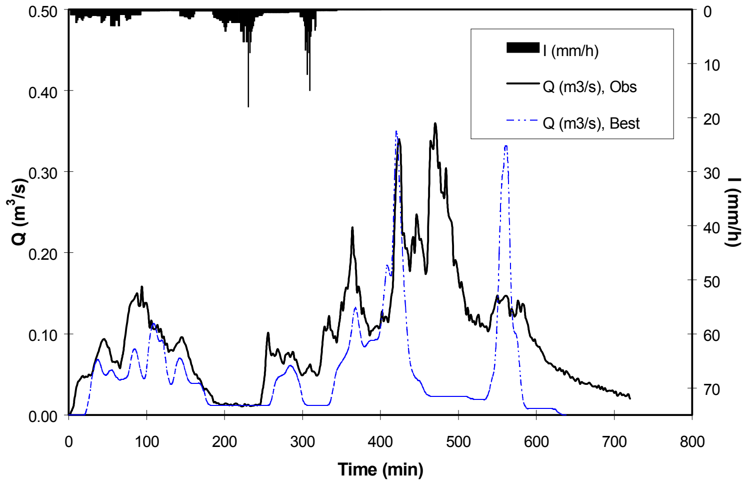


(e) Event V3

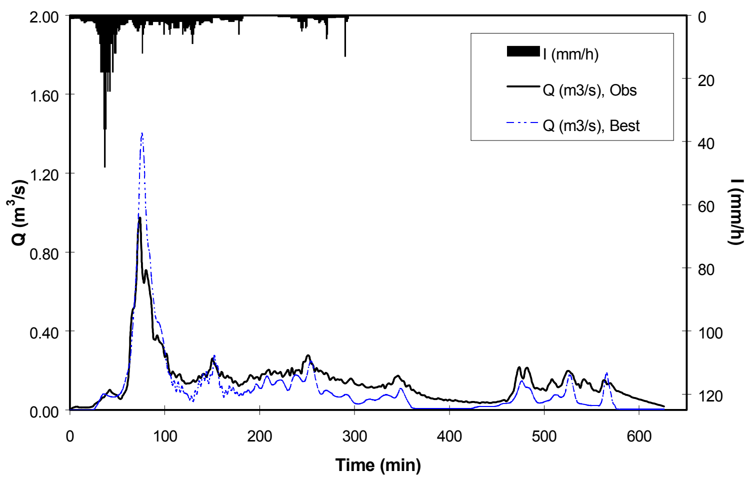
Figure F2: Verification Plots for Catchment BA2A



(a) Event V1

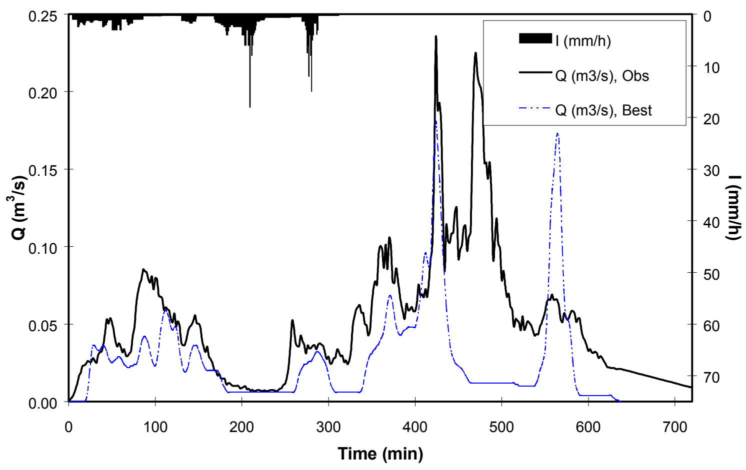


(b) Event V2

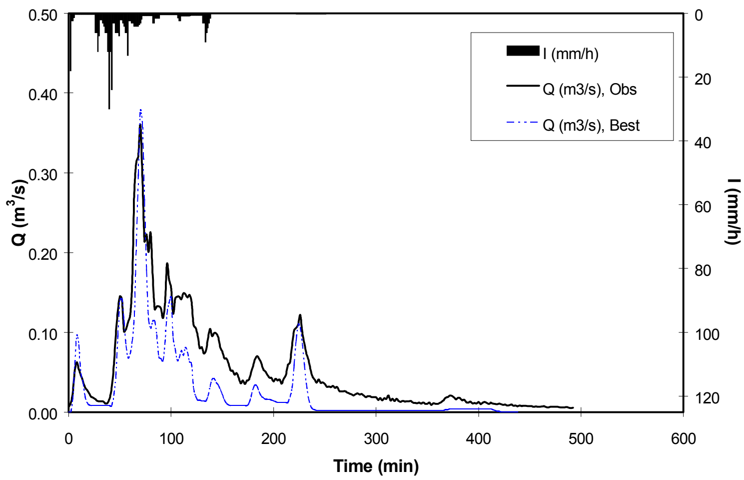


(c) Event V3

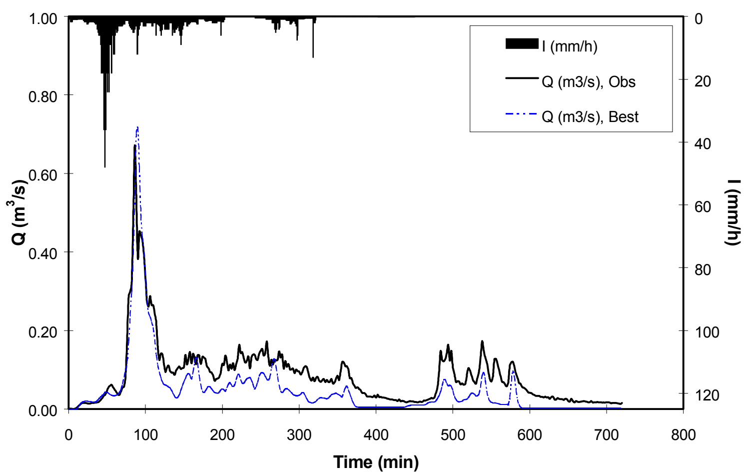
Figure F3: Verification Plots for Catchment BA3



(a) Event V1

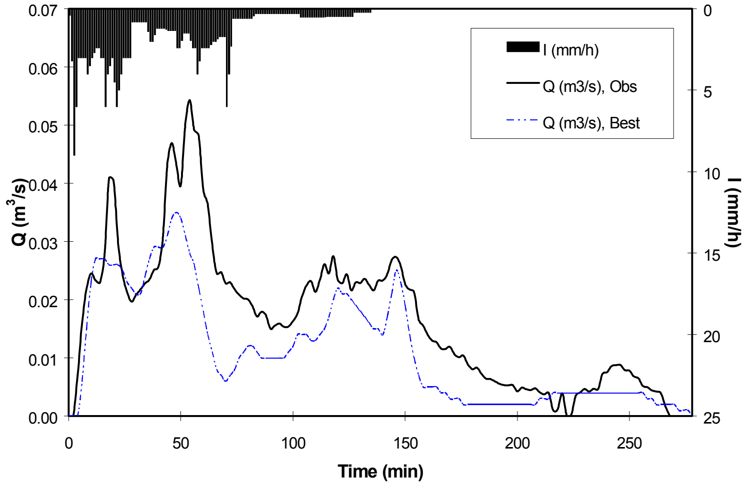


(b) Event V2

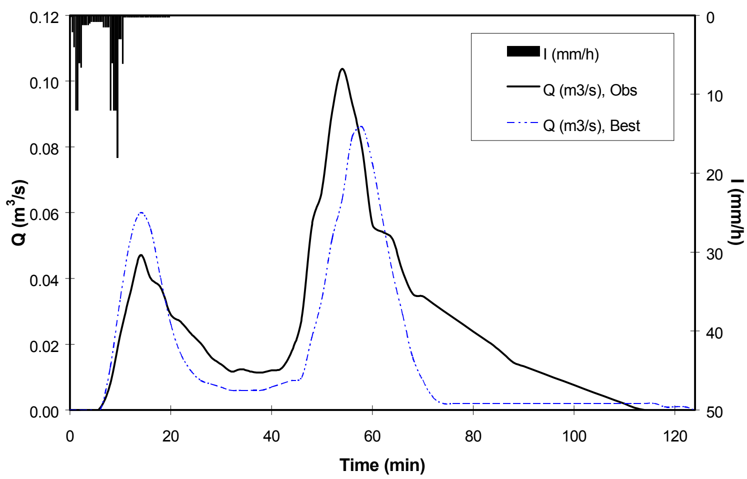


(c) Event V3

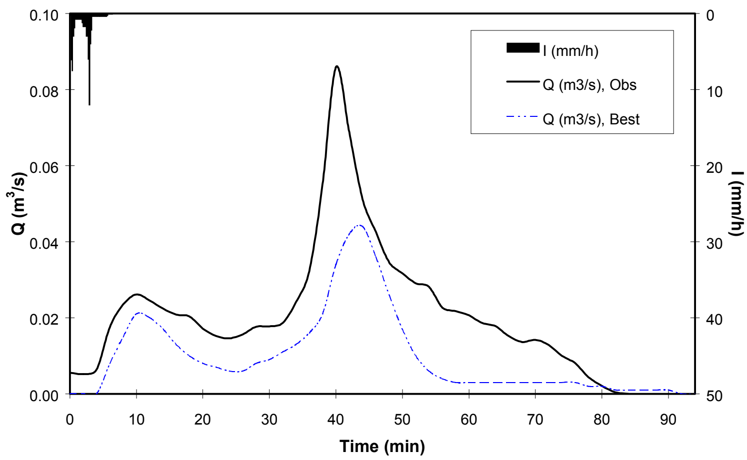
Figure F4: Verification Plots for Catchment BA3A



(a) Event V1

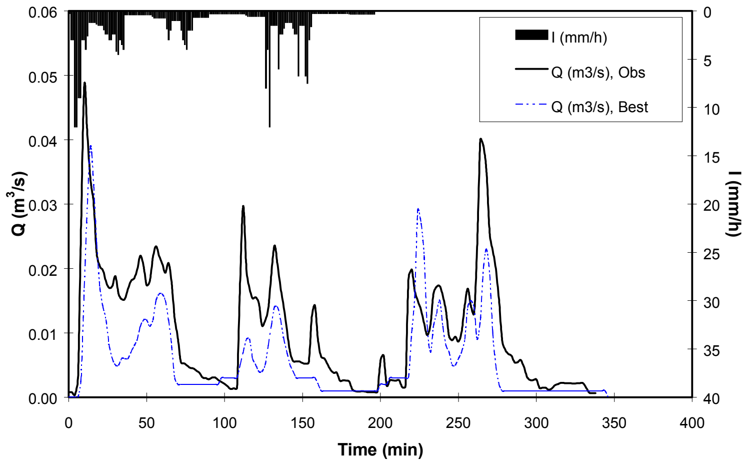


(b) Event V2

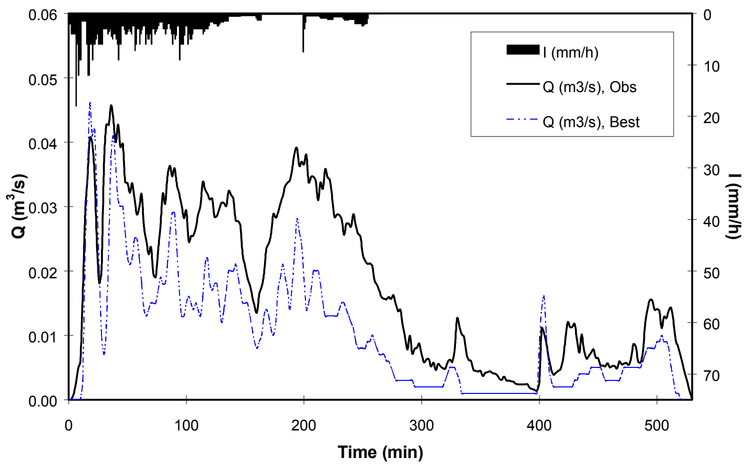


(c) Event V3

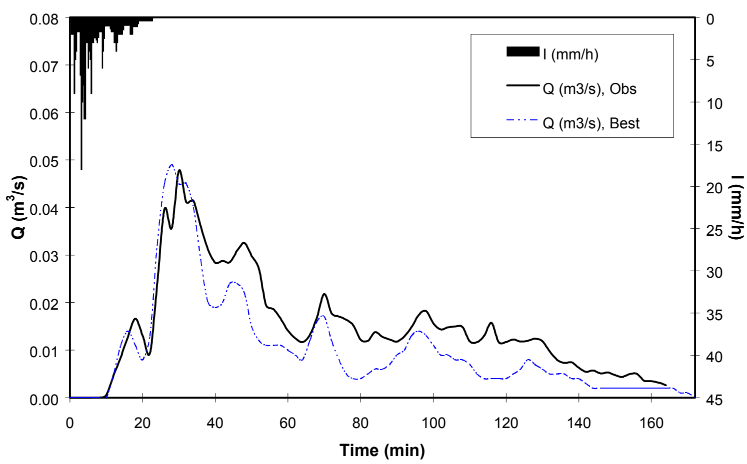
Figure F5: Verification Plots for Catchment BA3B



(a) Event V1

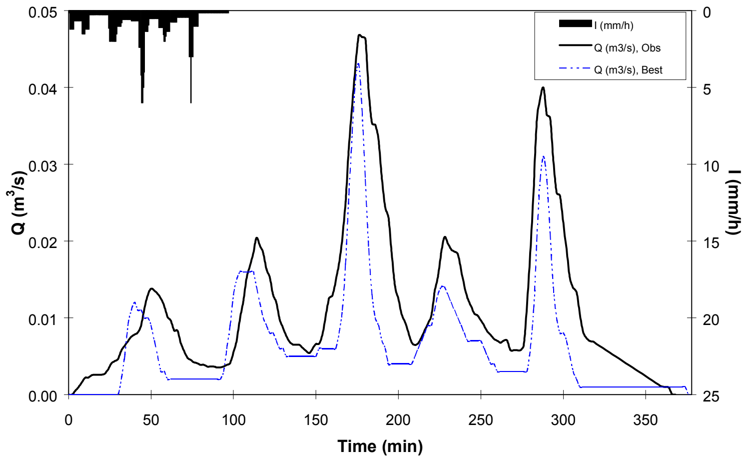


(b) Event V2

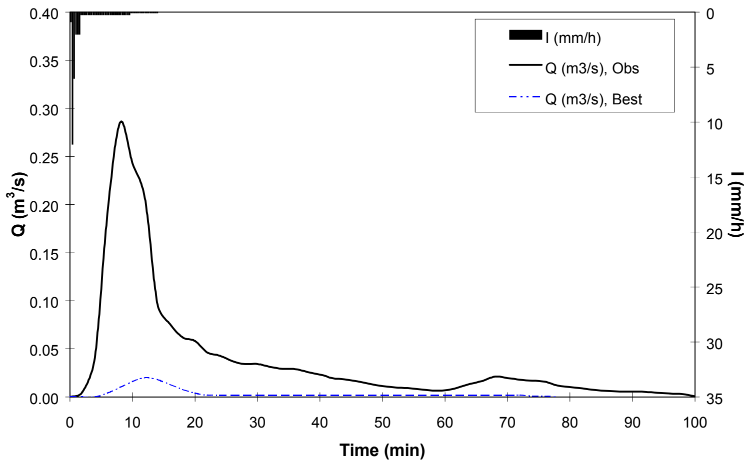


(c) Event V3

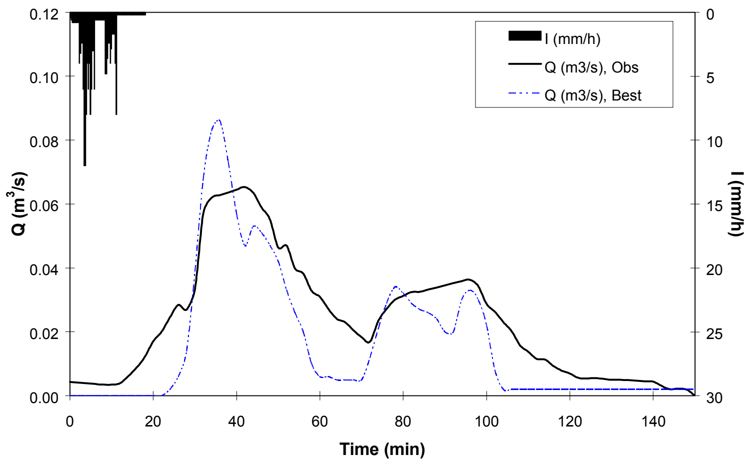
Figure F6: Verification Plots for Catchment BO1A



(a) Event V1

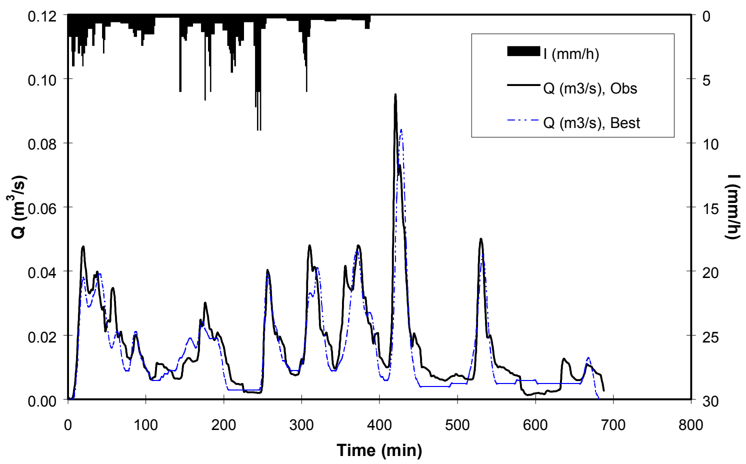


(b) Event V2

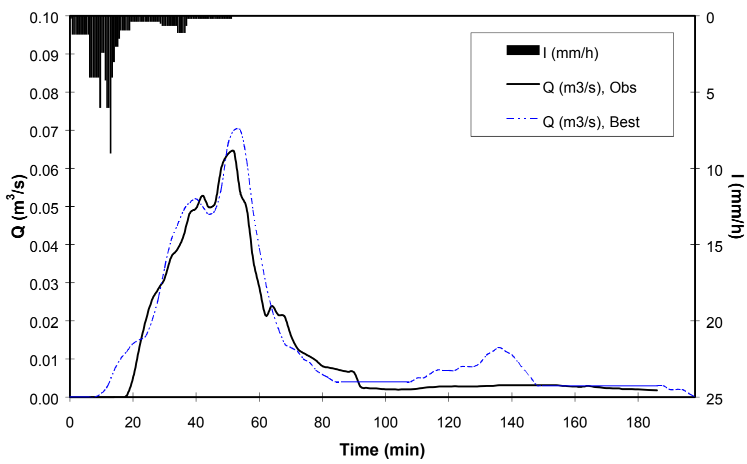


(c) Event V3

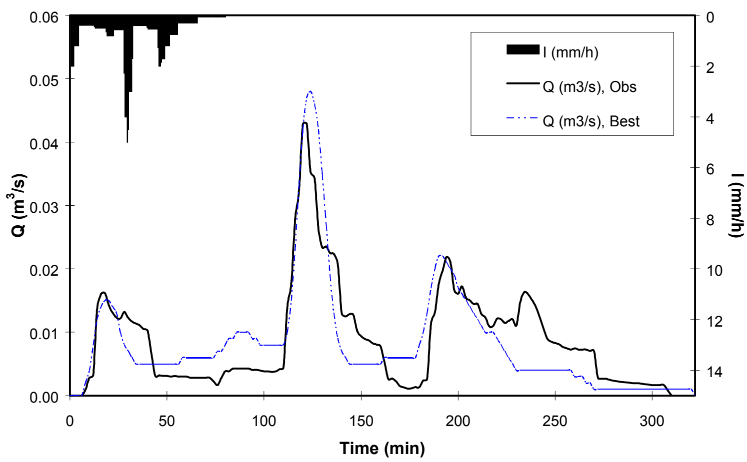
Figure F7: Verification Plots for Catchment BO2A



(a) Event V1

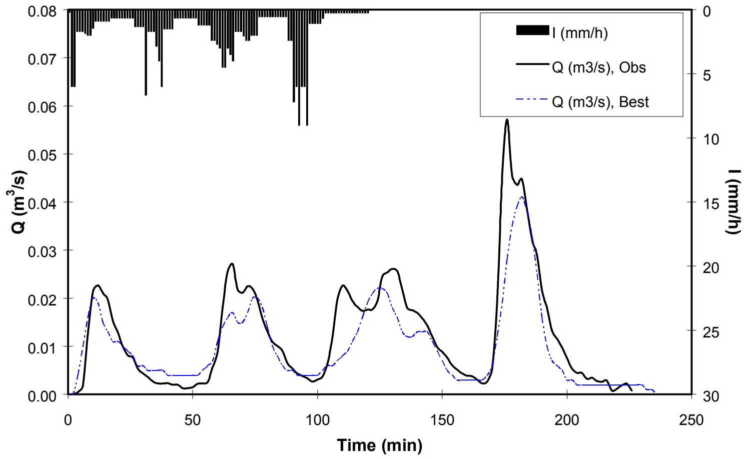


(b) Event V2

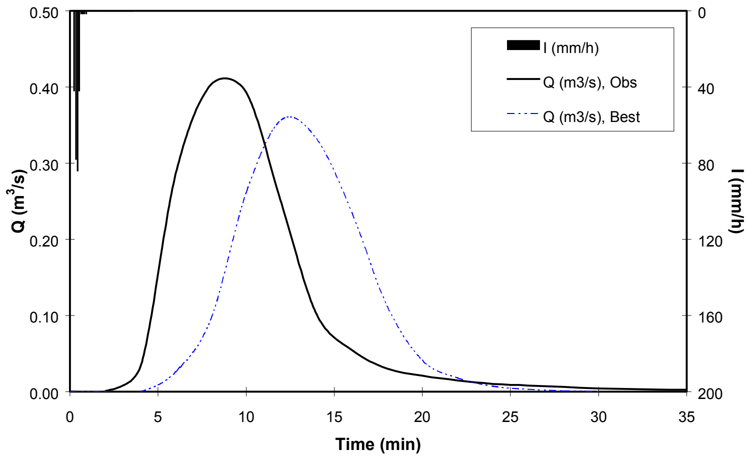


(c) Event V3

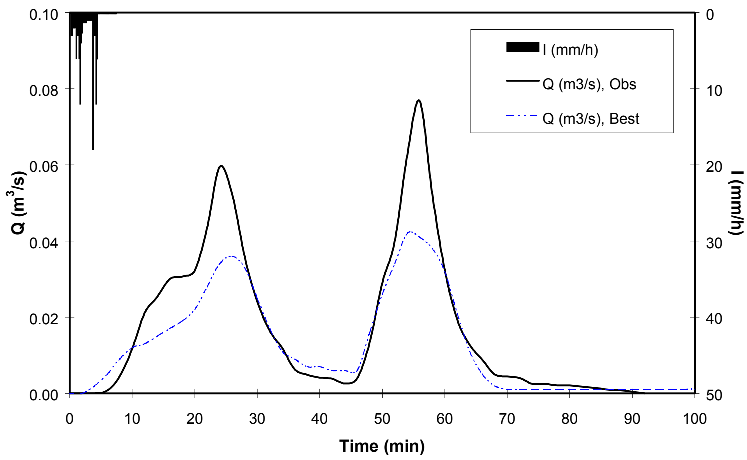
Figure F8: Verification Plots for Catchment BR1



(a) Event V1

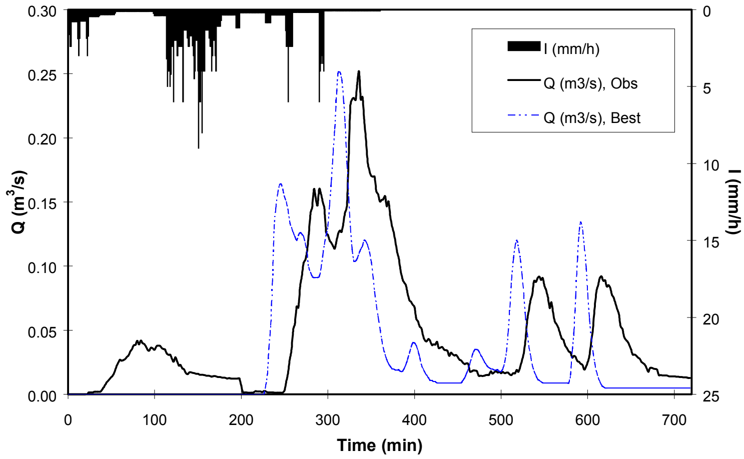


(b) Event V2

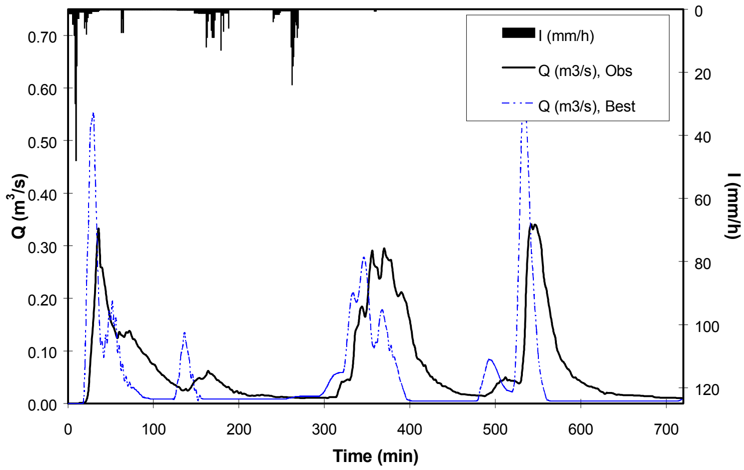


(c) Event V3

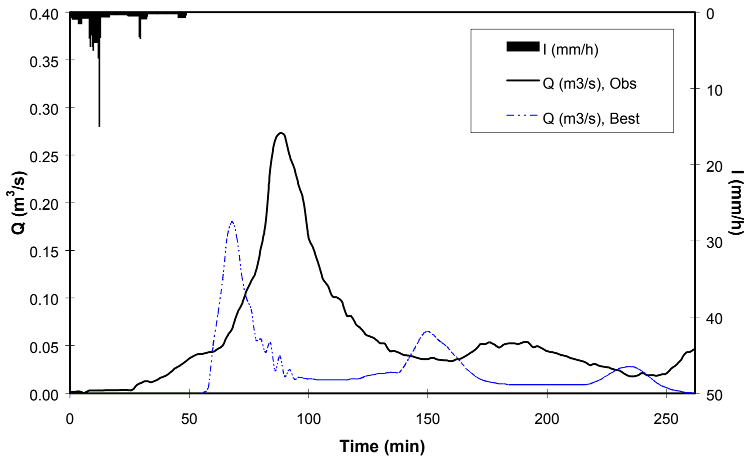
Figure F9: Verification Plots for Catchment BR1A



(a) Event V1

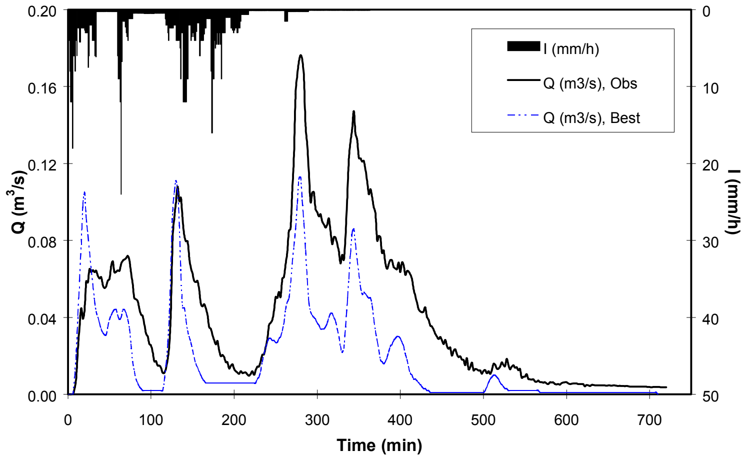


(b) Event V2

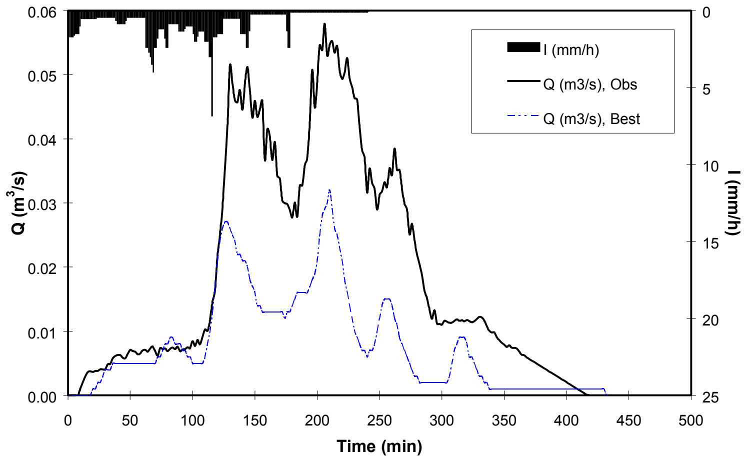


(c) Event V3

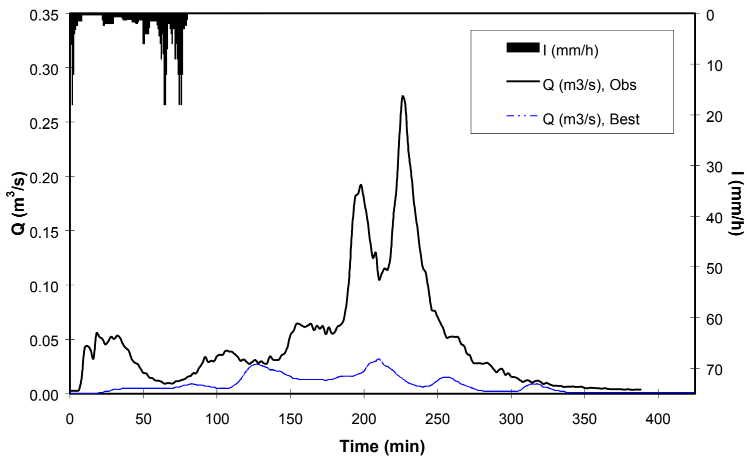
Figure F10: Verification Plots for Catchment BR2



(a) Event V1

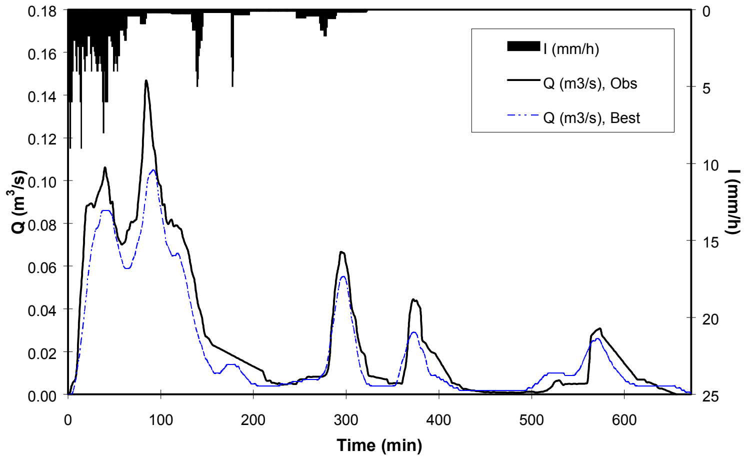


(b) Event V2

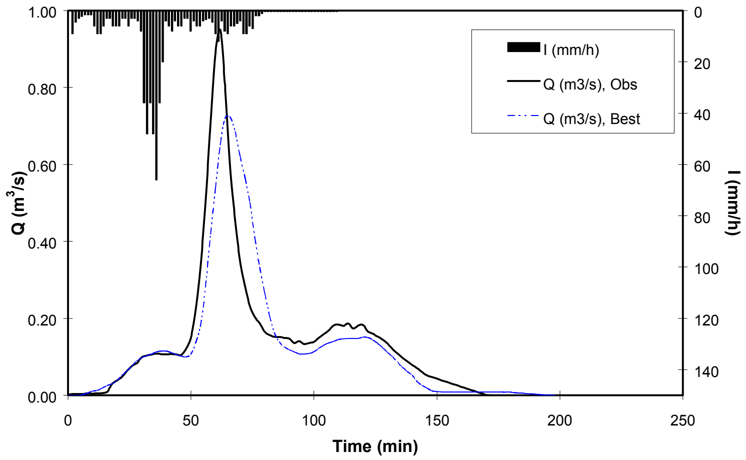


(c) Event V3

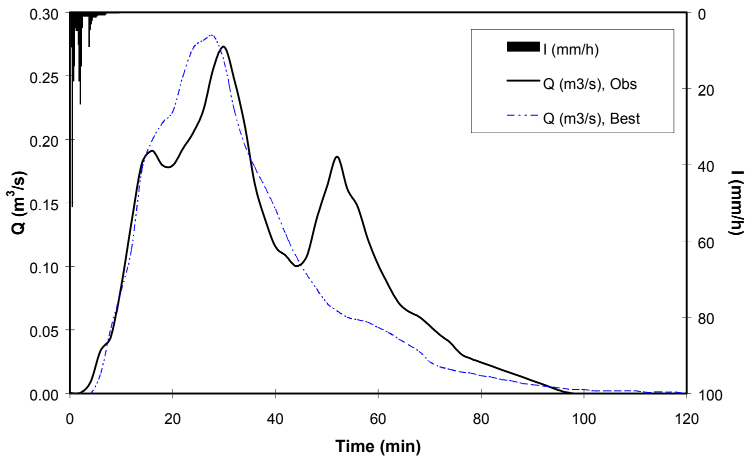
Figure F11: Verification Plots for Catchment BR2A



(a) Event V1

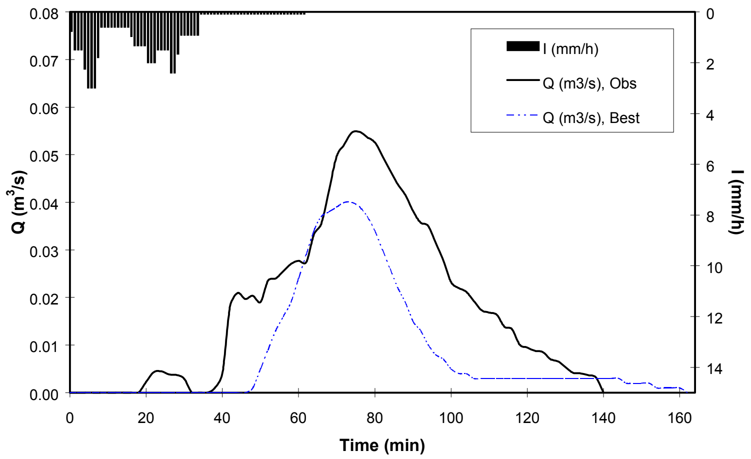


(b) Event V2

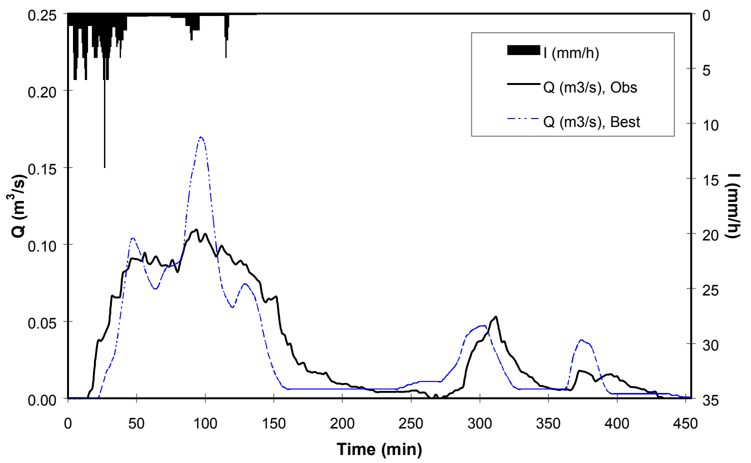


(c) Event V3

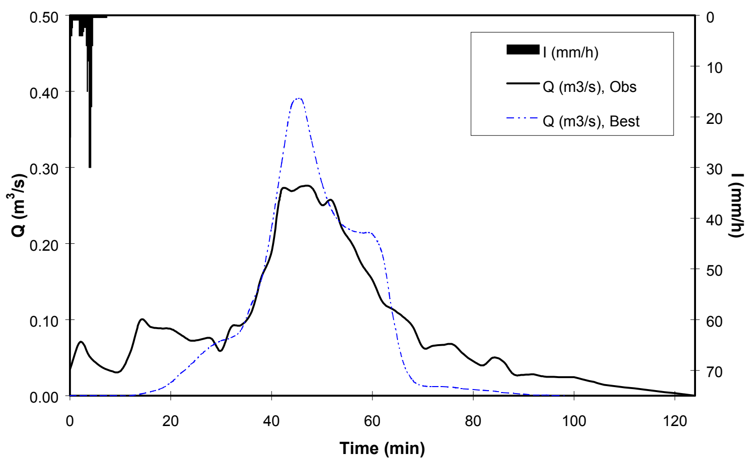
Figure F12: Verification Plots for Catchment BR3



(a) Event V1

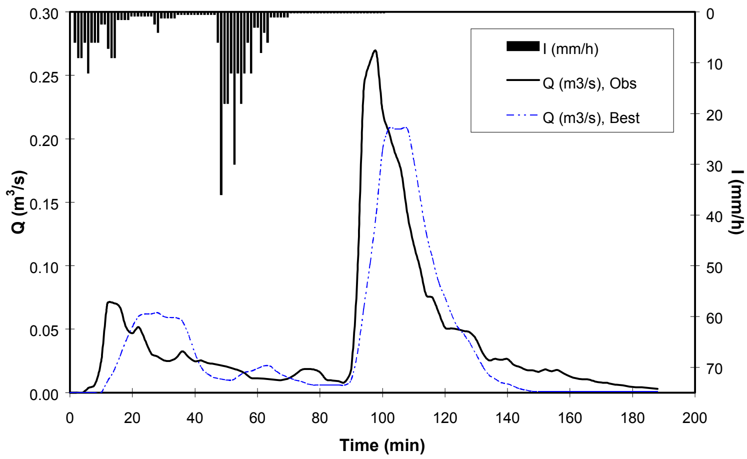


(b) Event V2

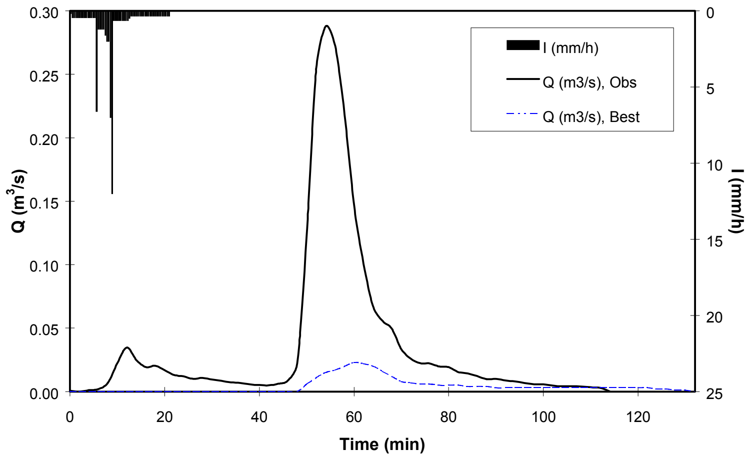


(c) Event V3

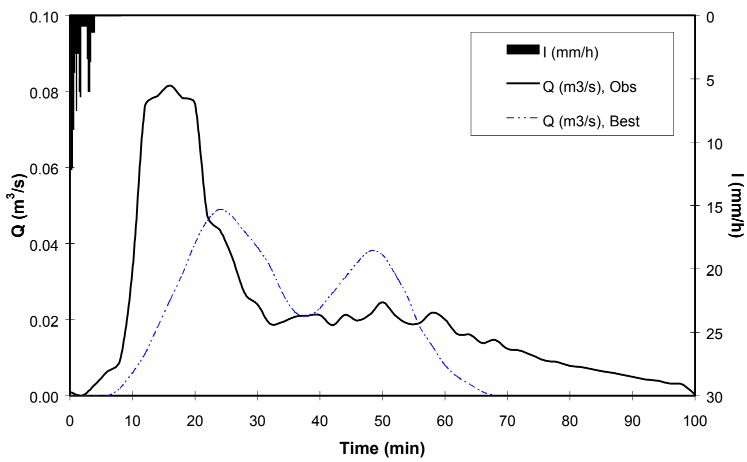
Figure F13: Verification Plots for Catchment H2



(a) Event V1

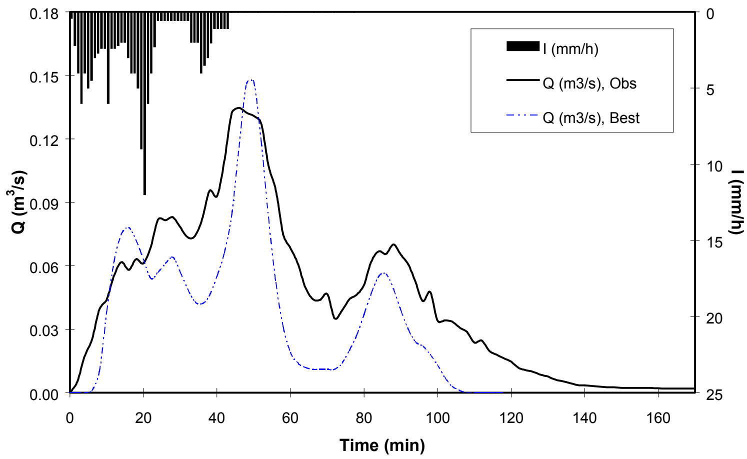


(b) Event V2

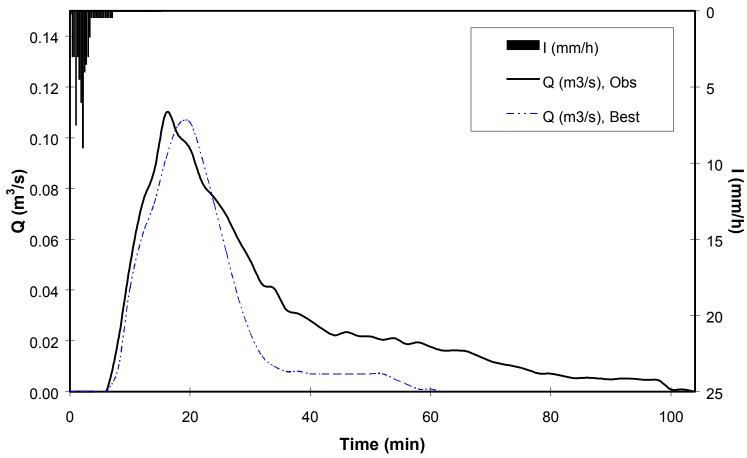


(c) Event V3

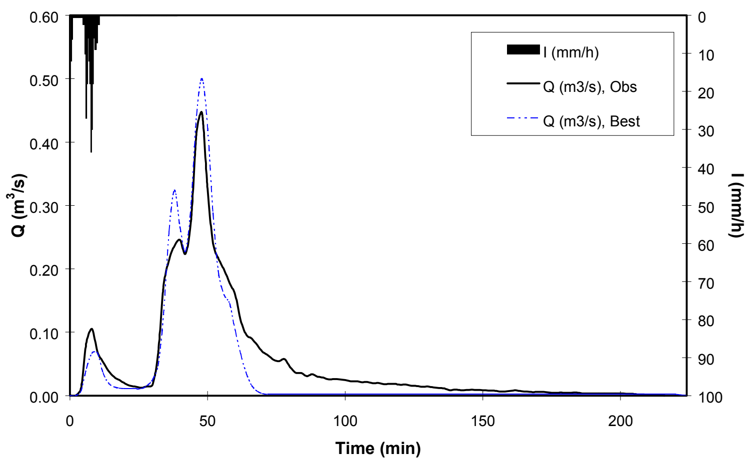
Figure F14: Verification Plots for Catchment H2A



(a) Event V1

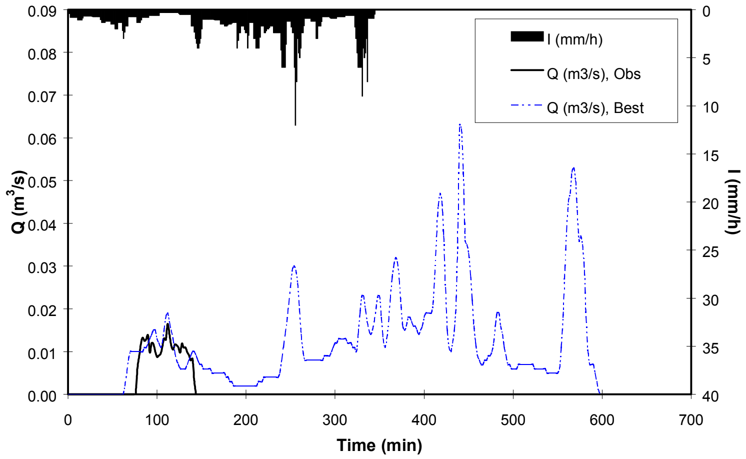


(b) Event V2

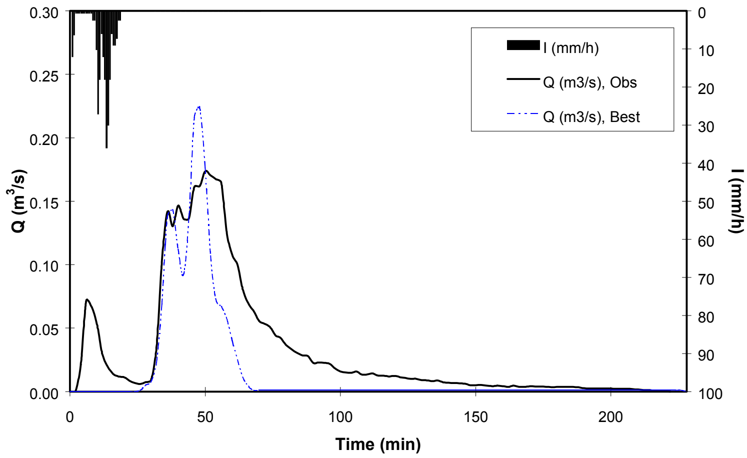


(c) Event V3

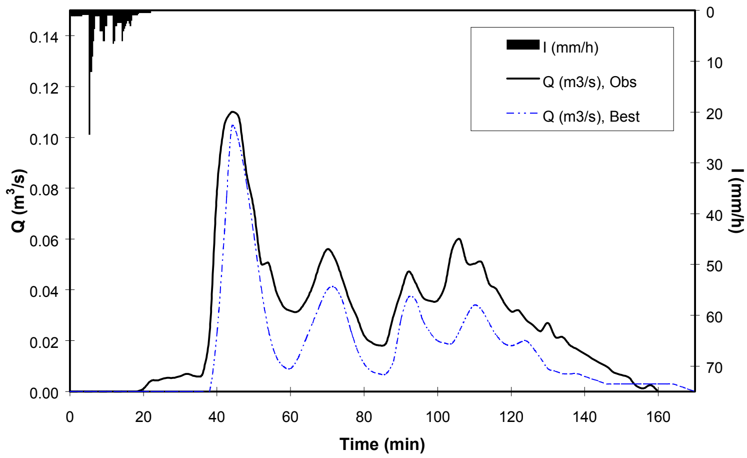
Figure F15: Verification Plots for Catchment K1



(a) Event V1

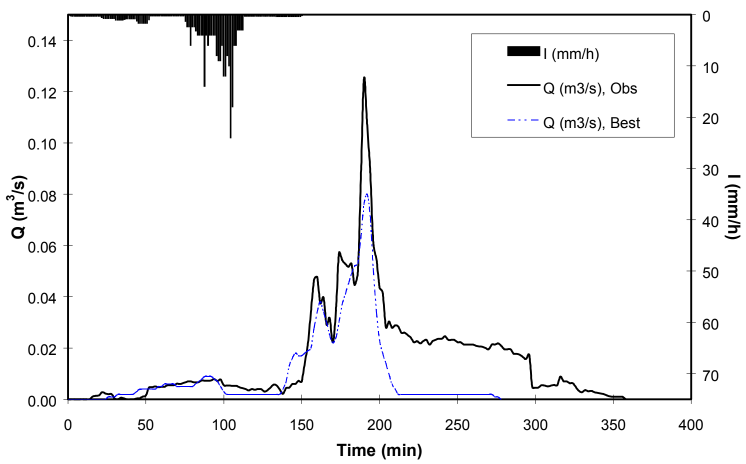


(b) Event V2

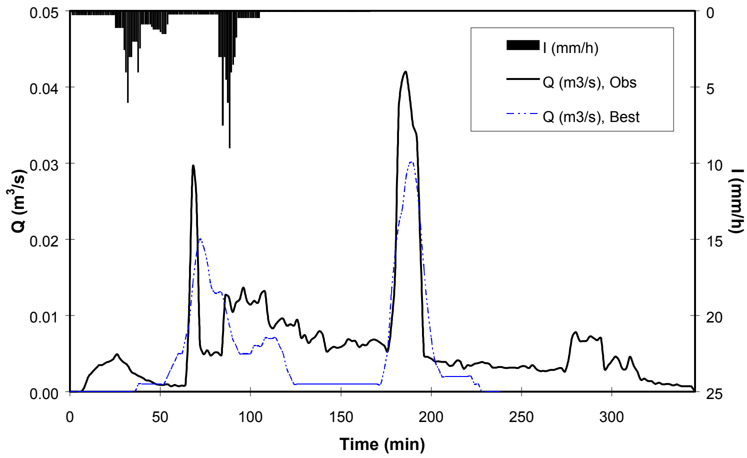


(c) Event V3

Figure F16: Verification Plots for Catchment K1A

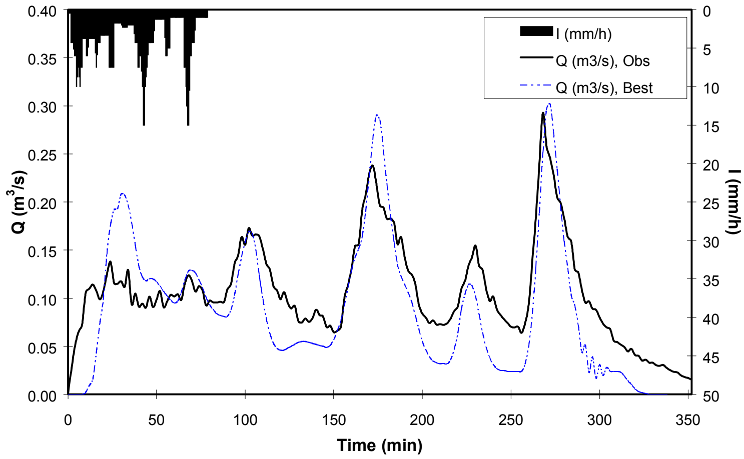


(a) Event V1

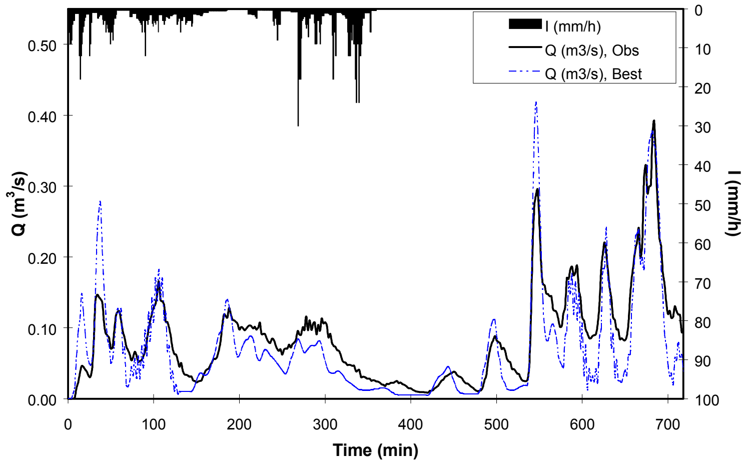


(b) Event V2

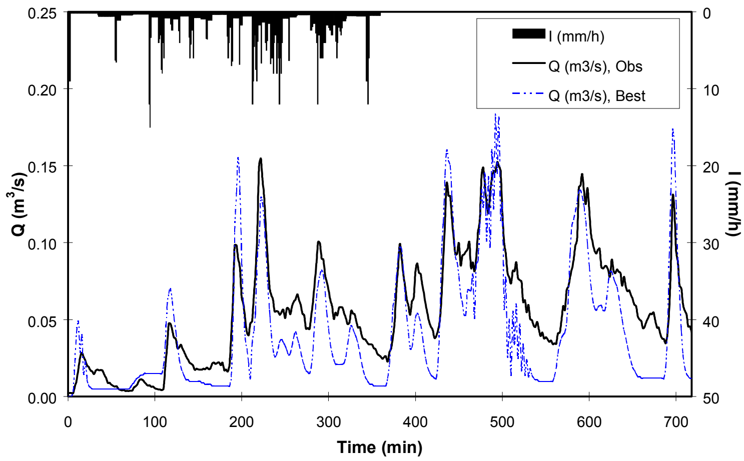
Figure F17: Verification Plots for Catchment K1B



(a) Event V1

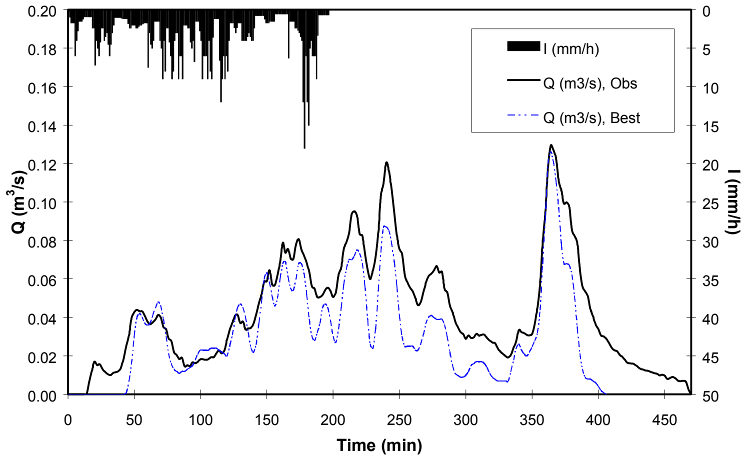


(b) Event V2

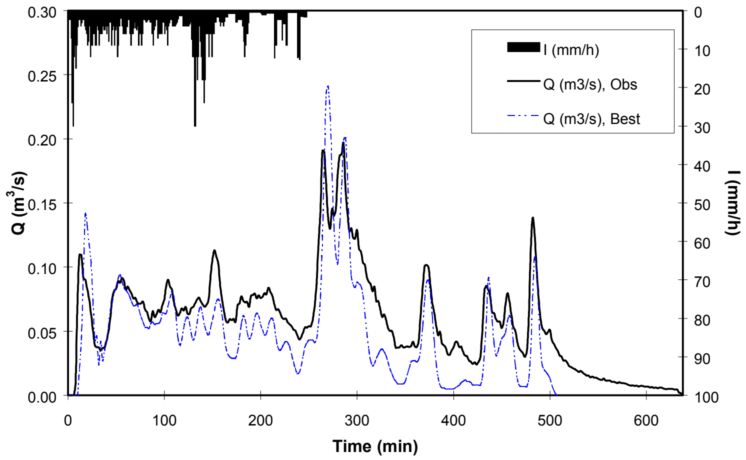


(c) Event V3

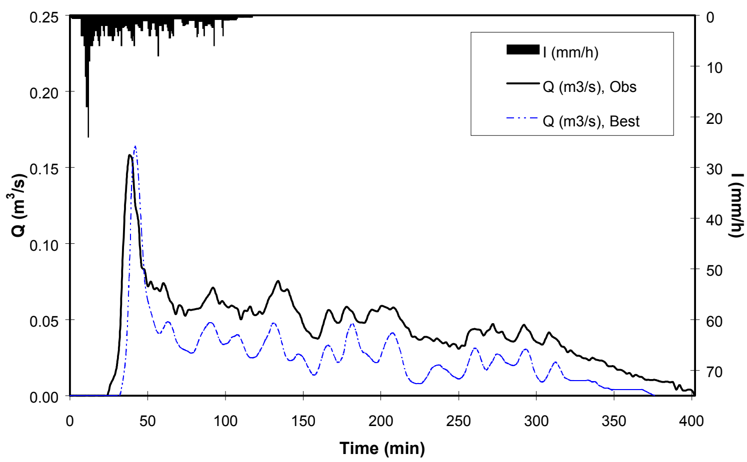
Figure F18: Verification Plots for Catchment K2



(a) Event V1

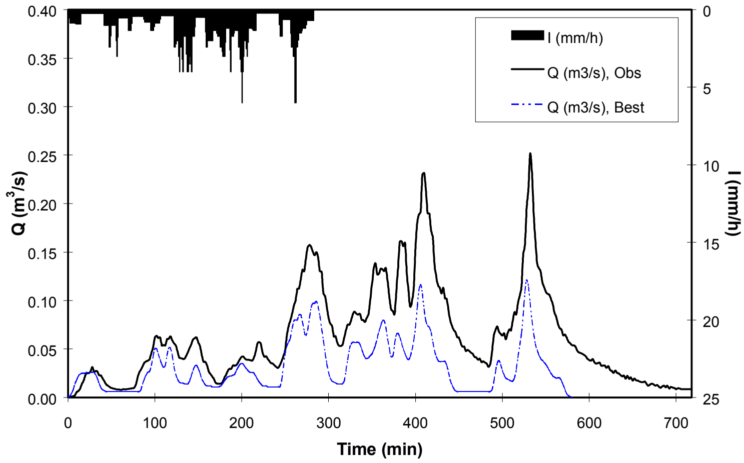


(b) Event V2

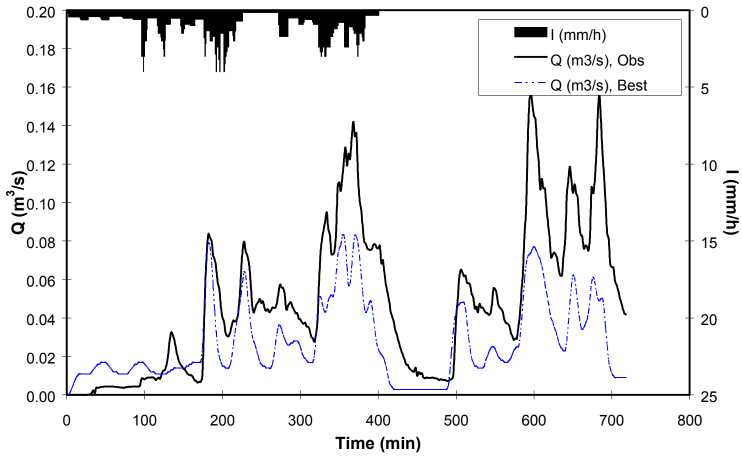


(c) Event V3

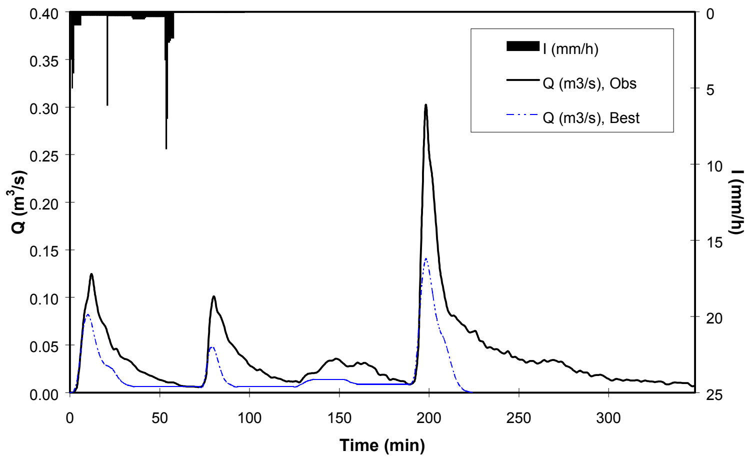
Figure F19: Verification Plots for Catchment K2A



(a) Event V1

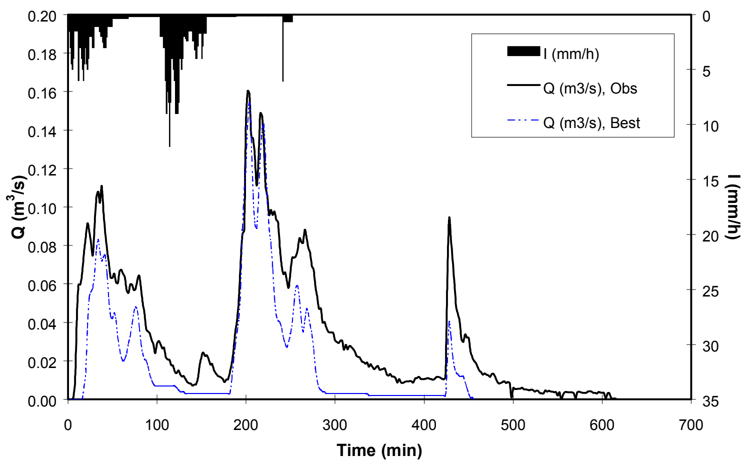


(b) Event V2

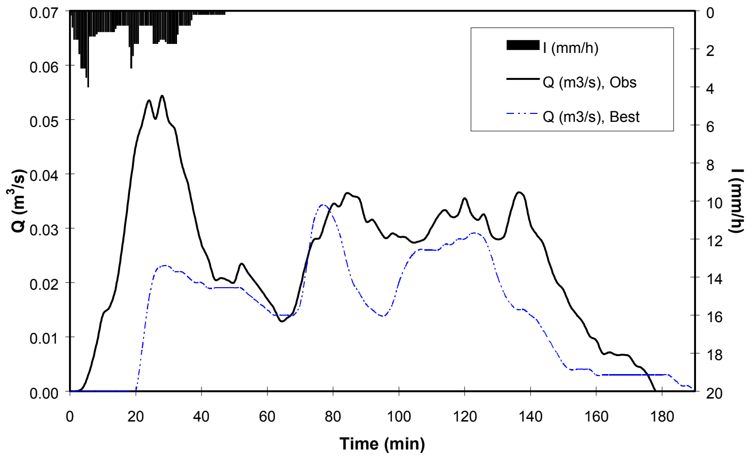


(c) Event V3

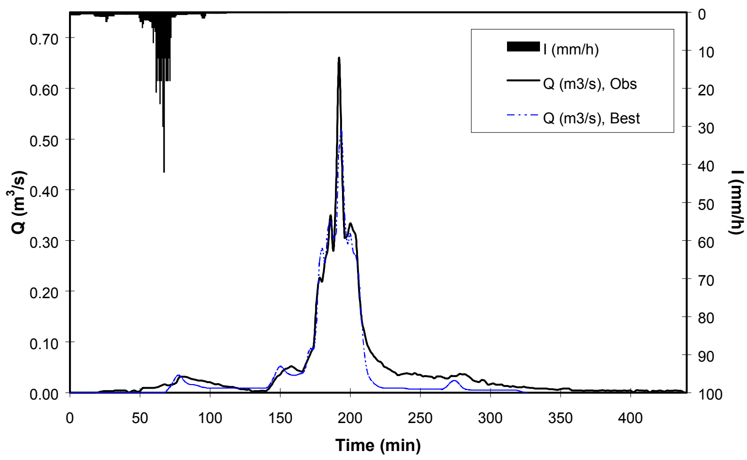
Figure F20: Verification Plots for Catchment K3



(a) Event V1

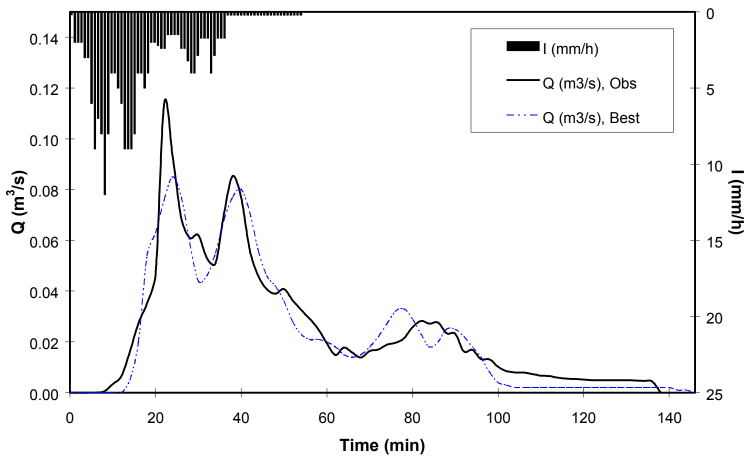


(b) Event V2

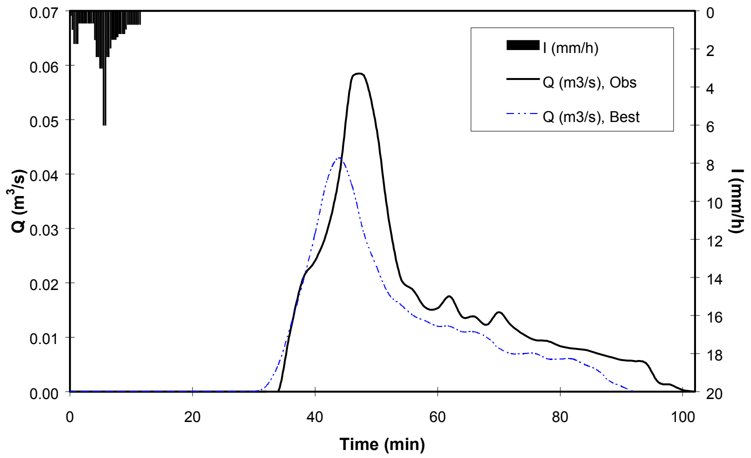


(c) Event V3

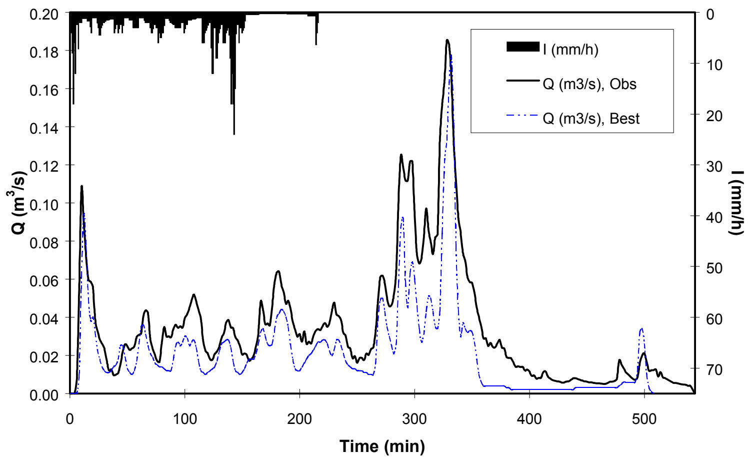
Figure F21: Verification Plots for Catchment K3A



(a) Event V1

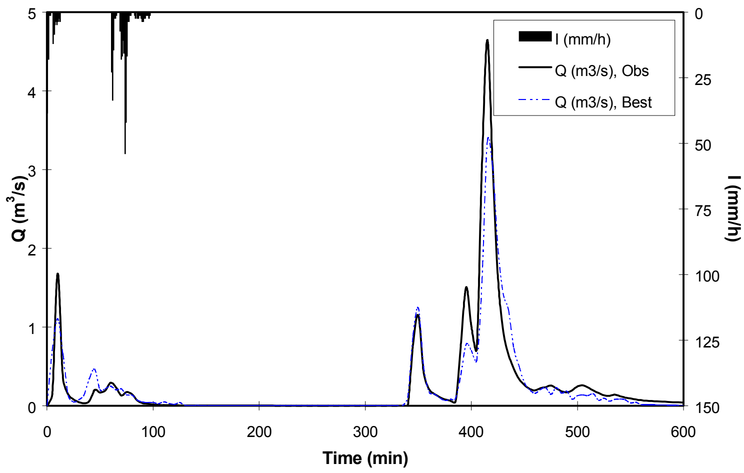


(b) Event V2

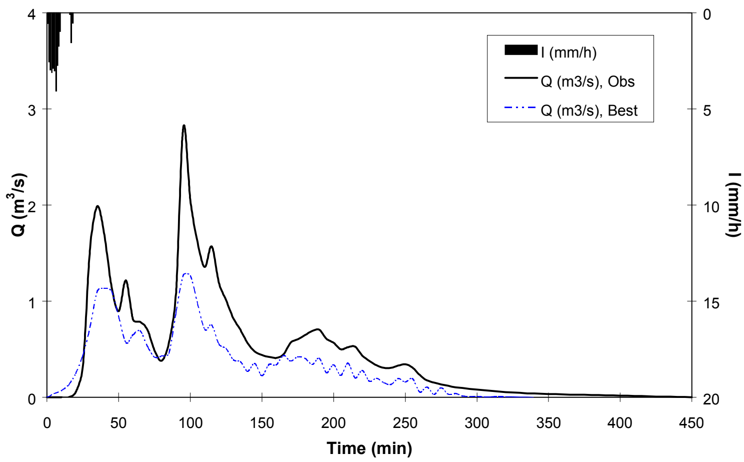


(c) Event V3

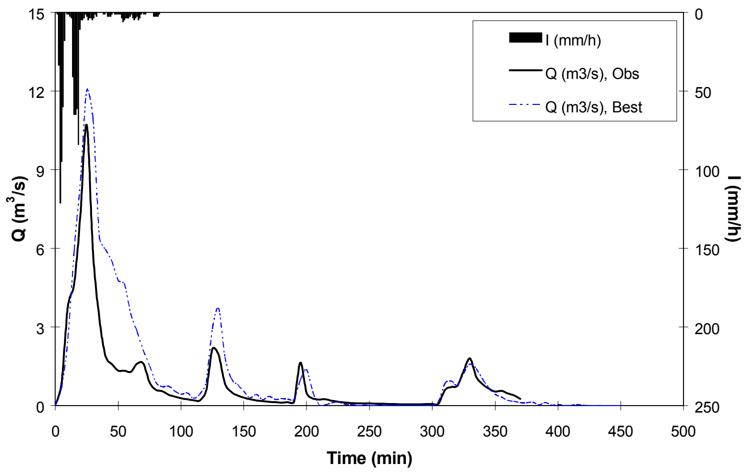
Figure F22: Verification Plots for Catchment K3B



(a) Event V1



(b) Event V2



(c) Event V3

Figure F23: Verification Plots for Giralang Catchment