Australian Rainfall & Runoff

Revision Projects

PROJECT 13

RATIONAL METHOD DEVELOPMENTS

Urban Rational Method Review

STAGE 3 REPORT
Draft for discussion

P13/S3/001

FEBRUARY 2014

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PROJECT 13 STAGE 3: URBAN RATIONAL METHOD REVIEW

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Project Project 13: Urban Rational Method Review	**AR&R Report Number** P13/S3/001
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FOREWORD

AR&R Revision Process

Since its first publication in 1958, Australian Rainfall and Runoff (ARR) has remained one of the most influential and widely used guidelines published by Engineers Australia (EA). The current edition, published in 1987, retained the same level of national and international acclaim as its predecessors.

With nationwide applicability, balancing the varied climates of Australia, the information and the approaches presented in Australian Rainfall and Runoff are essential for policy decisions and projects involving:

- infrastructure such as roads, rail, airports, bridges, dams, stormwater and sewer systems;
- town planning;
- mining;
- developing flood management plans for urban and rural communities;
- flood warnings and flood emergency management;
- operation of regulated river systems; and
- prediction of extreme flood levels.

However, many of the practices recommended in the 1987 edition of AR&R now are becoming outdated, and no longer represent the accepted views of professionals, both in terms of technique and approach to water management. This fact, coupled with greater understanding of climate and climatic influences makes the securing of current and complete rainfall and streamflow data and expansion of focus from flood events to the full spectrum of flows and rainfall events, crucial to maintaining an adequate knowledge of the processes that govern Australian rainfall and streamflow in the broadest sense, allowing better management, policy and planning decisions to be made.

One of the major responsibilities of the National Committee on Water Engineering of Engineers Australia is the periodic revision of ARR. A recent and significant development has been that the revision of ARR has been identified as a priority in the Council of Australian Governments endorsed National Adaptation Framework for Climate Change.

The update will be completed in three stages. Twenty one revision projects have been identified and will be undertaken with the aim of filling knowledge gaps. Of these 21 projects, ten projects commenced in Stage 1 and an additional 9 projects commenced in Stage 2. The remaining two projects will commence in Stage 3. The outcomes of the projects will assist the ARR Editorial Team with the compiling and writing of chapters in the revised ARR.

Steering and Technical Committees have been established to assist the ARR Editorial Team in guiding the projects to achieve desired outcomes. Funding for Stages 1 and 2 of the ARR revision projects has been provided by the Federal Department of Climate Change and Energy Efficiency. Funding for Stages 2 and 3 of Project 1 (Development of Intensity-Frequency-Duration information across Australia) has been provided by the Bureau of Meteorology.

Project 13: Rational Method Developments

Estimation of the peak flow on a small to medium sized rural catchment is probably one of the most common applications of flood estimation as well as having a significant economic impact. While the terms “small” and “medium” are difficult to define, upper limits of 25 km^2 and 500 km^2 can be used as guides. The Rational Method, which can be traced back to the mid-eighteenth century, is probably the most commonly used method for estimating the peak flow of a flood. Most urban drainage systems and culverts for rural roads, particularly those for small subdivisions, are designed using the Rational Method.

However, there are a number of problems associated with the use of the Rational Method. Most of these problems are associated with the estimation of parameter values such as the time of concentration and the runoff coefficient. As a result, the rational method may be easy to implement, but it is difficult to ensure that the predictions adequately represents processes occurring in the catchment.

Mark Babister
Chair Technical Committee for
ARR Research Projects

Associate Professor James Ball
AR&R Editor

AR&R REVISION PROJECTS

The 21 AR&R revision projects are listed below:

AR&R Project No.	Project Title
1	**Development of intensity-frequency-duration information across Australia**
2	Spatial patterns of rainfall
3	Temporal pattern of rainfall
4	**Continuous rainfall sequences at a point**
5	**Regional flood methods**
6	Loss models for catchment simulation
7	**Baseflow for catchment simulation**
8	Use of continuous simulation for design flow determination
9	**Urban drainage system hydraulics**
10	**Appropriate safety criteria for people**
11	**Blockage of hydraulic structures**
12	Selection of an approach
13	**Rational Method developments**
14	Large to extreme floods in urban areas
15	**Two-dimensional (2D) modelling in urban areas.**
16	Storm patterns for use in design events
17	Channel loss models
18	**Interaction of coastal processes and severe weather events**
19	Selection of climate change boundary conditions
20	Risk assessment and design life
21	IT Delivery and Communication Strategies

AR&R PROJECTS TECHNICAL COMMITTEE:

Chair: Mark Babister, WMAwater

Members: Associate Professor James Ball, Editor AR&R, UTS
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Associate Professor Ashish Sharma, UNSW
Dr Bryson Bates, CSIRO
Steve Finlay, Engineers Australia

Related Appointments:
ARR Project Engineer: Monique Retallick, WMAwater
Assisting TC on Technical Matters: Dr Michael Leonard, University of Adelaide

EXECUTIVE SUMMARY

The broad aim of Project 13 Stage 3 was to consider the merits of the continued usage of the Rational Method for estimating design flow peaks in urban catchments across Australia.

THE URBAN RATIONAL METHOD IN AUSTRALIA

The three editions of ARR (IEAust, 1958, 1977 and 1987) have each described the use of the urban Rational Formula method. The main differences between the three editions have been the need to assess partial area effects and the recommended procedures to estimate runoff coefficients and overland flow times of concentration.

The 1958 ARR provided recommendations for the use of the Rational Formula based primarily around the procedure introduced by Lloyd-Davies in England. Shortly after 1960 the original 1958 ARR received a minor updating that included an amendment to the Figure which plotted the runoff coefficients against rainfall intensity.

As described in the 1958 ARR:

> *"It is generally accepted that the values for the "coefficient of runoff are too high, primarily because they do not make adequate allowance for storage effects. Reduced values are now recommended. It is stressed, however, that these amended values are somewhat arbitrary, and based on intuitive judgement rather than adequately controlled experiments".*

The 1977 ARR retained the same time of concentration procedure and runoff coefficients as included in the 1958 ARR except for the change to metric units.

The 1987edition of ARR recommended changes to both the estimation of both time of concentration and runoff coefficient in urban drainage design.

In late 1988 and early 1989 a study was undertaken in Canberra by the then Willing & Partners to compare the methodologies for using the urban Rational Formula as recommended in both the 1977 and 1987 editions of Australian Rainfall and Runoff. Modelling was undertaken of both the Giralang (64 ha urban in a 94 ha catchment) and Mawson (382 - 400 ha depending on storm severity) gauged urban catchments and compared with flood frequency curves derived from gauged data. Both gauged catchments had in excess of twelve years of runoff records in 1988.

In Giralang the 1977 recommendations were found to give a good fit to the gauged flood frequency curve while the neither the runoff coefficient nor times of concentration for overland flow procedures calculated using the 1987 procedures individually or in concert provided acceptable results. Analysis of the Mawson catchment confirmed the findings from the Giralang catchment with the 1987 procedures giving peak design flows which were 40 - 60% lower than the flood frequency curve. In effect, the 5 Yr ARI peak flood discharge predicted using the AR&R, 1987 procedures was in fact equivalent to the gauged 1 Yr ARI peak flood discharge.

In view of the absence of data to support the urban runoff coefficient estimation procedure proposed in the 1987 ARR the comment of Munro (1956) may still apply:

> "*The literature abounds with tabulations of graphs of C for various conditions, but few are observed from reliable evidence .... Apparently, Horner and Flynt (1936) are the only ones to have carried out a really comprehensive set of measurements."*

REVIEW OF GAUGED URBAN CATCHMENTS IN AUSTRALIA

The evolution of gauged urban catchments in Australia since the 1970s is overviewed in Section 3.

In 1977 a total of 69 urban catchments across Australia were being gauged in 1977 with a further 5 catchments being proposed for gauging (Black and Aitken (1977)). The breakdown of catchments was:

ACT (Canberra)	7	NSW (Sydney)	3
QLD (Brisbane)	13	NT (Darwin)	3
VIC (Melbourne)	24	TAS (Hobart)	0
WA (Perth)	11	SA (Adelaide)	8

By 2009 only 24 urban gauged catchments were identified by Hicks et al. (2009) based on a number of criteria:

- Area less than 20 km^2 (smaller areas preferable, in the order of 1 km^2);
- Continuous records greater than 10 years in length;
- Fairly urbanised (greater than 50%);
- Acceptable gauge rating (max gauged flow: max recorded flow); and
- Stationary upstream urbanisation.

The breakdown of gauged catchments is:

ACT (Canberra)	5	NSW (Sydney)	2
QLD (Brisbane)	3	NT (Darwin)	2
VIC (Melbourne)	3	TAS (Hobart)	2
WA (Perth)	2	SA (Adelaide)	5

Based on the review described herein it is recommended that:

(i) Engineers Australia consult with major stakeholders to formulate a strategy to ensure the current collection of data is maintained and that data collection is expanded to encompass representative urban catchments across Australia to ensure that sufficient good quality data is available to allow the update of the Rational Formula method to reduce the potential error levels in the peak flows estimated using the procedure and/or to improve the guidance on rainfall-runoff model parameters for urban catchments;

(ii) Existing gauged urban catchments be reviewed to identify any features that may be distorting gauging records (eg. basins) and that any review should include preliminary simulation studies to quantify the effect of any features and the need or otherwise to develop a procedure to correct the gauged data;

(iii) Existing gauged catchments should be categorised based on regions, topography, geology and/or drainage systems;

(iv) Identify possible urban catchments that could be gauged to provide data for any regions, topography, geology and drainage systems not represented by existing gauged catchments;

(v) Undertake preliminary modelling of any new candidate gauged catchments;

(vi) Filter future potential gauged catchments to prioritize installations;

(vii) As a matter of priority seek to increase the density of rainfall gauges across existing gauged catchments to further qualify areal effects within smaller urban catchments.

POSSIBLE USES OF CURRENT AVAILABLE GAUGED URBAN DATA

One of the objectives of this Discussion Paper was to identify potential uses of the gauged urban streamflow data that is currently available. It was concluded that the current available gauged urban streamflow data could be used to undertake Part I, Part II, Part III or Part IV studies as follows.

Part I Study

The Part I study approach is to calibrate relations for the estimation of time of concentration and runoff coefficients for the urban Rational Method against flood quantiles derived from flood frequency analysis (FFA) of flows recorded in one or more gauged urban catchments.

A Part I study was undertaken in the ACT in 1989 (refer Appendix E).

A key conclusion of the Part I study was that the runoff coefficient and time of concentration relationships are paired ie. they both need to be derived concurrently using gauged data rather than derived relationships independently.

The preliminary application of the Part I study approach to gauged urban catchments in Canberra, Sydney, Melbourne and Darwin is described in **Appendix C**.

Part II Study

The Part II study approach is to calibrate parameter values for hydrological models by matching predicted peak flows against flood quantiles derived from flood frequency analysis (FFA) of gauged flows recorded in one or more gauged urban catchment.

A Part II study was undertaken in the ACT in 1993 (refer Appendix F).

In the case of the 1993 Part II study in the ACT it was found that in order to match the flow quantiles obtained from FFA that the initial pervious rainfall loss needed to increase with increasing ARI ie. the 2 yr ARI peak flow was best fitted by a 5.0 mm initial pervious area rainfall loss while the 100 yr ARI peak flow was best fitted by a 15.0 mm initial pervious area rainfall loss. This was counter-intuitive and this issue was overcome by adopting a infiltration/water balance procedure based on the Australian Representative Basin Program (ARBM). A further potential problem with the Part II study in the ACT was the recommended initial (high) values for moisture stores.

These issues are explored in the analysis of the Giralang catchment (in Canberra) and Hewitt catchment (in Sydney) in Appendix D.

Subsequent to the 1989 study Goyen (2000) incorporated an alternate sub-catchment analysis procedure into the **xprafts** program. As presented in Appendix D, an excellent level of agreement was achieved between gauged and predicted flows at the micro catchment and urban catchment scales in Giralang and at the urban catchment scale in Hewitt by this model.

Part III Study

A possible approach to increase the number of test catchments would be to undertake rainfall and flow gauging in new catchments for a period of 3-5 years only and to apply a Part III study approach to create benchmark flood frequency curves for these new catchments as the basis for the testing of runoff coefficient and time of concentration relations or identification of parameter values for hydrological models ie. further Part I and/or Part II studies.

The Part III approach involves the calibration of a hydrological model (of the form assembled by Goyen (2000) or a comparable model) against a range of storm events for which there is gauged rainfall and runoff. A sufficient number of storm events would then be extracted from long term pluviograph records and the calibrated model would be run to estimate peak flows. A FFA of the peak flows could then be undertaken to estimate the flow quantiles.

This approach has been previously proposed by Aitken (1975) to utilize the available long term rainfall pluviograph record nearest a catchment together with short term calibration records to simulate all the major rainfall events in the rainfall record.

If multiple long term rainfall stations existed near or within the catchment the problems of rainfall spatial variance could also be eliminated or at least minimized.

Part IV Study

The Part IV study approach is similar to the Part III study approach. However, instead of calibrating a hydrological model (of the form assembled by Goyen (2000)) against a range of storm events for which there is gauged rainfall and runoff, the hydrological model would be calibrated using full continuous simulation for the period of gauging. This calibrated model would then be used to run the long term pluviograph record(s) and predicted peak flows would then be extracted to allow a FFA to be undertaken to estimate the flow quantiles.

Any continuous simulation would most likely rely on a scheme where the time step lengthens during dry spells and reduces to a time step of say 1 minute during storm events.

APPLICATION OF THE URBAN RATIONAL METHOD

Should the Urban Rational Method continue to be included in ARR?

Since the publication of 1987 ARR a number of water authorities as well as Councils have also published their own recommendations on how the Rational Method should be applied to urban catchments in their jurisdiction. Typically these guidelines recommend procedures for estimating runoff coefficient and time of concentration which differ from those recommended in the 1987 ARR. It is unclear if these guidelines are based on a comprehensive study of one or more gauged urban catchments or whether *values are somewhat arbitrary and based on intuitive judgement rather than adequately controlled experiments* (as concluded in the 1958 ARR).

Notwithstanding that the 1989 Part I study in the ACT concluded that the results from the study lent further support to the continued use of the Rational Formula for drainage design in small to medium sized urban catchments, this was on the basis that further studies be undertaken to further examine possible modifications to the recommended 1987 ARR procedures to improve the estimation of surface flow times of concentration and corresponding runoff coefficients. In particular, it recommended that further studies should aim to determine appropriate surface roughness values for use in the kinematic wave formulation for overland flow in Australia.

These further studies have not been undertaken in the 24 years since.

Notwithstanding the preliminary assessment of gauged urban catchments in Sydney, Melbourne and Darwin disclosed that in general the 1977 ARR Rational Method gives peak flows which better match the peak flows calculated by flood frequency analysis (FFA) than the peak flows estimated using 1987 ARR Rational Method (refer Appendix C) without carrying out Part I studies on a significant number of additional gauged urban catchments it is the view of the authors that continued use of the Rational Method for urban drainage analysis and design can no longer be justified.

Should the Urban Rational Method be used to Calibrate Hydrological Models?

With the advent of PCs in the 1980s and the improvements in computer speed and capabilities since that time as well as the continued development of urban rainfall runoff catchment simulation models, computer based modelling has almost totally supplanted the role of Rational Method calculations in urban drainage design. Notwithstanding these advances some authorities still require urban hydrological models to be “calibrated” to match peak flows estimated using the 1987 ARR urban Rational Method.

It is the view of the authors that the urban Rational Method should not be used to calibrate urban hydrological models unless it can be demonstrated that:

(i) A detailed Part I study has been undertaken on one or more gauged urban catchments in the relevant city or town which has calibrated and validated relations for the calculation of runoff coefficients and times of concentration; and
(ii) The urban catchment which is being modelled is subject to a similar hydrological regime and has a level of imperviousness comparable to the gauged urban catchment(s) analysed in the Part I study; and
(iii) WSUD measures are not present in the urban catchment which is being modelled.

CONSISTENCY WITH REGIONAL RURAL FLOOD METHOD

One of the objectives of this Discussion Paper was to investigate if it is practical to develop a method to adjust the procedures recommended in Project 5 Regional Flood Methods to estimate peak flows in small to medium sized urban catchments. .

An initial benchmark annual and partial series analysis of gauged flows has been undertaken for nine urban catchments and one paired rural catchment as described in **Appendix B**. At the same time the peak flows for each catchment under pre-development (rural) conditions for 2, 5, 10, 20, 50 and 100 yr ARIs were estimated for most of these catchments using the procedures recommended under Project 5.

It was concluded from a comparison of flow quantiles for selected gauged urban catchments derived from FFA and estimated peak flows for the selected catchments under rural conditions (estimated using the Project 5 procedures) that:

- The 2yr ARI peak flows for all urban catchments (derived from FFA) are higher than the estimated 2 yr ARI peak flows under pre-development (rural) conditions (derived from Project 5);
- The ratio of urban to rural peak flows decreases as ARI increases;
- In the case of the Canberra urban catchments the 100 yr ARI peak flow (derived from FFA) are higher than the estimated 100 yr ARI peak flow under pre-development (rural) conditions (derived from Project 5);
- In the case of the Gungahlin paired rural catchment the Project 5 quantiles were consistently and significantly higher than the corresponding FFA quantiles;
- In the case of the Sydney, Melbourne, and Darwin urban catchments the 100 yr ARI peak flow (derived from FFA) are lower than the estimated 100 yr ARI peak flow under pre-development (rural) conditions (derived from Project 5).

It was further concluded that based on the scatter of the calculated ratios of urban to rural peak flows and the overestimation of rural peak flows in comparison with urban peak flows derived from FFA in major events in a number of catchments that it is not practical to develop a simple method to adjust the peak flows from rural catchments to give reliable estimates of peak flows in urban catchments at this time.

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LIST OF ABBREVIATIONS

AEP	Annual Exceedance Probability
ARI	Average recurrence interval
ARR	Australian Rainfall and Runoff
FFA	Flood Frequency Analysis
QRT	Quantile Regression Technique
RFFA	Regional flood frequency analysis

1 INTRODUCTION

Estimation of the peak flow on a small to medium sized rural catchment is probably one of the most common applications of flood estimation as well as having a significant economic impact. While the terms “small” and “medium” are difficult to define, upper limits of 25 km^2 and 500 km^2 can be used as guides. The Rational Method, which can be traced back to the mid-eighteenth century, is probably the most commonly used method for estimating the peak flow of a flood. Most urban drainage systems and culverts for rural roads, particularly those for small subdivisions, are designed using the Rational Method.

The Rational Formula has been included in each of the Australian Rainfall and Runoff documents since the release of the first edition in 1958. The method has been recommended for smaller urban drainage design projects with an emphasis of providing a simple method that can be carried out generally using hand calculations.

There are, however, a number of problems associated with the use of the Rational Method. Most of these problems are associated with the estimation of parameter values such as the time of concentration and the runoff coefficient. As a result, the Rational Method may be easy to implement, but it is difficult to ensure that the predictions adequately represents processes occurring in the catchment.

With the advent of PCs in the 1980s and the improvements in computer speed and capabilities since that time as well as the continued development of urban rainfall runoff catchment simulation models, computer based modelling has almost totally supplanted the role of hand calculations in urban drainage design. Notwithstanding these advances some authorities still require urban hydrological models to be “calibrated” to match peak flows estimated using the Rational Method.

In late 1988 and early 1989 a study was undertaken in Canberra by the then Willing & Partners to compare the methodologies for using the urban Rational Formula as recommended in both the 1977 and 1987 editions of Australian Rainfall and Runoff.

The two documents differed significantly in their specific recommendations for estimating both the subarea time of concentration for overland flow and the appropriate subcatchment runoff coefficient.

To test the acceptability of either the 1977 or 1987 recommendations, modelling was undertaken of both the Giralang (64 ha urban in a 94 ha catchment) and Mawson (382 - 400 ha depending on storm severity) gauged urban catchments and compared with flood frequency curves derived from gauged data. Both gauged catchments had in excess of twelve years of runoff records in 1988.

In Giralang the 1977 recommendations were found to give a good fit to the gauged flood frequency curve while the neither the runoff coefficient nor times of concentration for overland flow procedures calculated using the 1987 procedures individually or in concert provided acceptable results. In the Giralang analysis in particular, it was shown that it was essential to estimate peak flood flows from partial areas. The peak flood flow at the catchment outlet was underestimated by 33% when only the total area was considered.

To verify these findings, similar simulations were undertaken of the second gauged urban catchment at Mawson. This catchment confirmed the findings from the Giralang catchment with the 1987 procedures giving peak design flows which were 40 - 60% lower than the flood frequency curve. In effect, the 5 Yr ARI peak flood discharge predicted using the AR&R, 1987 procedures was in fact equivalent to the gauged 1 Yr ARI peak flood discharge.

A number of lag times were also determined from recorded hydrographs from the Giralang and Mawson gauging stations. These lag times lend further support to the acceptability of the AR&R, 1977 procedure for the estimation of surface flow times of concentration.

Notwithstanding that the 1989 review concluded that the results from the study lent further support to the continued use of the Rational Formula for drainage design in small to medium sized urban catchments this was on the basis that further studies be undertaken to further examine possible modifications to the recommended AR&R, 1987 procedures to improve the estimation of surface flow times of concentration and corresponding runoff coefficients.

In particular, further studies should aim to determine appropriate surface roughness values for use in the kinematic wave formulation for overland flow in Australia.

These further studies have not been undertaken in the 24 years since.

In 1958 Professor Crawford Munro the lead author of the first edition of Australian Rainfall and Runoff described in particular the runoff coefficients recommended in ARR as “intuition” based and nothing has changed to date.

The proposition put forward by Tony Aitken in 1975 in AWRC Technical Paper 10 that Rational Method parameters should be based on calibrated catchment simulation using long term rainfall pluviograph data has now become practical, It is paradoxical that at a point in time when we can now extract the maximum out of existing data collected over the last 40 years to recommend more factual parameters for the Rational Method it is also the time to consider whether there is any merit in continuing to clasp to the Rational Method for urban drainage system analysis or design.

1.1 AIM

The broad aim of Project 13 Stage 3 was to consider the merits of the continued usage of the Rational Method for estimating design flow peaks in urban catchments across Australia.

This was considered in five steps as follows:

1. Assess the current availability of long term urban streamflow data to support the calibration and verification of the urban Rational Method and the merit of continued collection of urban streamflow data in the long term;

2. Identify appropriate uses of the gauged urban streamflow data that is currently available;

3. Identify possible limitations on the application of the urban Rational Method eg. catchment size, event frequency, retarding basin analysis, etc and/or the need to include a factor of safety when using the urban Rational Method;

4. Review the current practice of some authorities that require other hydrological methods to be "calibrated" to the peak flows estimated using the using the urban Rational Method; and

5. Investigate if it is practical to develop a method adjust the procedures recommended in Project 5 Regional Flood Methods to estimate peak flows in small to medium sized urban catchments.

Related ARR Projects include:

- ARR Project No. 13 Stage 1
- ARR Project No. 5 Regional Flood Methods

1.2 REPORT STRUCTURE

This report is structured as follows:

- **Section 2** Urban Rational Method in Australia
- **Section 3** Review of Gauged Urban Catchments
- **Section 4** Possible Uses of Current Available Gauged Urban Data
- **Section 5** Application of The Urban Rational Method
- **Section 6** Consistency with Rural Regional Flood Method

Further information is also provided in the Appendices.

2 URBAN RATIONAL METHOD IN AUSTRALIA

The three editions of ARR (IEAust, 1958, 1977 and 1987) have each described the use of the urban Rational Formula method. The main differences between the three editions have been the need to assess partial area effects and the recommended procedures to estimate runoff coefficients and overland flow times of concentration.

2.1 1958 EDITION

The first edition of ARR released in 1958 provided only two basic methods to estimate design flow magnitudes within urban catchments in Australia. These were the Rational Method as prescribed by Lloyd Davies for smaller urban drainage system design and the Unit Hydrograph procedure for larger catchments for bridges and major drains. Both methods were considered to be deterministic models of the rainfall-runoff process.

ARR, 1958 provided recommendations for the use of the Rational Formula based primarily around the procedure introduced by Lloyd-Davies in England.

The Rational Formula for the peak discharge at the outlet of a drainage area was described as (IEAust, 1958):

$$q = A\ C\ p \qquad (1)$$

where q = peak discharge (cusecs)
A = drainage area (acres)
C = a non-dimensional coefficient of runoff
p = temporal mean point-rainfall intensity (inches per hour) for a duration equal to the time of concentration and for a specified storm recurrence interval."

A nomograph and formula for the time of concentration was provided on Figure 2-3 in the 1958 ARR. This nomograph is reproduced in **Figure 2.1**. Note the following attribution on the nomograph:

Data attributed to US Dept.of Agriculture. 1942,
Nomograph published in "Municipal Utilities", Sept 1951
Formula and values of "n" added by J.A. Friend 19th Nov 1954.

For permeable areas the coefficient of runoff was plotted in Figure 2-2 in the 1958 ARR. This Figure is reproduced in **Figure 2.2**.

Shortly after 1960 the original 1958 ARR received a minor updating that included an amendment to Figure 2-2. The amended Figure 2-2 is reproduced in **Figure 2.3**. This figure changed from the previous ASCE Hydrology Handbook figure to one based on a figure published by Ordon in 1954.

This amendment was described as follows: "*Pending the collection of further data FIG 2-2 (amended) is submitted as an interim improvement. This is the figure utilised by the metropolitan Water, Sewerage and Drainage Board, Sydney N.S.W. and even this may give results somewhat on the high side"* ….

As described in the 1958 ARR:

"It is generally accepted that the values for the "coefficient of runoff are too high, primarily because they do not make adequate allowance for storage effects. Reduced values are now recommended. It is stressed, however, that these amended values are somewhat arbitrary, and based on intuitive judgement rather than adequately controlled experiments".

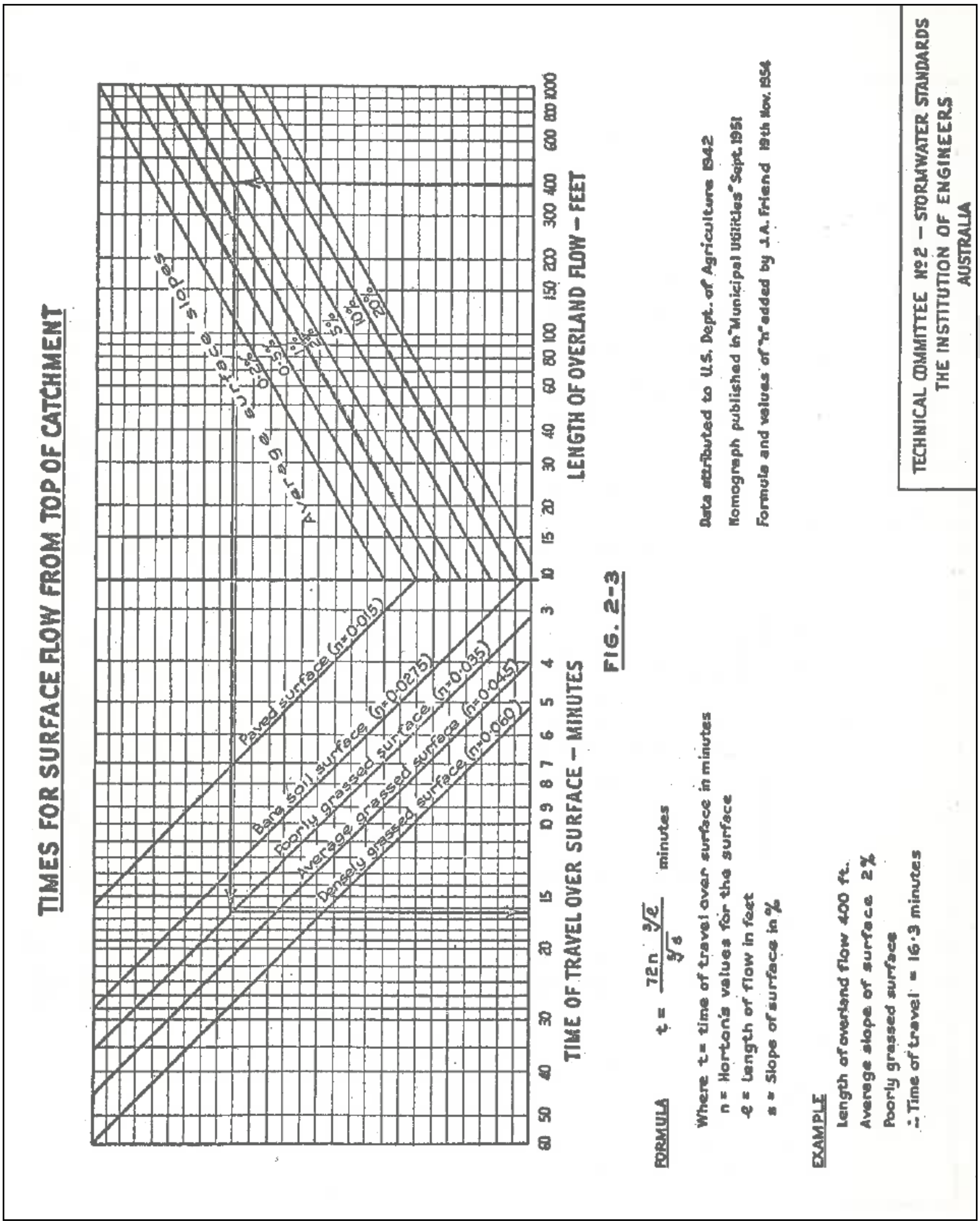


Figure 2.1 Times for Surface Flow from Top of Catchment (after Figure 2-3, 1958 ARR)

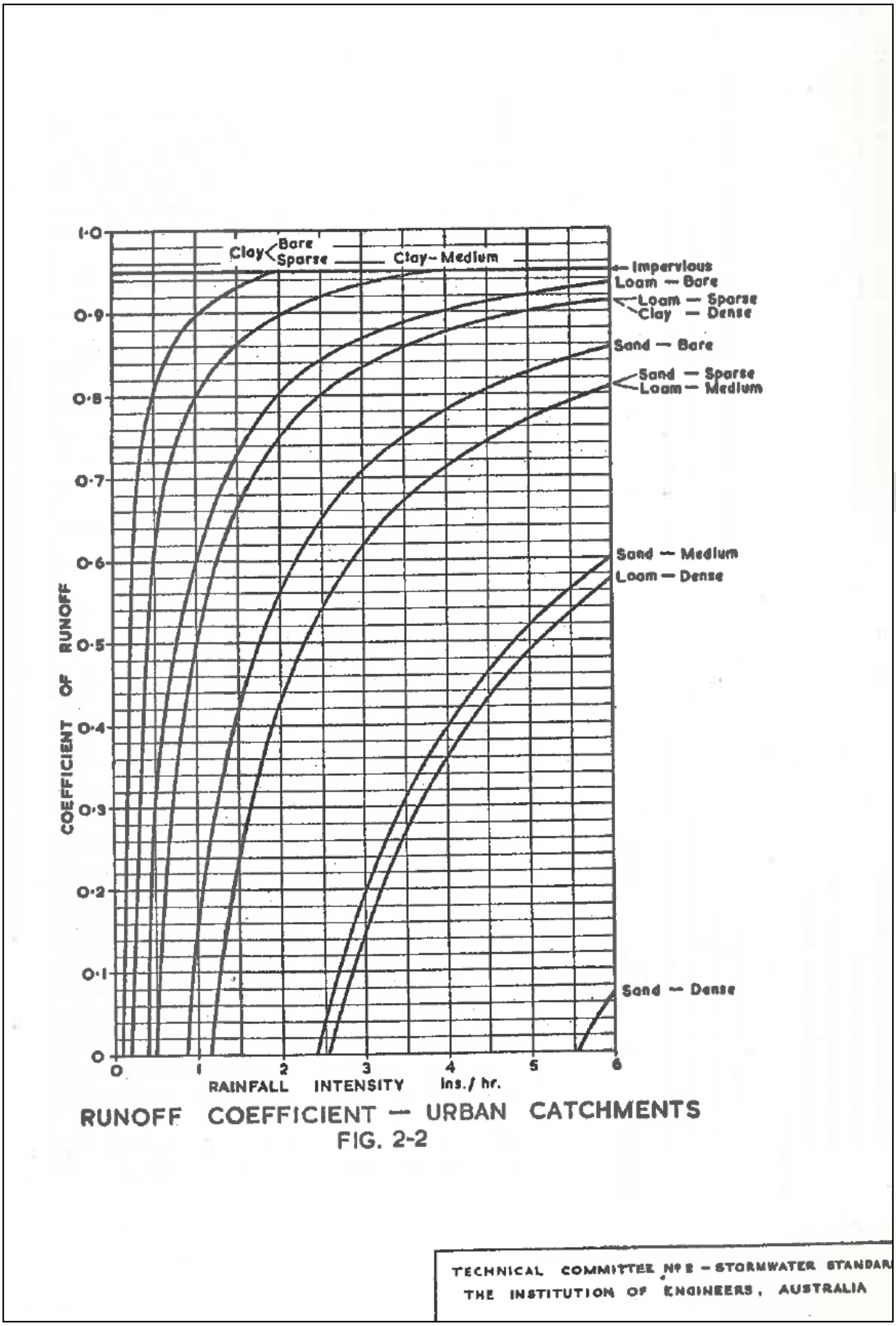
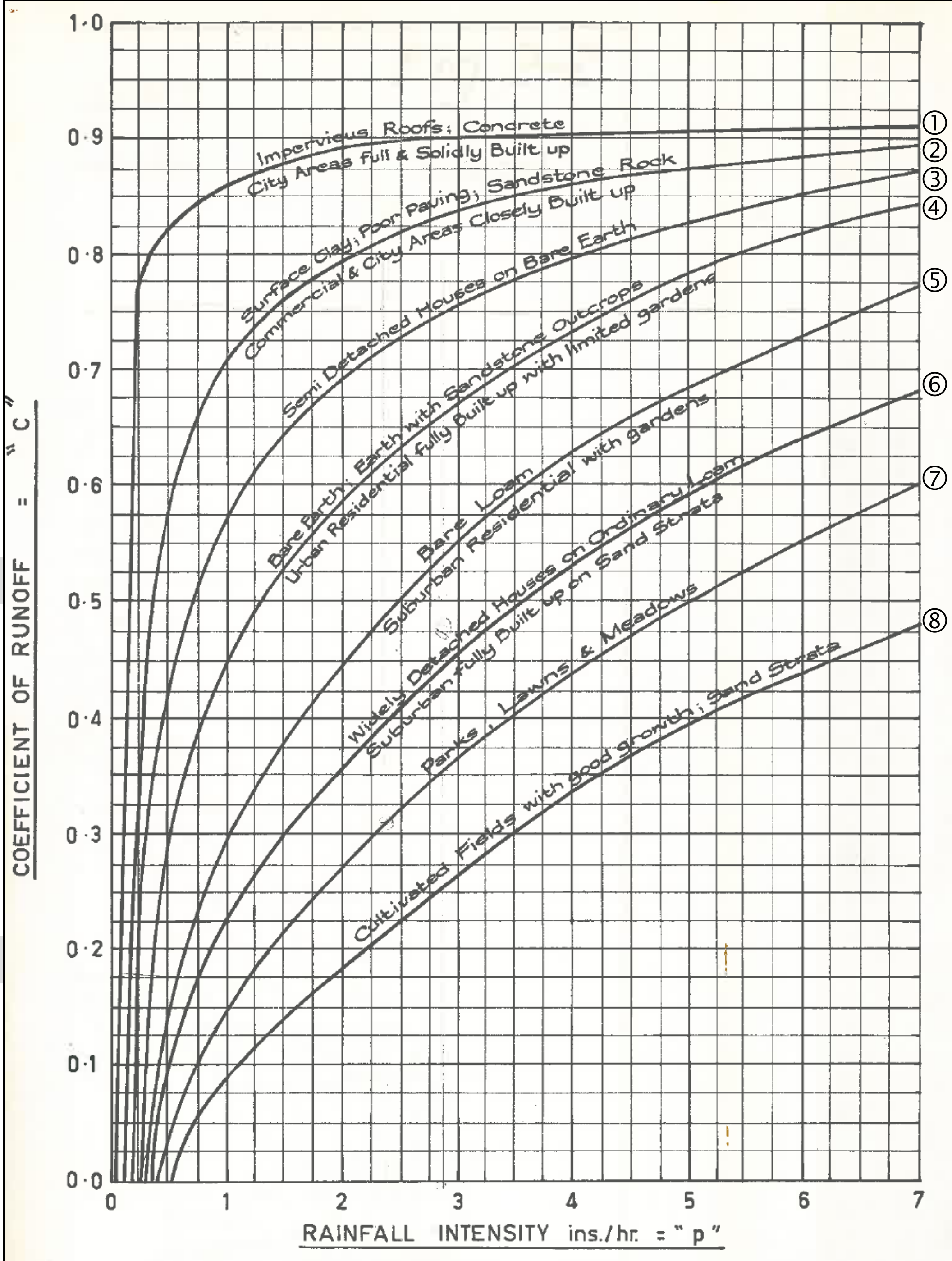


Figure 2.2 Runoff Coefficients – Urban Catchments (after Figure 2-2, 1958 ARR)



RUNOFF COEFFICIENT —— URBAN CATCHMENTS

— FIG. 2-2 (Amended) —

(a) Adapted from "A Modified Rational Formula for Stormwater Runoff" Ordon C.J.. Water and Sewerage Works. June. 1954

(b) Where the average slope is flatter than 1 in 100 deduct 0·05 from value of "C"

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Figure 2.3 Runoff Coefficients – Urban Catchments Amended (after Figure 2-2 (Amended), 1958 ARR)

2.2 1977 EDITION

The 1977 edition of ARR retained the same time of concentration procedure and runoff coefficients as included in the 1958 ARR except for the change to metric units.

The AR&R, 1958 nomograph also presented a formula for the calculation of the overland flow time which was attributed to Friend, 1954. This equation is as follows (S.I. units):

$$t_o = 107 \frac{n\ L^{0.333}}{S^{0.2}} \tag{2}$$

where	to	=	overland flow travel time (minutes)
	L	=	flow path length (m)
	n	=	Horton's roughness value for the surface
	S	=	slope of surface (%)

2.3 1987 ARR

The 1987edition of ARR recommended changes to both the estimation of both time of concentration and runoff coefficient in urban drainage design.

1987 ARR departed from the empirical relationship given in Equation 2. Instead, it recommended the use of the "kinematic wave" equation for overland flow time previously described by Ragan & Duru (1972). This equation is as follows:

$$t_o = \frac{6.94\ (L\ n^*)^{0.6}}{I^{0.4}\ S^{0.3}} \tag{3}$$

where	t_o	=	overland flow travel time (minutes)
	L	=	flow path length (m)
	n*	=	surface roughness
	I	=	rainfall intensity (mm/h)
	S	=	slope (m/m)

While the later equation for estimating overland flow times is based on a rigorous solution of the shallow overland flow equations, the appropriate values particularly for the surface roughness, n*, are not well defined. The reported roughness values for pervious surfaces range between 0.05 and 0.70.

Reported values for Horton's roughness values in Equation 3 are similar to Manning 'n' roughness values and range between 0.015 for paved surfaces up to 0.06 for densely grassed surfaces.

The estimation of overland flow times can have a significant effect on the predicted peak flow due to its influence on the value of rainfall intensity input into the Rational Formula.

The 1987 ARR varies from the 1958 and 1977 editions in its presentation of runoff coefficients for design purposes. This edition presents a:

> "*composite relationship reflecting experience of drainage authorities and evidence from the few gauged urban catchments with suitable lengths of record ...*"

It is stated that:

> *"it should be used in preference to the runoff coefficient relationships given in previous editions..."*

The 10 Year ARI runoff coefficients recommended in the 1987 AR&R are presented in **Figure 2.4**. Also shown for comparison are the data used to define the upper and lower bounds of the interpolation zone. The location of the gauged catchments, their size and representative rainfall intensity are given in **Table 2.1**.

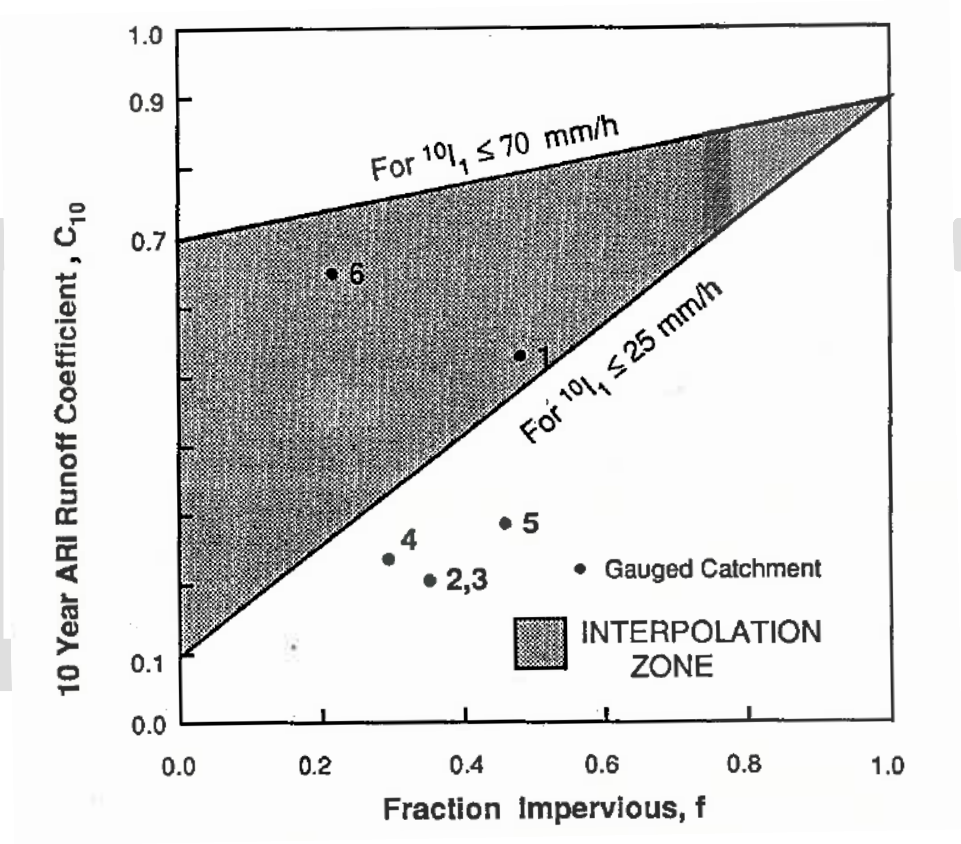

Figure 2.4 10 year ARI Runoff Coefficients (after 1987 ARR)

Table 2.1 Gauged Urban Catchment Descriptions (after 1987 ARR)

Gauged Urban Catchment No.	Location	Catchment Area (ha)	$^{10}I_1$ (mm/h)
1	Powells Creek, Strathfield, Sydney	231	48.9
2	Box Hill Main Drain, Box Hill, Melbourne	113	28.0
3	Vine Street Main Drain, Braybrook, Melbourne	70	29.0
4	Ashmore Ave Main Drain, Mordialloc, Melbourne	53	26.5
5	Gardenia Road Main Drain, Doncaster, Melbourne	80	28.1
6	Yarralumla Creek, Mawson, Canberra	382-400	32.2

The graphical relationship is further supplemented by the following numerical relationships:

$$C_{10} = 0.9\, f + C^1_{10}\,(1 - f) \qquad (4)$$

and

$$C^1_{10} = 0.1 + 0.0133\,({}^{10}I_1 - 25) \qquad (5)$$

where
C_{10} = 10 year ARI runoff coefficient
C^1_{10} = pervious area 10 Year ARI runoff coefficient
f = fraction impervious (0.0 to 1.0)
${}^{10}I_1$ = 10 year ARI, 1 hour rainfall intensity

For ARIs other than 10 years the C_{10} value is multiplied by a frequency factor from **Table 2.2**. Hence:

$$C_y = F_y\, C_{10} \qquad (6)$$

where F_y = Frequency factor.

Table 2.2 Frequency Factors for Rational Method Runoff Coefficients (after 1987 ARR)

ARI (Years)	Frequency Factor, F_y
1	0.80
2	0.85
5	0.95
10	1.00
20	1.05
50	1.15
100	1.20

In view of the absence of data to support the urban runoff coefficient estimation procedure proposed in the 1987 ARR the comment of Munro (1956) may still apply:

> "*The literature abounds with tabulations of graphs of C for various conditions, but few are observed from reliable evidence .... Apparently, Horner and Flynt (1936) are the only ones to have carried out a really comprehensive set of measurements.*"

2.4 DISCUSSION

At the time of publication of the 1987 ARR the Rational Formula continued to attract wide spread use both in Australia and overseas as indicated by Mein and Goyen (1988).

As indicated by Hicks et al (2009) the urban Rational Method presented in 1987 ARR remained in its deterministic form notwithstanding a probabilistic version of the rural Rational Method was presented I the 1987 ARR based on the fitting of regionally varying C values based on a large number of rural gauged flow data records. This method was based on the research by Pilgrim and McDermott (1983) who formulated a probabilistic version of the Rational Method for small rural catchments in Eastern NSW.

It was considered in 1987 that there were insufficient gauged flow records to attempt to introduce a regionally based probabilistic urban Rational Method.

During the consultation period held prior to the release of 1987 ARR a study was carried out in the ACT at the request of the ACT Government to review the possible effects of differences between the urban Rational Method procedures as recommended in 1977 ARR and 1987 ARR. This review is described in a report titled “Drainage Design Practice for Land Development in the ACT. Part I: Rational Formula Procedures”, Willing and Partners (1989) which is attached in **Appendix E.**

This report ultimately recommended a semi-probabilistic based procedure for urban drainage design undertaken using the Rational Method in the ACT. The recommended procedure was based on the outcomes of testing different combinations of the 1977 and 1987 procedures for estimating runoff coefficient and time of concentration for estimating runoff coefficient to estimate flow peak quantiles in two gauged urban catchments. The estimated flow quantiles were then compared with peak flows determined using a flood frequency analysis. It was found that the combination of the procedures for estimating runoff coefficient and time of concentration given in the 1977 ARR best fitted the flood frequency curves from 2 yr ARI to 100 yr ARI.

The 1958 ARR provided a comprehensive procedure known as the "Tangent Check" to determine the critical time for an area and the appropriate partial area to be applied in the Rational Formula procedure. It was argued that a portion of the catchment area when multiplied by the higher rainfall intensity resulting from a shorter time of concentration could provide a higher peak flow than the peak flow contributed by the total area.

The 1977 ARR subjectively recommended against the use of partial area assessments including the "Tangent Check" on the premise that the Rational Method was not accurate enough to warrant such a check.

The 1987 ARR re-assessed the partial area question and recommended a single partial area check by calculating a partial area based on the times of concentration of impervious zones directly connected to the pipe system. Hence, 1987 ARR falls significantly short of the 1958 ARR recommendations for the checking of partial areas.

This deficiency is particularly important since it has been previously reported (Willing & Partners, 1983) that peak flows in urban stormwater systems can be seriously underestimated by ignoring partial area effects.

Since the publication of 1987 ARR a number of water authorities as well as Councils have also published their own recommendations for how the Rational Formula should be applied to urban catchments in their jurisdiction. Typically these guidelines recommend procedures for estimating runoff coefficient and time of concentration which differ from those recommended in the 1987 ARR.

3 REVIEW OF GAUGED URBAN CATCHMENTS

The availability of good quality gauged data gauged in urban catchments is a pre-requisite to any update of the Rational Formula method to reduce the potential error levels in the peak flows estimated using the procedure and/or to provide guidance on rainfall-runoff model parameters.

Urban catchments with long term flow gauging of say over 20 years or more in Australia, as in most western countries, are relatively rare. Even the number of urban catchments across Australia that have been gauged for even shorter periods to facilitate the calibration of catchment models using discrete storm events has also been limited.

The evolution of gauged urban catchments in Australia since the 1970s is overviewed as follows.

3.1 GAUGED URBAN CATCHMENTS IN THE 1970s

In 1975 only six urban or urbanising catchments were identified by Aitken in a study to investigate the hydrology and design of urban stormwater drainage systems for the Australian Water Resources Council Technical Paper No 10 (Aitkken, 1975)

These six catchments are reproduced in **Table 3.1**.

Table 3.1 Gauged Urban Catchments in 1975 (after Aitken, 1975)

Catchment	Area (ha)	Urbanisation Fraction	Slope (%)
Vine Street, Main Drain, Victoria	76.7	1.00	0.22
Yarralumla Creek at Mawson, ACT	510	0.72	2.9
Yarralumla Creek at Curtin, ACT	2770	0.57	1.3
Elsternwick Main Drain, Victoria	3210	1.00	0.44
Bulimba Creek at Mansfield, Queensland	5440	0.25	0.31
Kedron Brook at Technical College, Queensland	5620	0.56	0.43

There was a very limited available data set however it still allowed the probable responses of the total catchments to a number of individual gauged storm events to be investigated.

In a following AWRC study undertaken in 1977, Black and Aitken summarised the available gauged urban catchments in Australia at the time. The table has been reproduced in **Table 3.2**. It discloses a significant increase in the number of gauged urban catchments in comparison with the six catchments identified by Aitken in 1975. In fact, a total of 69 urban catchments across Australia were being gauged in 1977 with a further 5 catchments being proposed for gauging. The breakdown of candidate catchments was:

ACT (Canberra)	7	NSW (Sydney)	3
QLD (Brisbane)	13	NT (Darwin)	3
VIC (Melbourne)	24	TAS (Hobart)	0
WA (Perth)	11	SA (Adelaide)	8

Black and Aitken, 1975 stated that on the basis of the listed gauged catchments the *"the situation is potentially a very good one".*

Table 3.2 Gauged Urban Catchments in Australia in 1977 (after Black and Aitken, 1977)

OPERATING AUTHORITY	NUMBER OF CATCHMENTS	CATCHMENT SIZE, ha	WATER QUALITY MEASUREMENTS	PURPOSE OF INSTALLATIONS**
Environmental Protection Agency, Victoria	1	401	Yes	Determination of pollutant loads from urban catchment
Melbourne and Metropolitan Board of Works, Victoria	13	55-39600	Yes (for 6 catchments)	Determination of R-R and R-R-WQ relationships, flood forecasting
Dandenong Valley Authority, Victoria	10	770-27000	Yes	Determination of R-R and R-R-WQ relationships
Metropolitan Water, Sewerage and Drainage Board, New South Wales	2 (4 proposed)	234 & 290	No (Yes, for 4 proposed)	Determination of R-R relationships (Proposed catchments for R-R-WQ)
Wood and Grieve, Consulting Engineers, Western Australia	1	7	No	Determination of R-R relationship
Metropolitan Water Supply, Sewerage and Drainage Board, Western Australia	10	79-3600	Yes (for 1 catchment)	Determination of R-R and R-R-WQ relationships
Department of Construction, Australian Capital Territory	7	10-10050	Yes	Determination of R-R and R-R-WQ relationships
Department of Northern Territory, N.T.	3	22-1165	Yes	Determination of R-R and R-R-WQ relationships
Engineering and Water Supply Department, South Australia	4	1680-11840	Yes	Water quality monitoring
Brisbane City Council, Queensland	13	1750-13500	No	Determination of R-R relationships
State Pollution Control Commission, New South Wales	1 (1 proposed)	86	Yes	Determination of R-R-WQ relationships
Private consultants* in South Australia	4	116-641	No	Determination of R-R relationships

*Data not readily available.

**R-R denotes rainfall-runoff and R-R-WQ denotes rainfall-runoff-water quality.

3.2 GAUGED URBAN CATCHMENTS IN THE 1980s & 1990s

Boyd et al (1994) included a table listing Australian as well as overseas gauged urban catchments. Only nine gauged urban catchments in Australia were identified (refer to the first nine catchments summarised in **Table 3.3**). A tenth catchment was analysed by Bufill, 1989 (refer Table 3.3).

It is unclear whether more catchments were available for the research undertaken by Bufill, 1989.

Table 3.3 Gauged Urban Catchments in Australia in 1989 (after Bufill, 1989)

Catchment	Area (ha)	Imp Fraction.	No. of Events	Ratio of mean peak flows*	Maximum Event Difference Fraction +	
				Q_p (predicted) / Q_p (observed)	Negative	Positive
Maroubra, NSW	57.26	0.52	39	0.97	0.62	2.05
Strathfield, NSW	234	0.50	78	0.94	0.30	1.63
Jamison Park, NSW	20.58	0.357	85	0.98	0.17	2.56
Fisher's Ghost Creek, NSW	226	0.36	23	1.06	0.72	1.30
Giralang, ACT	96	0.25	14	0.77	0.29	1.05
Long Gully Ck, ACT	502	0.0478	14	-	-	-
Yarralumla Ck - Mawson, ACT	445	0.2584	11	0.84	0.56	1.34
Yarralumla Ck - Curtin, ACT	2690	0.1710	14	1.13	0.75	1.50
Vine Street, Victoria	70	0.314	11	1.21	0.83	1.5
Elster Ck, Victoria	3175	0.21	3	0.98	0.66	1.28

3.3 GAUGED URBAN CATCHMENTS IN THE 2000s

Hicks et al. (2009) identified 24 urban gauged catchments based on a number of criteria:

- Area less than 20 km^2 (smaller areas preferable, in the order of 1 km^2);
- Continuous records greater than 10 years in length;
- Fairly urbanised (greater than 50%);
- Acceptable gauge rating (max gauged flow: max recorded flow); and
- Stationary upstream urbanisation.

The table has been reproduced in **Table 3.4**. The breakdown of candidate catchments is:

ACT (Canberra)	5	NSW (Sydney)	2
QLD (Brisbane)	3	NT (Darwin)	2
VIC (Melbourne)	3	TAS (Hobart)	2
WA (Perth)	2	SA (Adelaide)	5

Most recently as part of an assessment of urban rainfall losses under ARR Project 6 Stage 2 – Losses for Design Flood Estimation, the Hicks et al (2009) criteria were adopted for the identification of candidate gauged urban catchments with some minor adjustments as follows:

- Area less than 5km^2 (500 ha), so that spatial variability in rainfall has less of an impact on the analysis (due to the use of point rainfall data);
- Record lengths of at least 10 years;

Table 3.4 Gauged Urban Catchments in Australia in 2009 (after Hicks et al, 2009)

STATE	#	station	river	Area (km2)	Max stage (m)	Max Gauged Stage (m)	Years
ACT	410746	Phillip	Long Gully Ck	4.8	1.28	1.13	39.1
ACT	410753	Mawson	Yarralumla Ck	4.4	1.67	0.82	38.1
ACT	410763	Giralang	Stormwater Drain	0.9	1.3	0.21	36.1
ACT	410764	Gungahlin Catchment	Ginninderra Trib	1.1	0.95	0.69	18.0
ACT	410763	Giralang West	Stormwater Drain	0.1	1.68	0.06	33.1
NSW	23	Strathfield	Powells Creek	2.3			47.0
NSW	213006	Bradbury Park	Fishers Ghost Ck	2.5	1.62	0.62	29.1
QLD			Highland Park - Gold Coast	2.0			10.63
QLD	143022A	Interstate Railway	Stable Swamp Ck	19	6.96	3.71	11.0
QLD	143028A	Jason St	Ithaca Ck	10	4.31	1.98	37.1
TAS	353	Gore St	Hobart Rivulet	16.3	1.06		49.1
TAS	353	Argyle St	Hobart Rivulet	19	1.8		46.1
VIC			Stony Ck at Spotswood		1.124	2.5	10.79
VIC	228229A	Tecoma	Monbulk Ck	19	1.57	0.88	21.1
VIC	407257A	Bendigo- Quarry Hill	Back Ck	14	2.62	2.02	12.0
SA	AW504546	Paddocks Inlet	Para Hills Drain	0.6	1.7	1.55	19.1
SA	AW504561	Glenelg	Frederick St Drain	0.6	63.4		17.1
SA	AW504579	Forsyth Grove	Third Ck	17	2.04	2.3	13.1
SA	AW504589	Lake	Urrbrae Wetlands	3.8	76		10.1
SA	AW504582	D/S West St	Adelaide Tce Pipe		2.94	2.52	13.1
NT	G8150231		Moil Catchment U	0.4	2.35	1.34	25.1
NT	G8150233	McArthur Park	Palmerston Catch	1.4	2.98	2.32	26.1
WA	602006	Duck Lake	Albany Urban Drain	0.1	10.39		10.4
WA	616087	Abernethy Rd	South Belmont Main Drain	11.3	11.52	10.61	21.6

- High quality measurements (more than 70% of the record classed as reasonable or high quality based on descriptors provided by the data collector);
- Fairly urbanised (% urbanisation by area greater than 50%), although variation in % urbanisation is desirable across catchments;
- Variation in effective runoff modifiers across catchments (eg. age of catchment, roof drainage methods, type of urbanisation etc.);
- 1 – 2 catchments for each state.

It was noted that in a number of states, there are only a minimal number of gauged catchments and therefore some flexibility was undertaken in the catchment selection process. For instance, no preference was identified for the Northern Territory and Tasmania due to the lack of information on the catchments identified by Hicks et al. (2009).

The set of candidate catchments were ranked for each state based on the above criteria. In assessing the suitability of these catchments, the following was carried out at in the initial stage:

- Percent urbanisation and age were determined qualitatively using Google Maps and an estimated extent of the catchment (since no catchment delineations were available);

- Where available, data quality was analysed based on descriptors provided by the data collector which consider the quality of measurement and correction methods.

Table 3.5 summarises the details of the selected urban catchments.

Table 3.5 2013 Study Catchment Details

State	**Catchment Name**	Total Area* (TA) (ha)	Urban Area^ (UA) (ha)	Total Impervious Area (TIA) (ha)	Urban TIA Fraction#
ACT	Giralang	90.98	61.8	28.4	46%
NSW	Powells Creek	231.9	223.4	151.7	68%
NT	McArthur Park	143.7	120.2	53.7	45%
QLD	Ithaca Creek	925.7	262.1	127.6	49%
SA	Parra Hills Drain	55.1	48.5	26.9	55%
TAS	Hobart City – Argyle Street	1,895.6	490.6	291.8	59%
VIC	Kinkora Road	202.1	184.2	121.9	66%
WA	Albany Drain near Duck Lake	8.2	8.2	2.9	35%

**Determined using the desktop GIS method*

^The Urban Area is classified as the total developed area excluding large open space

#The TIA fraction is defined as the percentage of impervious area in the urban area and was based on the desktop GIS method.

3.4 DISCUSSION

It is obvious from the above overview of gauged urban catchments that the early predictions of Black and Aitken (1977) that "the situation is potentially a very good one" has not been borne out over the subsequent 35 years.

The obvious conclusion is that there still remains a scarcity of suitable gauged urban catchment with sufficient data to update the Rational Formula method in order to reduce the potential error levels in the peak flows estimated using the procedure. This is not to say that additional analysis using the limited data already available could not lead to significant insights into the varying hydrologic responses experienced in urban catchments across Australia.

As in 1975 there still remains an urgent need for the collection of long term rainfall and flow data in gauged urban catchments to facilitate the updating of the Rational Method and/or other urban rainfall-runoff estimation procedures. However before considering any additional gauging of urban catchments in Australia it is important that the current catchments identified by Hicks et al, (2009) be carefully reviewed to consider the utility of the data which has already been collected and/or identify measures that need to be put in place to maximise the value of the data already collected and to be collected in the future. This could include identifying the impact of any retarding basins or other measures in a gauged urban catchment. A basin can modify the catchment runoff response by infiltrating any overland flows into the grassed base of a basin in frequent events and by reducing peak flows in major events eg. the basin in the McArthur Park catchment in Palmerston, NT.

In some cities such as Sydney for example, there may be also three or more distinct regions (eg. coastal catchments on sand, inland catchments on heavier loam clays, catchments with steeper slopes and/or ridgeline development only). Additionally drainage strategies can range from older systems in older suburbs (eg. Powells Creek catchment) to newer suburbs (eg. Hewitt catchment) only 30 km away, that includes a more contemporary, directly connected drainage scheme which in some new subdivisions include significant WSUD measures.

To this end catchment simulation models should be established for each of the listed urban catchments and an attempt be made to carry out a preliminary study on each similar to those described in **Appendix D**. These studies would test the utility of the data already collected and identify any issues with existing infrastructure that may distort the data or identify data that should be also collected. This would allow either the addition of additional infrastructure to overcome any existing problems or provide missing data or in some cases justify the cessation of the collection of data at the current site. Additional catchments could then be considered with future gauging sites only being selected after first passing a preliminary catchment analysis.

If the approach described in **Sections 4.3** or **4.4** was adopted it may well be possible to minimize the number of additional gauged catchments that would need to be established and monitored. Using long term rainfall records rather than long term flow gauging records may allow short term (3-5 years) snap shots of urban catchment which may be changing over time to be analysed to allow sufficient calibration of a rainfall-runoff model that could then be used to estimate flow quantiles. It is anticipated that there is a large amount of valuable event data available in many of the gauged catchments identified above that could be extracted and used to facilitate the updating of the Rational Method and/or other urban rainfall-runoff estimation procedures. This methodology is further described in **Section 4.3**.

The first priority in any future gauging initiatives should be therefore to increase the number of pluviographs located within existing gauged catchments to facilitate improved calibration of catchment models.

It is important that any future gauging be recorded at time steps far shorter than 6 minute interval which is currently accepted generally. This is extremely important for the simulation of urban catchments less than say 300 ha in area.

Questions relating to future gauging that still need to be quantified include:

- What is it worth to the taxpayer?
- What is the return on investment?
- Are there any future liabilities for Authorities and or Engineers Australia in recommending design methods based on intuition rather than gauged data?
- It would appear that past editions of ARR have not highlighted likely possible errors due to the scarcity of data to quantify errors.

While the scarcity of data has continued the complexity of drainage systems continues to evolve. We have moved from simple pipes and pits to widespread use of retarding basins, on-site detention (OSD) and WSUD measures at lot, neighbourhood and regional scales. Additionally the average size of lots has reduced substantially while the size of houses leading the imperviousness of urban catchments to substantially increase over time.

Urban gauging in the past has been carried out by many different government and private organizations at different times. This has often led to many gauging programs, which are enthusiastically supported in the beginning, falling by the wayside before the maximum benefit could be derived from the gauging program.

It may be possible Engineers Australia could co-ordinate or even project manage ongoing urban data collection as a legacy of the current revision of ARR to ensure the next few decades provide far more fruitful data then the last 30-40 years. This data could be then made available to universities and hydrologists to support the development of improved urban flow estimation and design procedures.

The aim would be to significantly reduce the potential error bands when recommending suitable urban drainage management solutions to meet both current and future demands in any urbanising catchment in Australia. The challenge is how best to fund any data collection that will be sustainable into the future.

3.5 RECOMMENDATIONS

Based on the review of gauged urban catchments in Australia since the 1970s it is recommended that:

(viii) Engineers Australia consult with major stakeholders to formulate a strategy to ensure the current collection of data is maintained and that data collection is expanded to encompass representative urban catchments across Australia to ensure that sufficient good quality data is available to allow the update of the Rational Formula method to reduce the potential error levels in the peak flows estimated using the procedure and/or to improve the guidance on rainfall-runoff model parameters for urban catchments;

(ix) Existing gauged urban catchments be reviewed to identify any features that may be distorting gauging records (eg. basins) and that any review should include preliminary simulation studies to quantify the effect of any features and the need or otherwise to develop a procedure to correct the gauged data;

(x) Existing gauged catchments should be categorised based on regions, topography, geology and/or drainage systems;

(xi) Identify possible urban catchments that could be gauged to provide data for any regions, topography, geology and drainage systems not represented by existing gauged catchments;

(xii) Undertake preliminary modelling of any new candidate gauged catchments;

(xiii) Filter future potential gauged catchments to prioritize installations;

(xiv) As a matter of priority seek to increase the density of rainfall gauges across existing gauged catchments to further qualify areal effects within smaller urban catchments.

4 POSSIBLE USES OF CURRENT AVAILABLE GAUGED URBAN DATA

One of the aims of this Discussion Paper was to identify potential uses of the gauged urban streamflow data that is currently available. The current available gauged urban streamflow data could be used to undertake Part I, Part II, Part III or Part IV studies of current gauged urban catchments as discussed below.

4.1 PART I STUDIES

In 1989 a review the possible effects of differences between the urban Rational Method procedures recommended in 1977 ARR and 1987 ARR was undertaken in the ACT (Willing & Partners, 1989). This review is described in a report titled "Drainage Design Practice for Land Development in the ACT. Part I: Rational Formula Procedures (**Part I Study**) which is attached in **Appendix E.**

This report ultimately recommended a semi-probabilistic based procedure for urban drainage design undertaken using the urban Rational Method in the ACT. The recommended procedure was based on the outcomes of testing different combinations of the 1977 and 1987 procedures for estimating runoff coefficient and time of concentration for estimating runoff coefficient to estimate flow peak quantiles in two gauged urban catchments. The estimated flow quantiles were then compared with peak flows determined using a flood frequency analysis. It was found that the combination of the procedures for estimating runoff coefficient and time of concentration given in the 1977 ARR best fitted the flood frequency curves from 2 yr ARI to 100 yr ARI.

A key conclusion of the Part I study was that the runoff coefficient and time of concentration relationships are paired ie. they both need to be derived concurrently using gauged data rather than derived relationships independently.

Since 1989 additional data has been collected in the Giralang catchment which has allowed the updating of the 1987 analysis as well as the preliminary testing of the sensitivity of the predicted peak flows to characterising a catchment based on total impervious area (TIA) or effective impervious area (EIA) as assessed in ARR Project 6 Stage 2 - Analysis of Effective Impervious Area & Pilot Study of Losses in Urban Catchments.

The preliminary application of the Part I study approach to gauged urban catchments in Canberra, Sydney, Melbourne and Darwin is described in **Appendix C**. The peak flows were estimated for various representations of each urban catchment using a single node **xprathgl** model of each catchment.

The conclusions from these preliminary analyses are as follows.

4.1.1 Canberra and Sydney

It was concluded from a comparison of the various Giralang catchment results that:

- The 1977 ARR procedures give peak flows which match the peak flows adopted for the composite series based on flood frequency analysis (FFA) except for flows based on EIA only;
- For 10 yr ARI and above the 1987 ARR procedures give similar peak flows to the ARR Project 5 procedures for rural catchments;

- The 1987 procedures give peak flows lower than the peak flows adopted for the composite series based on flood frequency analysis with the estimated 100 yr ARI peak flow comparable to the 10 yr ARI peak flow from the FFA;

It was concluded from a comparison of the various Hewitt catchment results that:

- The 1977 ARR procedures give peak flows which are slightly lower than the peak flows adopted for the composite series with the estimated 100 yr ARI peak flow being comparable to the 50 yr ARI peak flow from the FFA.
- The 1987 procedures give peak flows lower than the peak flows adopted for the composite series with the estimated 100 yr ARI peak flow being comparable to the 10 yr ARI peak flow from the FFA.

It was concluded from a comparison of the various Powells Creek catchment results that:

- The 1977 ARR procedures give peak flows which are higher than the peak flows obtained from an annual series analysis of gauged flows with the peak flows estimated for frequent runoff up to 10 yr ARI being significantly higher;
- One approach to improve agreement would be to test Curve No. 6 in comparison with the adopted Curve No. 5;
- The 1987 procedures give peak flows slightly higher than the peak flows adopted for the annual series but in good agreement.

4.1.2 Melbourne

Based on the results presented in **Table C.8** it is apparent all Rational Method peak flows are significantly higher than corresponding flood frequency peak flow estimates except where agreement is forced by adjusting the runoff coefficient or the time of concentration.

Based on the work of Pomeroy et al (2013) the Kinkora Road urban catchment shares many characteristics with the Powells Creek urban catchment in Sydney. The peak flows in both catchments appear to derive mostly from the EIA only. This may well encompass only the roads themselves plus very limited amounts of in block hard surfaces.

This also highlights the potential problems of adopting a limited number of long term gauged urban catchments as representative of all urban catchments. The Kinkora Road and Powells Creek catchments are probably representative of many older suburbs which were first developed in the 1950s or 1960s. They are however not representative of newer catchments with high degrees of directly connected impervious areas including the Hewitt catchment in Sydney and the Giralang catchment in ACT.

4.1.3 Darwin

It should be noted that roof drainage guttering across the Moil catchment is very limited with the majority of runoff simply falling into the allotment yard. It was concluded from a comparison of the various results for the Moil catchment that:

- The 1977 ARR procedures give peak flows which are slightly higher than the peak flows adopted for the composite series (based on the adoption of Curve 6 for runoff coefficients);

- The 1987 procedures give peak flows higher than the peak flows adopted for the composite series for events greater than a 10 yr ARI event.

In the case of the McArthur Park catchment in Palmerston, it was found that most predicted peak flows estimated using the 1977 ARR or 1987 ARR procedures gave peak flows considerably higher than the peak flows estimated by FFA of the gauged flows from the McArthur Park catchment.

While one approach to improve agreement would be to test Curve No. 5 in comparison with the adopted Curve No. 4 it was noted however that these FFA results are problematic due to the presence of a large retarding basin located upstream of the gauging station which can modify the runoff response from a significant proportion of the catchment by infiltrating any overland flows into the grassed base of the basin in frequent events and by reducing peak flows in major events.

4.1.4 Discussion

Since the publication of 1987 ARR a number of water authorities as well as Councils have also published their own recommendations for how the Rational Formula should be applied to urban catchments in their jurisdiction. Typically these guidelines recommend procedures for estimating runoff coefficient and time of concentration which differ from those recommended in the 1987 ARR. It is unclear if these guidelines are based on a comprehensive Part I study where the runoff coefficient and time of concentration relationships were derived concurrently or are *values which are somewhat arbitrary and based on intuitive judgement rather than adequately controlled experiments* (as highlighted in the 1958 ARR).

It is apparent from a comparison of the discussion in **Section 3** and **Appendix A** that in the past considerably greater effort has gone into flow gauging in rural catchments compared to flow gauging in urban catchments notwithstanding 70% of the population of Australia lives in Sydney, Melbourne, Brisbane, Perth, Adelaide, Canberra, Hobart and Darwin.

Stage 2 of Project 5 has assembled a quality controlled national database consisting of 727 stations located in rural catchments while Hicks et al (2009) identified 24 gauged urban catchments across Australia ie. there are 30 rural flow gauging stations for every 1 urban gauging station in Australia.

The length of record at stations in urban catchments is also often restricted to 10 years or less. As disclosed by Hicks et al (2009) the number of urban catchments (500 ha or less) with 20 years of records is only 11, with 30 years of record is 7, with 40 years of record is 4 and with 50 years record is 1 only.

Even if all the urban catchments listed in **Table 3.4** were studied using the Part I Study approach this would still only result in some 24 catchments to cover all of Australia. The results would also be specific to each catchment's hydrological regime, topography, geology and its stormwater drainage management strategy.

A possible approach to increase the number of test catchments would be to undertake rainfall and flow gauging in new catchments for a period of 3-5 years only and to apply a Part III or Part IV study approach (refer **Sections 4.3** and **4.4**) to create benchmark flood frequency curves for these new catchments as the basis for the testing of runoff coefficient and time of concentration relations within a catchment ie. further Part I studies.

4.2 PART II STUDIES

In 1993 a study was undertaken in Canberra to provide practice guidelines when utilising hydrograph based estimation procedures in urban drainage projects in the ACT (Willing & Partners, 1993). The work followed on from the earlier Part I study. It is described in a report titled "Drainage Design Practice Part II", Willing and Partners (1993) which is attached in **Appendix F.**

The goal of the Part II study was to test several currently available rainfall/runoff computer programs including RAFTS, RORB and ILSAX on Canberra's gauged urban catchments.

In particular, the objectives were to determine appropriate:

(i) design rainfall loss rate estimation parameters applicable to individual programs,
(ii) surface runoff routing parameters for pervious and impervious areas specific to each program tested, and
(iii) design storm event modelling procedures specific to each program tested.

Since the 1993 study was completed an addition of 20+ years of rainfall and runoff data collected including 3 years of data collected on micro catchments embedded within the Giralang urban catchment. Data from the micro catchments was collected and reported in the PhD thesis submitted by Goyen in 2000. The research reported by Goyen, 2000 further examined the processes within the Giralang catchment as well as the Hewitt urban catchment located near Penrith in Sydney.

A potential problem with the Part II study in the ACT was the recommended initial (and high) values for moisture stores. An embedded approach has been assessed to establish if it performs better than fixed initial values in a vertical water balance loss model.

These issues are explored in the analysis of the Giralang catchment (in Canberra) and Hewitt catchment (in Sydney) in **Appendix D**. These investigations applied a modified sub-catchment hydrograph estimation module from **xprafts** as described by Goyen (2000).

The modifications to the **xprafts** analysis procedure included an alternate sub-catchment analysis procedure that is indicated diagrammatically in **Figure D.4**. Runoff is estimated separately for the roof and gutter, adjacent road surface and paving and pervious gardens and lawn areas. A virtual allotment drainage network is constructed to represent lagging, bypass, capture and additional storage routing and infiltration/ evapotranspiration within the various WSUD facilities. The outputs from each structure as well as any bypass flows are combined to give the total runoff hydrograph from a typical allotment.

The method allows the definition of a wide range of WSUD/LID facilities including allotment storage devices, infiltration beds and rain water tanks. The procedures allows for variable structure sizes as well as variable capture and bypass percentages. Additional parameters to define the percentage breakdown in impervious surfaces between roofs, paving and road surfaces was also included.

The models as described by Goyen (2000) were adopted without any modification apart from the addition of evaporation from impervious surfaces during the extended duration summer daytime events.

An excellent level of agreement was achieved between gauged and predicted flows at the micro catchment and urban catchment scale in Giralang and at the urban catchment scale in Hewitt.

As discussed in **Appendix D**, if the simulation of historical storms is able to match the observed events as closely as achieved by Goyen (2000) in the Giralang catchment in the period 1993 – 1995 then the resulting flow quantiles obtained from FFA of the simulated peak flows should closely match the flow quantiles derived from the gauged peak flows. This creates an opportunity to use a calibrated hydrological model (of the form assembled by Goyen (2000)) to estimate peak flows from storm events extracted from long term pluviograph records and to then undertake FFA of the synthetic peak flows to estimate flow quantiles ie. a Part III or Part IV study approach.

This in turn could inform the determination of parameter values for other hydrological models (a Part II study).

4.3 PART III STUDIES

A possible approach to increase the number of test catchments would be to undertake rainfall and flow gauging in new catchments for a period of 3-5 years only and to apply a Part III study approach to create benchmark flood frequency curves for these new catchments as the basis for the testing of runoff coefficient and time of concentration relations within a catchment ie. further Part I and/or Part II studies.

The Part III method involves the calibration of a hydrological model (of the form assembled by Goyen (2000)) against a range of storm events for which there is gauged rainfall and runoff. A sufficient number of storm events would then be extracted from long term pluviograph records and the calibrated model would be run to estimate peak flows. A FFA of the peak flows could then be undertaken to estimate the flow quantiles.

This approach has been previously proposed by Aitken (1975) to utilize the available long term rainfall pluviograph record nearest a catchment together with short term calibration records to simulate all the major rainfall events in the rainfall record.

If multiple long term rainfall stations existed near or within the catchment the problems of rainfall spatial variance could also be eliminated or at least minimized.

4.4 PART IV STUDIES

The Part IV study approach is similar to the Part III study approach. However, instead of calibrating a hydrological model (of the form assembled by Goyen (2000)) against a range of storm events for which there is gauged rainfall and runoff the hydrological model would be calibrated using continuous simulation for the period of gauging. This calibrated model would then be used to run the long term pluviograph record(s) and predicted peak flows would then be extracted to allow a FFA to be undertaken to estimate the flow quantiles.

Any continuous simulation would most likely rely on a scheme where the time step lengthens during dry spells and reduces to a time step of say 1 minute during storm events.

5 APPLICATION OF THE URBAN RATIONAL METHOD

5.1 SHOULD THE URBAN RATIONAL METHOD CONTINUE TO BE INCLUDED IN ARR

The Rational Method has been included in each of the Australian Rainfall and Runoff documents since the release of the first edition in 1958. The ARR 1987 recommendations for the application of the urban Rational Formula are somewhat vague. They state that appropriate uses include design of small and medium street drainage systems, and large property drainage systems. Other authorities restrict the application of the urban Ration Method to urban catchments less than 400 hectares and in the case of some Councils this is further restricted to less than 1 hectare.

Since the publication of 1987 ARR a number of water authorities as well as Councils have also published their own recommendations on how the Rational Method should be applied to urban catchments in their jurisdiction. Typically these guidelines recommend procedures for estimating runoff coefficient and time of concentration which differ from those recommended in the 1987 ARR. It is unclear if these guidelines are based on a comprehensive study of one or more gauged urban catchments or whether *values which are somewhat arbitrary and based on intuitive judgement rather than adequately controlled experiments* (as concluded in the 1958 ARR).

There are, however, a number of problems associated with the use of the Rational Method. Most of these problems are associated with the estimation of parameter values such as the time of concentration and the runoff coefficient. As a result, the Rational Method may be easy to implement, but it is difficult to ensure that the predictions adequately represents processes occurring in the catchment.

In 1989 a review the possible effects of differences between the urban Rational Method procedures recommended in 1977 ARR and 1987 ARR was undertaken in the ACT. A key conclusion of this Part I study was that the runoff coefficient and time of concentration relationships are paired ie. they both need to be derived concurrently using gauged data rather than derived relationships independently.

It was concluded from a preliminary updated analysis of the Giralang catchment in the ACT that

- The 1977 ARR procedures give peak flows which match the peak flows adopted for the composite series based on flood frequency analysis (FFA) except for flows based on EIA only;
- The 1987 procedures give peak flows lower than the peak flows adopted for the composite series based on flood frequency analysis with the estimated 100 yr ARI peak flow comparable to the 10 yr ARI peak flow from the FFA;
- For 10 yr ARI and above the 1987 ARR procedures give similar peak flows to the ARR Project 5 procedures for rural catchments.

The preliminary assessment of gauged urban catchments in Sydney, Melbourne and Darwin disclosed that in general the 1977 ARR Rational Method gives peak flows which better match the peak flows calculated by flood frequency analysis (FFA) than the 1987 ARR Rational Method (refer **Appendix C**).

Notwithstanding that the 1989 Part I study concluded that the results from the study lent further support to the continued use of the Rational Formula for drainage design in small to medium sized urban catchments this was on the basis that further studies be undertaken to further examine possible modifications to the recommended 1987 ARR procedures to improve the estimation of surface flow times of concentration and corresponding runoff coefficients. In particular, it recommended that further studies should aim to determine appropriate surface roughness values for use in the kinematic wave formulation for overland flow in Australia.

These further studies have not been undertaken in the 24 years since.

Without carrying out similar studies to the Part I study undertaken in the ACT on a significant number of additional gauged urban catchments then it is the view of the authors that continued use of the Rational Method for urban drainage analysis and design can no longer be justified.

5.2 SHOULD THE URBAN RATIONAL METHOD BE USED TO CALIBRATE HYDROLOGICAL MODELS

With the advent of PCs in the 1980s and the improvements in computer speed and capabilities since that time as well as the continued development of urban rainfall runoff catchment simulation models, computer based modelling has almost totally supplanted the role of Rational Method calculations in urban drainage design. Notwithstanding these advances some authorities still require urban hydrological models to be “calibrated” to match peak flows estimated using the 1987 ARR urban Rational Method.

It is the view of the authors that the urban Rational Method should not be used to calibrate urban hydrological models unless it can be demonstrated that:

(iv) A detailed Part I study has been undertaken on one or more gauged urban catchments in the relevant city or town which has calibrated and validated relations for the calculation of runoff coefficients and times of concentration; and
(v) The urban catchment which is being modelled is subject to a similar hydrological regime and has a level of imperviousness comparable to the gauged urban catchment(s) analysed in the Part I study; and
(vi) WSUD measures are not present in the urban catchment which is being modelled.

6 CONSISTENCY WITH REGIONAL RURAL FLOOD METHOD

One of the aims of this Discussion Paper was to investigate if it is practical to develop a method to adjust the procedures recommended in Project 5 Regional Flood Methods to estimate peak flows in small to medium sized urban catchments. Project 5 is overviewed in **Appendix A**.

It is disclosed in **Appendix A** that in the past considerably greater effort has gone into flow gauging in rural catchments compared to flow gauging in urban catchments notwithstanding 70% of the population of Australia lives in Sydney, Melbourne, Brisbane, Perth, Adelaide, Canberra, Hobart and Darwin.

Stage 2 of Project 5 has assembled a quality controlled national database consisting of 727 stations located in rural catchments while Hicks et al (2009) identified 24 gauged urban catchments across Australia ie. there are 30 rural flow gauging stations for every 1 urban gauging station in Australia.

An initial benchmark annual and partial series analysis of gauged flows has been undertaken for nine urban catchments and one paired rural catchment as described in **Appendix B**. At the same time the peak flows for each catchment under pre-development (rural) conditions for 2, 5, 10, 20, 50 and 100 yr ARIs were estimated for most of these catchments using the procedures recommended under Project 5.

A comparison of the peak flows calculate from FFA and using the Project 5 procedures for the 2, 10, 20 and 100 yr ARIs are summarised in **Table 6.1** and the ratio of peak flows are plotted in **Figure 6.1**.

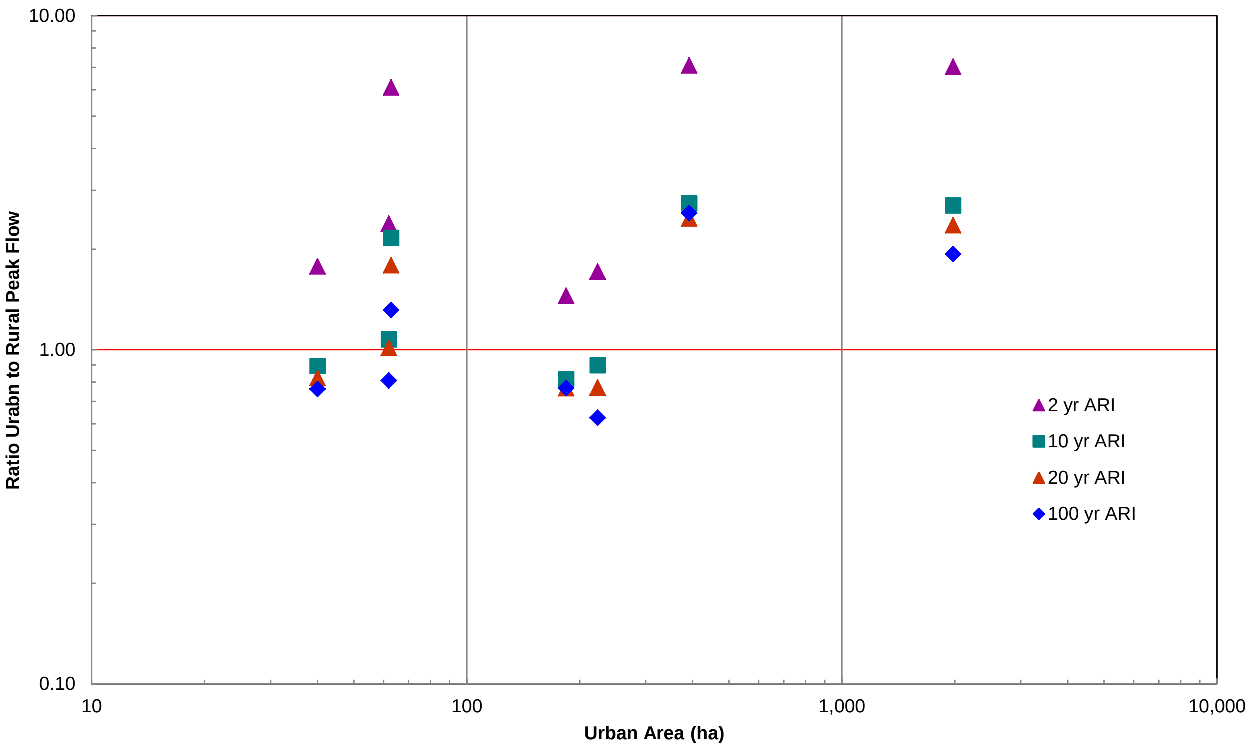

Figure 6-1 Ratio of Urban to Rural Peak Flows against Urban Area

Table 6.1 Estimated Urban and Rural Peak Flows in Selected Gauged Catchments

ARI (yrs)	2	10	20	100
Giralang, ACT				62.9
FFA Composite Series	6.7	9.3	10.7	14.2
ARR Project 5	1.1	4.3	6.0	10.8
Ratio Urban/Rural	6.09	2.16	1.79	1.31
Mawson, ACT				392
FFA Composite Series	22	34	43.0	80
ARR Project 5	3.1	12.4	17.4	31.2
Ratio Urban/Rural	7.10	2.74	2.47	2.56
Curtin, ACT				1980
FFA Composite Series	64	97	119.0	175
ARR Project 5	9.1	35.9	50.5	90.5
Ratio Urban/Rural	7.03	2.70	2.36	1.93
Hewitt				62
FFA Composite Series	6.2	11.8	15.8	22.8
ARR Project 5	2.6	11	15.6	28.2
Ratio Urban/Rural	2.38	1.07	1.01	0.81
Powells Creek				223.4
FFA Annual Series	13.7	30.1	36.6	53.8
ARR Project 5	8	33.5	47.5	86.1
Ratio Urban/Rural	1.71	0.90	0.77	0.62
Kinkora Road				184.2
FFA Annual Series	2.9	5.3	6.6	10.5
ARR Project 5	2	6.5	8.6	13.7
Ratio Urban/Rural	1.45	0.82	0.77	0.77
Moil				40
FFA Composite Series	7.1	10.1	11.6	15.1
ARR Project 5	4	11.3	14.1	19.8
Ratio Urban/Rural	1.78	0.89	0.82	0.76

It was concluded from the results presented in **Table 6.1** and plotted in **Figure 6.1** that:

- The 2yr ARI peak flows for all urban catchments (derived from FFA) are higher than the estimated 2 yr ARI peak flows under pre-development (rural) conditions (derived from Project 5);
- The ratio of urban to rural peak flows decreases as ARI increases;
- In the case of the Canberra urban catchments the 100 yr ARI peak flow (derived from FFA) are higher than the estimated 100 yr ARI peak flow under pre-development (rural) conditions (derived from Project 5)

- In the case of the Sydney, Melbourne, and Darwin urban catchments the 100 yr ARI peak flow (derived from FFA) are lower than the estimated 100 yr ARI peak flow under pre-development (rural) conditions (derived from Project 5).

It was further concluded that based on the scatter of the calculated ratios of urban to rural peak flows and the overestimation of rural peak flows in comparison with urban peak flows derived from FFA in major events in a number of catchments that it is not practical to develop a simple method to adjust the peak flows from rural catchments to give reliable estimates of peak flows in urban catchments at this time.

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APPENDIX A

ARR PROJECT 5 REGIONAL FLOOD METHODS

In Australia, there are many streams where there is little/no recorded streamflow data. In these ungauged and poorly gauged catchments, there is insufficient information/data to obtain design flood estimates which are needed to size hydraulic structures, plan and design other water infrastructure and undertake various environmental and ecological studies. Regional flood frequency analysis (RFFA) is the most commonly adopted technique to derive design flood estimates on the ungauged catchments. A RFFA method attempts to transfer flood characteristics information from a group of gauged catchments to an ungauged catchment of interest. The RFFA methods recommended in the Australian Rainfall and Runoff (ARR) in 1987 need updating to reflect the advancements in RFFA methods and new additional streamflow data. To update the RFFA methods in the ARR, a project team was formed in 2008 and since then the team has been carrying out research and investigations, which have now formed part of Project 5 „Regional Flood Methods in Australia in the ARR revision projects.

So far, Stage I and Stage II of Project 5 have been completed.

A.1 PROJECT 5 STAGE 1

The major outcomes of Stage I project were as follows.

- Formation of Project 5 team and establishment of contacts and cooperations with various state agencies to obtain necessary streamflow data and relevant information. About 31 researchers/engineers from over 14 organisations of various Australian states directly contributed to Project 5 Stage I;

- Preparation of initial version of national database which involved examination of a large number of potential stations from each state, short-listing of the stations, infilling the gaps in annual maximum flood series, test for outliers, test for trends and test for rating curve extrapolation error. In Stage I, databases for Victoria, NSW, Qld, Tasmania and SA were prepared;

- Based on detailed literature review, consultation with Project 5 team and various state representatives and ARR Technical Committee, a number of RFFA methods were selected for detailed investigation which included the Probabilistic Rational Method, Quantile Regression Technique and Parameter Regression Technique. For the regression-based methods, both ordinary least squares and generalised least squares methods were considered. For the formation of regions, fixed state-based regions and region-of-influence (based on geographical proximity) were considered.

- From initial trend analysis, a good number of stations showed trends in the annual maximum flood series data; these stations were not included in the development and testing of the RFFA methods. However, it was decided to conduct further investigation e.g. impact of serial and cross-correlation on the trends, and relationship between the identified trends and catchment and climate change/variability indices and impacts of the identified trends on regional flood estimates with respect to locations and ARIs of the flood estimates.

A.2 PROJECT 5 STAGE 2

The major achievements and/or findings from the Stage II project were as follows.

- A quality controlled national database consisting of 727 stations;

- That regression-based RFFA methods (such as the quantile regression technique (QRT) or parameter regression technique (PRT)) are preferable to the Probabilistic Rational Method;

- That that Bayesian QRT and Bayesian PRT methods perform very similarly for various Australian states. Since the PRT method offers several additional advantages over the QRT (namely, in the PRT flood quantiles increase smoothly with increasing ARIs and from the regional LP3 distribution, flood quantiles of any ARI (in the range of 2 to 100 years) can be estimated), this has been recommended for general application in Australia.

- From the comparison of fixed regions and region-of-influence (ROI) approaches, it has been found that, where a region contains a sufficient number of sites, the ROI approach outperforms the fixed regions. The mean annual flood model generally has the highest model error as compared to the SD and skew models. However, the SD and skew estimates are suffered greatly by sampling errors.

- The developed RFFA methods in Stage II require data of two or three climatic and physical catchment characteristics (i.e. catchment area, representative design rainfall intensity and mean annual rainfall), which are easy to obtain. This would make the application of the recommended RFFA methods easy and simple.

- It has been found that the recommended RFFA methods i.e. GLS-PRT-ROI and GLS-PRT-fixed region perform quite well for the smaller catchments in the database where there is no evidence that smaller catchments perform poorly than the medium and larger catchments. The possibility of extending the RFFA method to very small catchments beyond the limit of the current Project 5 database has been examined; however, further study is needed to develop an acceptable method.

- The development of a simple Large Flood Regionalisation Model for regional flood estimation in the major flood range was investigated in Stage I of the project (see Stage I report), which however did not consider the impacts of inter-station correlation of the annual maximum flood series among different pairs of stations on final design flood estimates. Some preliminary investigations on inter-station correlation have been undertaken in this report, which however needs further investigation.

- There is insufficient streamflow data availability at both temporal and spatial scales in the arid and semi-arid regions of Australia that can be used to develop statistically meaningful RFFA methods. A simplified index type RFFA is recommended for arid/semi-arid regions of Australia where four separate regions are recommended at this stage (this needs further development and testing before inclusion in the ARR).

In the preliminary investigation (see Stage I report), about 13% of the selected stations (for Project 5) showed a trend in the annual maximum flood series data. In the Stage II report, the impacts of serial and cross-correlation on trend analysis have been investigated. At the significance level of 10% and with the consideration of the cross-correlation among the sites in the network, the field significance of downward trends in the annual maximum flood series was detected over the whole country. However, the field significance of upward trends was discovered to be statistically non-significant at 10% significant level. The impacts of the identified trends on regional flood quantile estimates for ARIs in the range of 2 to 100 years will be investigated in Stage III of the project. This is expected to produce climate change adjustment factors as a function of ARIs and locations across Australia.

The testing of the recommended RFFA methods for Australia by various states/stakeholders in cooperation with the Project 5 team has been recommended. A set of future tasks has been identified. Also, the scope of developing an application tool/software has been indicated.

Stage II developed a firm basis for recommendations on the RFFA methods to be included in the revised ARR Chapter (4th edition). It has also identified future research and development work in Stage III of the Project, required to develop the Stage II findings into a final set of methods, design databases, user guidelines and application tools.

The results presented in this report are applicable to the rural catchments in the vicinity of the catchments selected in this study; this should not be applied to urban catchments.

APPENDIX B

FLOOD FREQUENCY ANALYSIS IN SELECTED GAUGED URBAN CATCHMENTS

B.1 DISPARITY BETWEEN NUMBER OF RURAL AND URBAN GAUGED CATCHMENTS

It is apparent from a comparison of the discussion in **Section 3** and **Appendix A** that in the past considerably greater effort has gone into flow gauging in rural catchments compared to flow gauging in urban catchments notwithstanding 70% of the population of Australia lives in Sydney, Melbourne, Brisbane, Perth, Adelaide, Canberra, Hobart and Darwin.

Stage 2 of Project 5 has assembled a quality controlled national database consisting of 727 stations located in rural catchments while Hicks et al (2009) identified 24 gauged urban catchments across Australia ie. there are 30 rural flow gauging stations for every 1 urban gauging station in Australia.

The length of record at stations in urban catchments is also often restricted to 10 years or less. As disclosed by Hicks et al (2009) the number of urban catchments (500 ha or less) with 20 years of records is only 11, with 30 years of record is 7, with 40 years of record is 4 and with 50 years record is 1 only.

The length of record can have a significant effect on the flood frequency curve.

When developing and testing a rainfall-runoff estimation procedure including a simple statistical form of the Rational Formula, the flood frequency curve derived from gauged flows or individual peak flow quantiles are the objective function.

Even if the quality of the gauged data is of a very high standard and the annual maximum peaks as well as any additional significant peak values are judged to be of high accuracy, this does not mean that the flood frequency curve calculated using a limited record length will be accurate over the full range of AEPs eg. up to 1% AEP.

B.2 FLOOD FREQUENCY ANALYSIS

The updating of the Rational Formula method to reduce the potential error levels in the peak flows estimated using the procedure and/or to improve the guidance on rainfall-runoff model parameters for urban catchments can only occur if there is a flood frequency curve available for the gauged catchment to provide the benchmark against which peak flow predictions can be tested in accordance with the approaches adopted in the Part I and Part II studies conducted in the ACT (Willing & Partners, 1989 and 1993). These studies are attached in **Appendix E** and **F** respectively.

In the 1987 edition of ARR, Chapter 10 on flood frequency analysis extensively discusses the methodology, limitations and qualifications associated with estimating a flood frequency curve from data of varying quality, inclusive outliers, and record length. The roles of partial and annual series in flood frequency analysis is described. The importance of the partial series is explained particularly when considering the frequent floods of say less than 10% AEP.

An initial benchmark has been created for nine urban catchments and one paired rural catchment based on annual and partial series analysis of gauged flows for each urban catchment. The catchments that have been analysed are summarised in **Table B.1**.

Table B.1 Gauged Urban Catchments in ACT, NSW, VIC and NT

State	Catchment	Total Area (ha)	Urban Area (ha)
ACT	Giralang	91	63
NSW	Yarralumla Creek at Mawson	413	392
NT	Yarralumla Creek at Curtin	2,701	1980
QLD	Gungahlin	112	0
NSW	Hewitt	62	
NSW	Powells Creek	232	224
NSW	Parramatta River at Parramatta	11,000	
VIC	Kinkora Road	202	184
NT	Moil	40	40
NT	McArthur Park	144	120

B.2.1 Canberra Gauged Catchments

The peak flows determined from flood frequency analysis of the gauged flows for the Giralang urban catchment are summarised in **Table B.2** for three different periods of record. This highlights the impact that the length of record can have on peak flows estimated by flood frequency analysis.

Table B.2 Flood Frequency Analysis of Peak Flows (m^3/s) in Giralang Catchment

Period of Record: 1973 - 1989

Return Period (years)	1.01	2	5	10	20	25	50	100
Partial Series		5.4	7.0					
Annual Series				8.7	10.5		11.5	13.2
Composite Series		5.4	7.0	8.7	10.5		11.9	13.2

Period of Record: 1973 - 1991

Return Period (years)	1.01	2	5	10	20	25	50	100
Partial Series		5.2	7.2					
Annual Series				8.6	10.0		12.1	13.8
Composite Series		5,2	7.2	8.6	10.0		12.1	13.8

Period of Record: 1973 - 2013

Return Period (years)	1.01	2	5	10	20	25	50	100
Partial Series	4.4	6.7	8.2	9.2		10.6	11.6	12.7
Annual Series	1.9	3.4	7.7	9.3		11.2	12.7	14.2
Composite Series	4.4	6.7	8.2	9.3		11.2	12.7	14.2

ARR Project 5		1.1	2.8	4.3	6.0		8.7	10.8

Table B.3 Flood Frequency Analysis of Peak Flows (m^3/s) in Yarralumla Creek Catchment at Mawson

Period of Record: 1971 - 1992

Return Period (years)	**1.01**	**2**	**5**	**10**	**20**	**25**	**50**	**100**
Partial Series		22	29	34		42	47	53
Annual Series		18	28	35		45	53	61
Composite Series		22	29	34	43	50	63	80

ARR Project 5		3.1	8.0	12.4	17.4		24.9	31.2

The peak flows determined from flood frequency analysis of the gauged flows for the Yarralumla Creek catchment at Mawson are summarised in **Table B.3**. The 2, 5 and 10 yr ARI values in the composite series were based on a LP3 partial series analysis. The 20, 25, 50 and 100 yr ARI peak flows in the composite series were scaled off a graph with the highest (outlier) peak flow of 195 m^3/s recorded in the 1971 flood plotted at an estimated position of 1,000 year ARI.

The peak flows determined from flood frequency analysis of the gauged flows for the Yarralumla Creek catchment at Curtin are summarised in **Table B.4**. The 2, 5 and 10 yr ARI values in the composite series were based on a LP3 partial series analysis. The 20, 25, 50 and 100 yr ARI peak flows in the composite series were scaled off a graph with the highest (outlier) peak flow of 240 m^3/s recorded in the 1971 flood plotted at an estimated position of 350 year ARI.

The peak flows determined from flood frequency analysis of the gauged flows for the Gungahlin rural catchment are summarised in **Table B.5**.

Table B.4 Flood Frequency Analysis of Peak Flows (m^3/s) in Yarralumla Creek Catchment at Curtin

Period of Record: 1970 - 1992

Return Period (years)	**1.01**	**2**	**5**	**10**	**20**	**25**	**50**	**100**
Partial Series		64	83	97		118	135	154
Annual Series		54	76	93		115	135	156
Composite Series		64	83	97	119	125	150	175

ARR Project 5		9.1	23.1	35.9	50.5		72.3	90.5

Table B.5 Flood Frequency Analysis of Peak Flows (m^3/s) in Gungahlin Catchment

Period of Record: 1973 - 1991

Return Period (years)	**1.01**	**2**	**5**	**10**	**20**	**25**	**50**	**100**
Partial Series	0.25	0.90	1.6	2.3		3.3	4.3	5.5
Annual Series	0.03	0.46	1.2	1.9		3.0	4.1	5.4
Composite Series	0.25	0.90	1.6	1.9		3.0	4.1	5.4

ARR Project 5		1.5	3.8	6.0	8.4		12.0	15.0

B.2.2 Sydney Gauged Urban Catchments

The estimated peak flows for the Hewitt catchment are summarised in **Table B.6**. The estimated series is based on the results from catchment simulation as described in **Appendix D.**

Table B.6 Estimated Peak Flows (m^3/s) in Hewitt Catchment

Period of Record: 1993 - 1995

Return Period (years)	**1.01**	**2**	**5**	**10**	**20**	**25**	**50**	**100**
Partial Series								
Annual Series								
Estimated Series	4.5	6.2	10.1	11.8	15.8		18.3	22.8
ARR Project 5		2.6	7.0	11.0	15.6		22.5	28.2

The peak flows determined from flood frequency analysis of the gauged flows for the Powells Creek catchment are summarised in **Table B.7.** At the time of preparation of this Discussion Paper only the annual series results were available. The slope of the annual series flood frequency curve is significantly flatter than the curve for the Parramatta River at Parramatta (refer **Table B.8**).

The peak flows determined from flood frequency analysis of the gauged flows for the Parramatta River catchment at Parramatta are summarised in **Table B.8**. No data was available for partial series over the period 1979 – 2000. The highest peak on record over the 21 years of record was 781 m^3/s during the 1988 flood. It is likely that if the partial series analysis over the same period was included the peak flows for floods less than 10 year ARI would be higher. It is also likely that the 100 yr ARI peak flow could decrease significantly as the length of record increases.

Table B.7 Flood Frequency Analysis of Peak Flows (m^3/s) in the Powells Creek Catchment

Period of Record: 1973 - 1989

Return Period (years)	**1.01**	**2**	**5**	**10**	**20**	**25**	**50**	**100**
Partial Series								
Annual Series	4.4	13.7	23.9	30.1	36.6		46.0	53.8
Composite Series								
ARR Project 5		8.0	21.2	33.5	47.5		68.5	86.1

Table B.8 Flood Frequency Analysis of Peak Flows (m^3/s) in the Parramatta River Catchment at Parramatta

Period of Record: 1979 - 2000

Return Period (years)	**1.01**	**2**	**5**	**10**	**20**	**25**	**50**	**100**
Partial Series								
Annual Series	25	130	293	469	710		1,165	1,647
Composite Series								
ARR Project 5		76	202	318	451		651	818

B.2.3 Melbourne Gauged Urban Catchment

Rainfall and streamflow records spanned 35 years (1977 to 2012) in six-minute intervals at the Kinkora Road Retarding Basin (Gauge Station 229636A). The retarding basin located at the base of the catchment is not included within the catchment boundary and the inflows to the station are not influenced by the retarding basin (Pomeroy et al, 2013). An analysis of the peak flows recorded in the Kinkora Road urban catchment was undertaken using HEC-SSP.

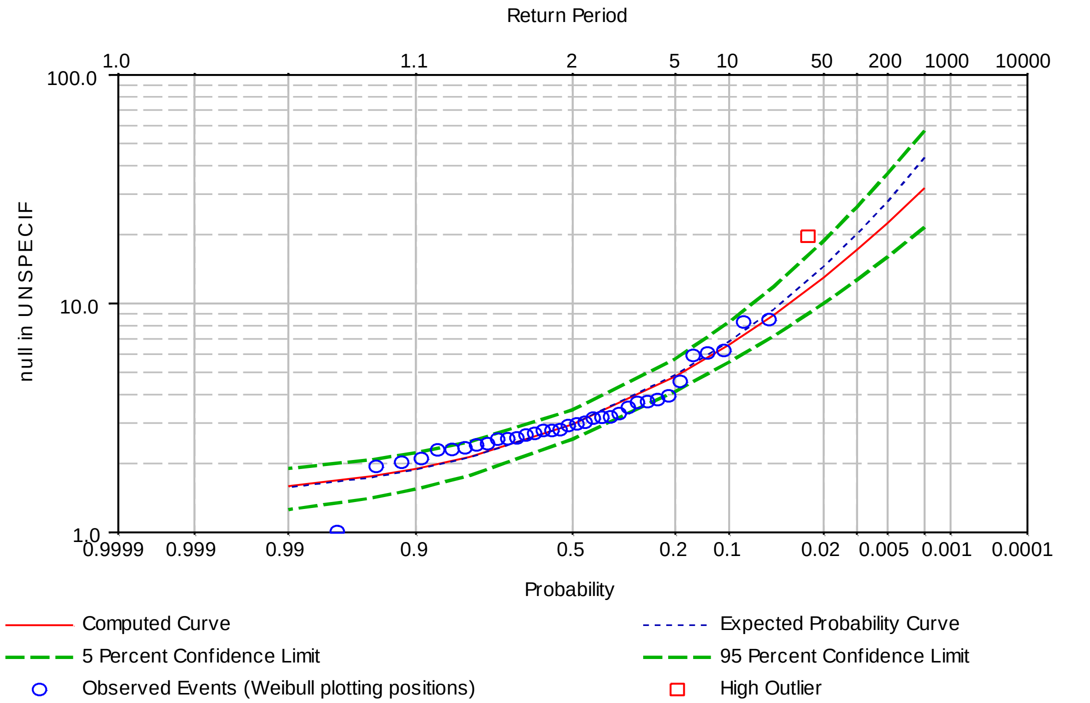


Figure B.1 Flood Frequency Curve for Kinkora Road Catchment including 1997 Outlier

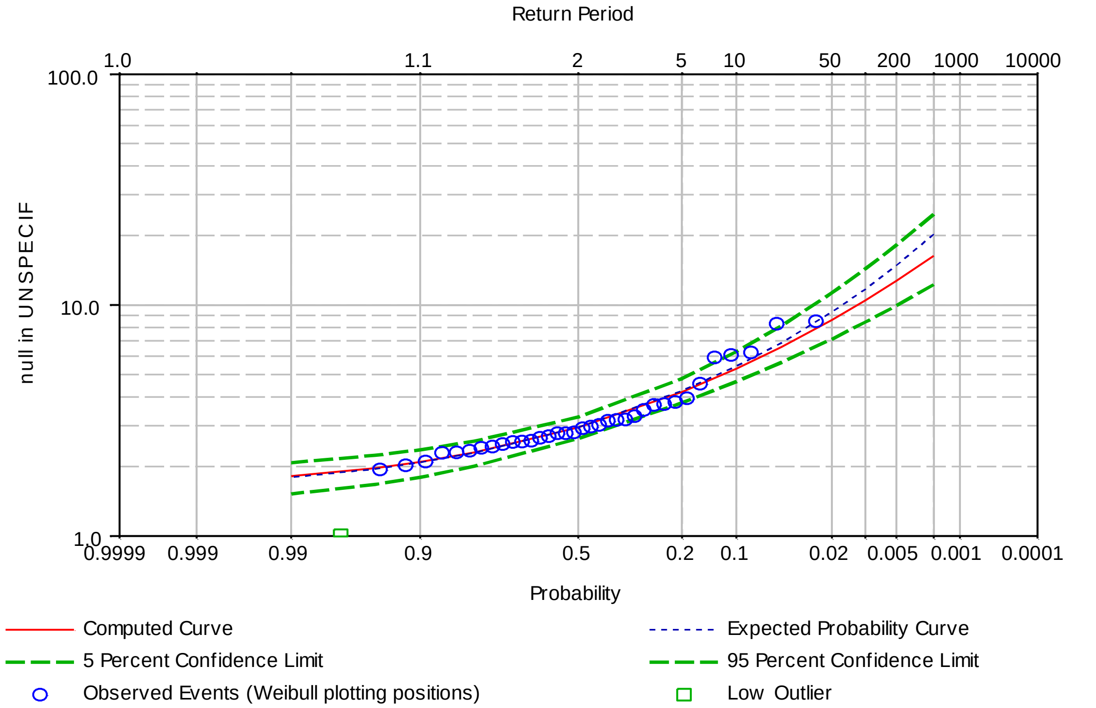


Figure B.2 Flood Frequency Curve for Kinkora Road Catchment with 2.5 m^3/s substituted for 1977 Outlier

The analysis presented in **Figure B.1** highlighted that the peak flow of 19.69 m^3/s recorded in 1977 is an outlier. A review of the gauged flow records found that a peak flow of 19.69 m^3/s was recorded at four separate times during the same event. These peak flows were recorded at 15:16 hours on 27 July 1977 at, at 18:48 hours on 29 July1977, at 00:06 hours on 30 July 1977 and at 04:48 hours on 30 July 1977. A review of the rainfall data for the same period disclosed that there was only low rainfall of several millimetres was recorded. Additionally many data crashes were observed during this event.

The peak flow of 5.92 m^3/s recorded in 1984 was also examined. In this instance recorded flow event aligned with in excess of 20 mm of rainfall immediately prior to the recorded peak flow. It was concluded that this observed rainfall and runoff in 1984 was consistent and that the recorded peak flows in July 1977 were not supported by the rainfall record and are highly suspect.

The peak flows were re-analysed based on substituting a nominal peak flow of 2.5 m^3/s for the 1977 event and gave the flood frequency curve presented in **Figure B.2**. This is the annual series reported in **Table B.9**.

The peak flows determined from flood frequency analysis of the gauged flows for the Kinkora Road catchment are summarised in **Table B.9**. It is likely that if the partial series analysis over the same period was included the peak flows for floods less than 10 year ARI would be higher.

Table B.9 Flood Frequency Analysis of Peak Flows (m^3/s) in the Kinkora Road Catchment

Period of Record: 1977 - 2012

Return Period (years)	**1.01**	**2**	**5**	**10**	**20**	**25**	**50**	**100**
Partial Series								
Annual Series	1.8	2.9	4.2	5.3	6.6		8.6	10.5
Composite Series								
ARR Project 5		2.0	4.5	6.5	8.6		11.5	13.7

B.2.4 Darwin Gauged Urban Catchments

The peak flows determined from flood frequency analysis of the gauged flows for the Moil urban catchment are summarised in **Table B.10.** In this analysis the partial series analysis was approximated by an analysis of monthly peak flows. A peak over threshold style approach was adopted by selecting only monthly peaks within a year that exceeded a certain threshold. This threshold was selected so that there was the same number of peak flows as there were years.

Table B.10 Flood Frequency Analysis of Peak Flows (m^3/s) in the Moil Catchment

Period of Record: 1984 - 2010

Return Period (years)	**1.01**	**2**	**5**	**10**	**20**	**25**	**50**	**100**
Partial Series	4.43	7.1	8.9	10.1				
Annual Series	1.83	5.8	8.4	10.1	11.6		13.6	15.1
Composite Series	4.43	7.1	8.9	10.1	11.6		13.6	15.1
ARR Project 5		4.0	8.3	11.3	14.1		17.4	19.8

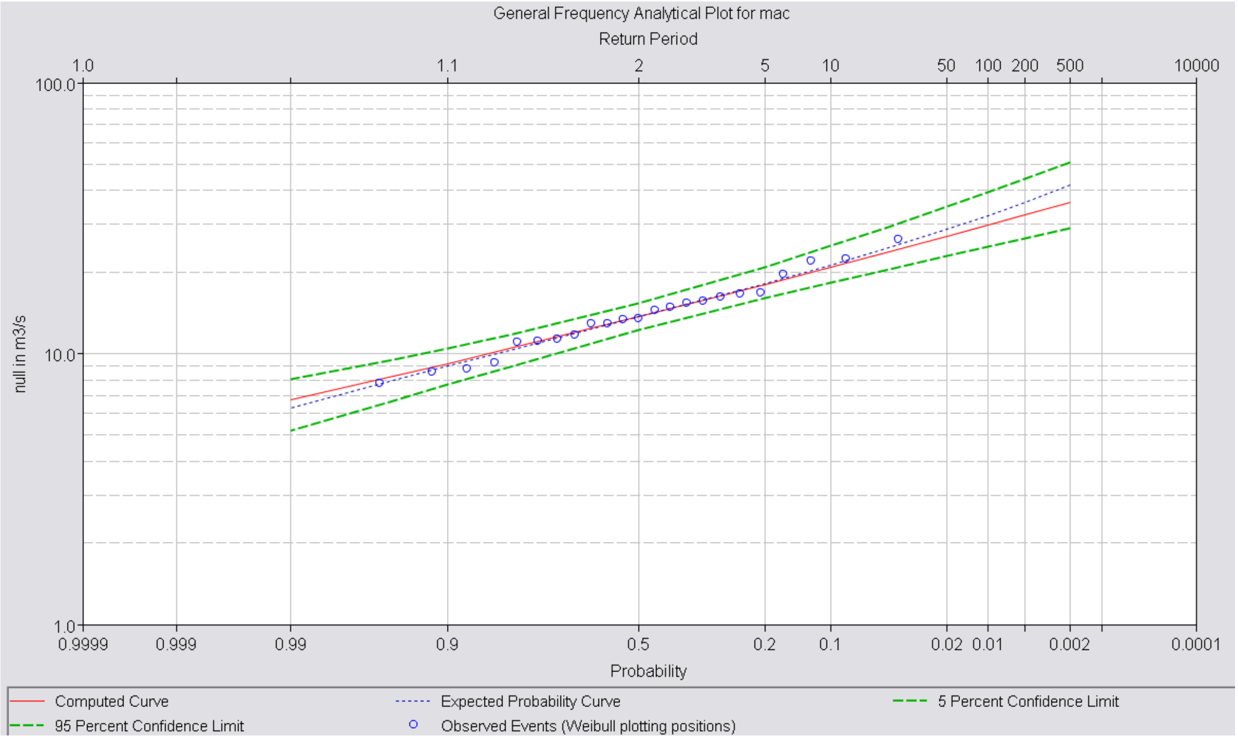


Figure B.3 Flood Frequency Curve for McArthur Park Catchment

In 1990 Ross Knee in his Master's thesis provided a detailed evaluation of the differences between the partial, monthly and annual series analyses with the different theoretical distributions. He concluded, at least for the ACT, that the monthly series was similar to the partial and both different from the annual series for the 1, 2 and 5 year return periods.

An analysis of the peak flows recorded in the McArthur Park urban catchment was undertaken using HEC-SSP. The analysis presented in **Figure B.3**. The peak flows determined from flood frequency analysis of the gauged flows for the McArthur Park urban catchment are summarised in **Table B.11.** It is noted however that these results are problematic due to the presence of a large retarding basin located upstream of the gauging station which can modify the runoff response from a significant proportion of the catchment by infiltrating any overland flows into the grassed base of the basin in frequent events and by reducing peak flows in major events.

Table B.11 Flood Frequency Analysis of Peak Flows (m^3/s) in the McArthur Park Catchment

Period of Record: 1983 - 2005

Return Period (years)	**1.01**	**2**	**5**	**10**	**20**	**25**	**50**	**100**
Partial Series								
Annual Series	6.8	13.7	17.9	20.8	23.5		27.0	29.7
Composite Series								

ARR Project 5								

APPENDIX C

PRELIMINARY PART I STUDY –
CANBERRA, SYDNEY, MELBOURNE, DARWIN

C.1 BACKGROUND

The 1987edition of ARR recommended changes to both the estimation of both time of concentration and runoff coefficient in urban drainage design. 1987 ARR departed from the empirical relationship given in Equation 2. Instead, it recommended the use of the "kinematic wave" equation for overland flow time previously described by Ragan & Duru (1972). The 1987 ARR varies from the 1958 and 1977 editions in its presentation of runoff coefficients for design purposes. This edition presents a:

> "*composite relationship reflecting experience of drainage authorities and evidence from the few gauged urban catchments with suitable lengths of record ..."*

During the consultation period held prior to the release of 1987 ARR a study was carried out in the ACT at the request of the ACT Government to review the possible effects of differences between the urban Rational Method procedures as recommended in 1977 ARR and 1987 ARR. This review is described in a report titled “Drainage Design Practice for Land Development in the ACT. Part I: Rational Formula Procedures”, Willing and Partners (1989) which is attached in **Appendix E.**

This report ultimately recommended a semi-probabilistic based procedure for urban drainage design undertaken using the Rational Method in the ACT. The recommended procedure was based on the outcomes of testing different combinations of the 1977 and 1987 procedures for estimating runoff coefficient and time of concentration for estimating runoff coefficient to estimate flow peak quantiles in two gauged urban catchments. The estimated flow quantiles were then compared with peak flows determined using a flood frequency analysis. It was found that the combination of the procedures for estimating runoff coefficient and time of concentration given in the 1977 ARR best fitted the flood frequency curves from 2 yr ARI to 100 yr ARI.

Since the publication of 1987 ARR a number of water authorities as well as Councils have also published their own recommendations for how the Rational Formula should be applied to urban catchments in their jurisdiction. Typically these guidelines recommend procedures for estimating runoff coefficient and time of concentration which differ from those recommended in the 1987 ARR.

Since 1989 additional data has been collected in the Giralang catchment which has allowed the updating of the 1987 analysis as well as the preliminary testing of the sensitivity of the predicted peak flows to characterising a catchment based on total impervious area (TIA) or effective impervious area (EIA) as assessed in ARR Project 6 Stage 2 - Analysis of Effective Impervious Area & Pilot Study of Losses in Urban Catchments.

C.2 CANBERRA

The Giralang catchment assessment was updated based on the following catchment properties.

Catchment	Year	Total Area* (TA) (ha)	Urban Area^ (UA) (ha)	Total Impervious Area* (TIA) (ha)	Effective Impervious Area# (EIA) (ha)
Giralang	1990	90.74	62.9	19.8	
Giralang	2013	90.98	61.8	28.4	21.0 – 22.7

**Determined using the desktop GIS method*

^The Urban Area is classified as the total developed area excluding large open space

#EIA varies based on rainfall gauge adopted for analysis in period 1973 - 2012

Table C.1
Adopted Design IFD parameters for the Giralang Catchment

Parameter	Value
2 Year ARI 1 hour Intensity	22.0 mm/hr
2 Year ARI 12 hour Intensity	4.3 mm/hr
2 Year ARI 72 hour Intensity	1.14 mm/hr
50 Year ARI 1 hour Intensity	43.0 mm/hr
50 Year ARI 12 hour Intensity	8.0 mm/hr
50 Year ARI 72 hour Intensity	2.25 mm/hr
Location Skew	0.24
F2	4.28
F50	15.55

The adopted IFD parameter values are given in **Table C.1**.

In the case of assessment undertaken using the 1977 ARR runoff coefficients the adopted runoff coefficient curve number was No. 4 (refer Figure 2.3)

The overland flow time was calculated using the following equation (S.I. units):

$$t_o = 107 \frac{n\ L^{0.333}}{S^{0.2}} \qquad \text{(C.1)}$$

where to = overland flow travel time (minutes)
L = flow path length (m)
n = Horton's roughness value for the surface
S = slope of surface (%)

The adopted parameter values for assessment purposes were:
L = 50 m
n = 0.015 for impervious surfaces
n = 0.04 for pervious surfaces
S = 4.5 %

Giving to = 15.7 mins for pervious surfaces including an estimated 4 mins travel time
to = 8.4 mins for impervious surfaces including an estimated 4 mins travel time

In the case of the 1987 ARR procedures, runoff coefficients were estimated using Equations 4, 5 and 6 (refer Section 2).

1987 ARR recommended the use of the "kinematic wave" equation for overland flow time previously described by Ragan & Duru (1972). This equation is as follows:

$$t_o = \frac{6.94\ (L\ n^*)^{0.6}}{I^{0.4}\ S^{0.3}} \qquad (C.2)$$

where t_o = overland flow travel time (minutes)
L = flow path length (m)
n^* = surface roughness
I = rainfall intensity (mm/h)
S = slope (m/m)

The adopted parameter values for assessment purposes were:
L = 50 m
n^* = 0.015 for impervious surfaces
n^* = 0.3 for pervious surfaces
S = 4.5 %

Giving to = 27 mins for pervious surfaces and 17 mins for impervious surfaces including an estimated 4 mins travel time when assessing the total urban area; and
to = 6.7 mins to 5.5 mins for impervious surfaces depending on ARI including an estimated 4 mins travel time when assessing the EIA only

Table C.2 Estimated Peak Flows (m^3/s) in the Giralang Catchment

Return Period (years)	**1.01**	**2**	**5**	**10**	**20**	**25**	**50**	**100**
Partial Series	4.4	6.7	8.2	9.2		10.6	11.6	12.7
Annual Series	1.9	3.4	7.7	9.3		11.2	12.7	14.2
Composite Series	4.4	6.7	8.2	9.3		11.2	12.7	14.2
1977 ARR Procedures								
TU + TIA – 1990 Values	3.9	5.5	7.6	9.0	10.8		13.4	15.5
TU + TIA – 2013 Values	4.2	5.7	7.9	9.2	11.1		13.7	15.8
TU + EIA – 2013 Values	4.0	5.5	7.7	9.1	11.0		13.6	15.6
EIA only – 2013 Value	*2.5*	*3.3*	*4.5*	*5.2*	*6.2*		*7.5*	*8.6*
1987 ARR Procedures								
TU + TIA – 1990 Values	1.5	2.3	3.6	4.4	5.7		7.7	9.1
TU + EIA – 2013 Values	1.6	2.4	3.8	4.7	6.0		8.1	9.6
EIA only – 2013 Value	2.78	3.7	5.1	6.0	7.2		8.9	10.2
ARR Project 5		1.1	2.8	4.3	6.0		8.7	10.8

The peak flows reported in **Table C.2** were estimated using a single node **xprathgl** model of the Giralang catchment.

It was concluded from a comparison of the various results that:

- The 1977 ARR procedures give peak flows which match the peak flows adopted for the composite series based on flood frequency analysis (FFA) except for flows based on EIA only;
- The 1987 procedures give peak flows lower than the peak flows adopted for the composite series based on flood frequency analysis with the estimated 100 yr ARI peak flow comparable to the 10 yr ARI peak flow from the FFA;
- For 10 yr ARI and above the 1987 ARR procedures give similar peak flows to the ARR Project 5 procedures for rural catchments.

C.3 SYDNEY

The Hewitt catchment assessment was based on the following catchment properties.

Catchment	Year	Total Area (TA) (ha)	Urban Area (UA) (ha)	Total Impervious Area* (TIA) (ha)	Effective Impervious Area (EIA) (ha)
Hewitt	2013	62.0	62.0	19.8	

Table C.3
Adopted Design IFD parameters for the Powells Creek Catchment

Parameter	**Value**
2 Year ARI 1 hour Intensity	29.69 mm/hr
2 Year ARI 12 hour Intensity	6.53 mm/hr
2 Year ARI 72 hour Intensity	1.89 mm/hr
50 Year ARI 1 hour Intensity	59.06 mm/hr
50 Year ARI 12 hour Intensity	12.79 mm/hr
50 Year ARI 72 hour Intensity	4.32 mm/hr
Location Skew	0.02
F2	4.3
F50	15.8

The adopted IFD parameter values are given in **Table C.3**.

In the case of assessment undertaken using the 1977 ARR runoff coefficients the adopted runoff coefficient curve number was No. 4 (refer Figure 2.3).

For 1977 ARR procedures the adopted parameter values for assessment of time of concentration were:

L = 50 m
n = 0.015 for impervious surfaces
n = 0.04 for pervious surfaces
S = 2.5 %

Giving t_o = 18.0 mins for pervious surfaces including an estimated 4 mins travel time

t_o = 8.4 mins for impervious surfaces including an estimated 4 mins travel time

For 1987 ARR procedures the adopted parameter values for assessment of time of concentration were:

L = 50 m

n* = 0.015 for impervious surfaces

n* = 0.3 for pervious surfaces

S = 2.5%

Giving t_o = 30 mins to 20 mins for impervious surfaces depending on ARI including an estimated 4 mins travel time when assessing the total urban area; and

The peak flows reported in **Table C.4** were estimated using a single node **xprathgl** model of the Hewitt catchment. It was concluded from a comparison of the various results that:

- The 1977 ARR procedures give peak flows which are slightly lower than the peak flows adopted for the composite series with the estimated 100 yr ARI peak flow being comparable to the 50 yr ARI peak flow from the FFA.
- The 1987 procedures give peak flows lower than the peak flows adopted for the composite series with the estimated 100 yr ARI peak flow being comparable to the 10 yr ARI peak flow from the FFA;

Table C.4 Estimated Peak Flows (m^3/s) in the Hewitt Catchment

Return Period (years)	**1.01**	**2**	**5**	**10**	**20**	**25**	**50**	**100**
Partial Series								
Annual Series								
Composite Series	4.5	6.2	10.1	11.8	15.8		18.3	22.8
1977 ARR Procedures								
TA + TIA	5.1	6.9	9.4	10.8	12.7		15.2	17.2
DCIA Only*	5.0	6.8	9.3	10.7	12.6		15.1	17.1
**DCIA only assumed to be 17.8 ha*								
1987 ARR Procedures								
TA + TIA	2.4	3.5	5.4	6.6	8.3		10.9	12.6
DCIA Only*	3.4	2.1	5.3	6.4	8.0		10.5	12.2
**DCIA only assumed to be 17.8 ha*								
ARR Project 5		2.6	7.0	11.0	15.6		22.5	28.2

The Powells Creek catchment assessment was based on the following catchment properties.

Catchment	Year	Total Area* (TA) (ha)	Urban Area^ (UA) (ha)	Total Impervious Area* (TIA) (ha)	Effective Impervious Area# (EIA) (ha)
Powells Creek	1990	234	223.4	117.0	
Powells Creek	2013	231.9	223.4	151.7	90.6 – 95.1

**Determined using the desktop GIS method*

^The Urban Area is classified as the total developed area excluding large open space

#EIA based on analysis in period 1973 - 1989

Table C.5
Adopted Design IFD parameters for the Powells Creek Catchment

Parameter	**Value**
2 Year ARI 1 hour Intensity	34.45 mm/hr
2 Year ARI 12 hour Intensity	7.31 mm/hr
2 Year ARI 72 hour Intensity	2.41 mm/hr
50 Year ARI 1 hour Intensity	65.94 mm/hr
50 Year ARI 12 hour Intensity	15.53 mm/hr
50 Year ARI 72 hour Intensity	5.04 mm/hr
Location Skew	0.0
F2	4.3
F50	18.84

The adopted IFD parameter values are given in **Table C.5**.

In the case of assessment undertaken using the 1977 ARR runoff coefficients the adopted runoff coefficient curve number was No. 5 (refer Figure 2.3)

For 1977 ARR procedures the adopted parameter values for assessment of time of concentration were:

	L	=	50 m
	n	=	0.015 for impervious surfaces
	n	=	0.04 for pervious surfaces
	S	=	1 %
Giving	to	=	22.7 mins for pervious surfaces including an estimated 7 mins travel time
	to	=	12.9 mins for impervious surfaces including an estimated 7 mins travel time

For 1987 ARR procedures the adopted parameter values for assessment of time of concentration were:

	L	=	50 m
	n*	=	0.015 for impervious surfaces
	n*	=	0.3 for pervious surfaces
	S	=	1 %
Giving	to	=	40 mins for pervious surfaces and 27 mins for impervious surfaces including an estimated 7 mins travel time when assessing the total urban area; and
	to	=	11 mins to 9 mins for impervious surfaces depending on ARI including an estimated 7 mins travel time when assessing the EIA only.

The peak flows reported in **Table C.6** were estimated using a single node **xprathgl** model of the Powells Creek catchment.

Table C.6 Estimated Peak Flows (m^3/s) in the Powells Creek Catchment

Return Period (years)	1.01	2	5	10	20	25	50	100
Partial Series								
Annual Series	4.4	13.7	23.9	30.1	36.6		46.0	53.8
Composite Series								
1977 ARR Procedures								
TA + TIA – 1990 Values	19.9	26.5	35.0	39.5	45.9		54.2	60.6
TU + DCIA Road Only* – 2013 Values	15.7	21.7	29.5	33.7	39.7		47.6	53.6
**DCIA Road only estimated to be 42 ha*								
TA + EIA#	18.9	25.4	33.7	38.2	44.5		52.7	59.0
#EIA assumed to be 100 ha								
EIA only #	14.9	19.0	24.3	27.0	31.0		36.0	39.8
#EIA assumed to be 100 ha								
1987 ARR Procedures								
TA + TIA – 1990 Values	11.6	16.7	25.2	30.3	37.5		48.1	55.0
TA + EIA#	11.0	15.9	23.9	28.7	35.5		45.7	52.4
#EIA assumed to be 100 ha								
EIA only #	16.1	21.0	27.1	30.4	35.0		41.0	45.5
#EIA assumed to be 100 ha								
ARR Project 5		8.0	21.2	33.5	47.5		68.5	86.1

C.3 MELBOURNE

Melbourne Water provides guidelines on the application of the Rational Method in urban catchments as follows.

The Rational Method is generally used to calculate design peak flow rates throughout the pipeline drainage system, provided the drainage catchment is less than 400 hectares. The method does not allow for flood storage effects. Therefore, when there are or will be retarding basins in the system, suitable adjustments must be made for the basin outflows, or an alternative method that provides for flood storage effects must be used.

The Rational Method is described in Book 8 of Australian Rainfall and Runoff. ……

The following guidelines are provided for use of the Rational Method, including values that Melbourne Water requires to be used:

1. *The downstream design peak flow rate should not be less than the upstream flow rate for a piped system*
2. *Partial area effects should be considered in the design (refer Australian Rainfall and Runoff Book 8 for guidance)*
3. *The applicable average recurrence interval, runoff coefficient, area of catchment and design average rainfall intensity will be determined as shown below.*

Table 1 below presents a range of coefficients to be applied to various land use. The values presented are slightly higher than the values that would be obtained by following the method prescribed by ARR. They contain adjustments to suit Melbourne's conditions and must be used in preference to the ARR values.

Table 1 Runoff Coefficients

Land Use	*C (5 Year ARI)*	*C (100 Year ARI)*
Major open space	*0.20*	*0.30*
Residential (avg lot size):		
4000 m2	*0.30*	*0.40*
750 m2	*0.40*	*0.50*
500 m2	*0.50*	*0.65*
350 m2	*0.60*	*0.75*
< 350 m2	*0.70 to 0.90*	*0.9*
Major road reserves	*0.50 to 0.80*	*0.65 to 0.9*
Commercial/industrial	*0.70 to 0.90*	*0.9*

If different ARIs are required, and for situations in which there are a range in values in the table, or where the proposed land use is different to that prescribed, the fraction impervious must be estimated and taken into consideration. In such instances ***the method prescribed by ARR Book 8 should be followed****.*

The time of concentration at a particular location is generally the time required for runoff to travel by the longest available flowpath to that location.

In many cases however a "partial area" affect occurs through the lower part of the catchment, where flows are higher than those calculated for the entire catchment, because the time of concentration is lower and the design rainfall intensity is higher.

The method prescribed by 1998 ARR Book 8 is exactly the same as the method detailed in Chapter 14 of 1987 ARR.

The Kinkora Road catchment assessment was based on the following catchment properties.

Catchment	Year	Total Area* (TA) (ha)	Urban Area^ (UA) (ha)	Total Impervious Area* (TIA) (ha)	Effective Impervious Area# (EIA) (ha)
Kinkora Road		202.1	184.2	121.9	72.3 ha

**Determined using the desktop GIS method*

^The Urban Area is classified as the total developed area excluding large open space

#EIA based on analysis of period 1977 - 2012

Table C.7
Adopted Design IFD parameters for the Kinkora Road Catchment

Parameter	**Value**
2 Year ARI 1 hour Intensity	19.12 mm/hr
2 Year ARI 12 hour Intensity	4.22 mm/hr
2 Year ARI 72 hour Intensity	1.23 mm/hr
50 Year ARI 1 hour Intensity	37.42 mm/hr
50 Year ARI 12 hour Intensity	7.25 mm/hr
50 Year ARI 72 hour Intensity	2.30 mm/hr
Location Skew	0.36
F2	4.28
F50	14.97

The adopted IFD parameter values are given in **Table C.7**.

In the case of assessment undertaken using the 1977 ARR runoff coefficients the adopted runoff coefficient curve number was No. 4 (refer Figure 2.3).

For 1977 ARR procedures the adopted parameter values for assessment of time of concentration were:

L = 50 m
n = 0.015 for impervious surfaces
n = 0.04 for pervious surfaces
S = 1.8 %

Giving to = 21.0 mins for pervious surfaces including an estimated 7 mins travel time
to = 12.3 mins for impervious surfaces including an estimated 7 mins travel time

For 1987 ARR procedures the adopted parameter values for assessment of time of concentration were:

L	=	50 m
n*	=	0.015 for impervious surfaces
n*	=	0.3 for pervious surfaces
S	=	1.8 %

Giving to = 43 mins for pervious surfaces and 26 mins for impervious surfaces in the 100 yr ARI event including an estimated 7 mins travel time when assessing the total urban area; and

to = 11 mins to 9 mins for impervious surfaces depending on ARI including an estimated 7 mins travel time when assessing the EIA only.

The peak flows reported in **Table C.8** were estimated using a single node **xprathgl** model of the Kinkora Road catchment. The annual series peak flows were based on substituting a nominal peak flow of 2.5 m^3/s for the 1977 event.

Table C.8 Estimated Peak Flows (m^3/s) in the Kinkora Road Catchment

Return Period (years)	**1.01**	**2**	**5**	**10**	**20**	**25**	**50**	**100**
Partial Series								
Annual Series	1.8	2.9	4.2	5.3	6.6		8.6	10.5
Composite Series								

1977 ARR Procedures

TA + TIA	9.8	13.3	18.3	21.5	26.0		32.3	37.6

EIA only[#]	5.5	7.2	9.8	11.4	13.7		17.0	19.6

[#]EIA assumed to be 66 ha from Pomeroy et al, 2013

EIA only[#]	2.7	3.6	4.9	5.7	6.8		8.5	9.8*

[#]EIA assumed to be 66 ha from Pomeroy et al, 2013

**FFA results matched by reducing C value by 50% ie. to 0.45*

EIA only [#]	2.9	3.8	5.0	5.8	6.9		8.3	9.6^

[#]EIA assumed to be 100 ha

^ FFA results matched by increasing time of concentration from 12.3 mins to 42.3 mins. C value = 0.9

1987 ARR Procedures

TIA only	4.6	6.8	10.8	13.6	17.6		24.0	28.7

EIA only[#]	5.7	7.7	10.7	12.7	15.4		19.3	22.6

[#]EIA assumed to be 66 ha from Pomeroy et al, 2013

EIA only	5.2	6.8	9.2	10.7	12.8		15.8	18.3^^

^^Adopted Melbourne Water minimum time to entry of 7 mins giving to = 13 mins

ARR Project 5		2.0	4.5	6.5	8.6		11.5	13.7

C.3 DARWIN

The Moil catchment assessment was based on the following catchment properties.

Catchment	Year	Total Area (TA) (ha)	Urban Area (UA) (ha)	Total Impervious Area* (TIA) (ha)	Effective Impervious Area (EIA) (ha)
Moil	2013	40.0	40.0	12.4	*9.08^*

** Assumed to have same imperviousness as Giralang ie. 31%*

^ EIA assumed equal to (Road + Paths + 50% roof area only because a roof typically does not have a gutter and instead sheds runoff onto the ground

Table C.9
Adopted Design IFD parameters for the Moil Catchment

Parameter	**Value**
2 Year ARI 1 hour Intensity	63.0 mm/hr
2 Year ARI 12 hour Intensity	9.80 mm/hr
2 Year ARI 72 hour Intensity	3.00 mm/hr
50 Year ARI 1 hour Intensity	100.0 mm/hr
50 Year ARI 12 hour Intensity	16.0 mm/hr
50 Year ARI 72 hour Intensity	6.0 mm/hr
Location Skew	0.37
F2	4.37
F50	18.5

The adopted IFD parameter values are given in **Table C.9**.

In the case of assessment undertaken using the 1977 ARR runoff coefficients the adopted runoff coefficient curve number was No. 6 (refer Figure 2.3). Curve No. 4 was tested but it was found the Curve No. 4 gave a better fit to the FFA.

For 1977 ARR procedures the adopted parameter values for assessment of time of concentration were:

L = 50 m
n = 0.015 for impervious surfaces
n = 0.04 for pervious surfaces
S = 1 %

Giving to = 19.8 mins for pervious surfaces including an estimated 4 mins travel time
to = 9.9 mins for impervious surfaces including an estimated 4 mins travel time

For 1987 ARR procedures the adopted parameter values for assessment of time of concentration were:

L = 50 m
n* = 0.015 for impervious surfaces
n* = 0.3 for pervious surfaces
S = 1%

Giving to = 28.5 mins to 19.9 mins for impervious surfaces depending on ARI including an estimated 4 mins travel time when assessing the total urban area; and

The peak flows reported in **Table C.10** were estimated using a single node **xprathgl** model of the Moil catchment. It was concluded from a comparison of the various results that:

- The 1977 ARR procedures give peak flows which are slightly higher than the peak flows adopted for the composite series (based on the adoption of Curve 6 for runoff coefficients);
- The 1987 procedures give peak flows higher than the peak flows adopted for the composite series for events greater than a 10 yr ARI event.

Table C.10 Estimated Peak Flows (m^3/s) in the Moil Catchment

Return Period (years)	**1.01**	**2**	**5**	**10**	**20**	**25**	**50**	**100**
Partial Series	4.4	7.1	8.9	10.1				
Annual Series	1.8	5.8	8.4	10.1	11.6		13.6	15.1
Composite Series	4.4	7.1	8.9	10.1	11.6		13.6	15.1
1977 ARR Procedures								
TA + TIA	5.9	7.7	9.6	10.6	12.2		14.3	16.0
TA + EIA	5.5	7.3	9.1	10.1	11.7		13.8	15.5
1987 ARR Procedures								
TA + TIA	4.9	6.9	9.7	11.3	13.8		17.8	20.6
TA + EIA	4.8	6.8	9.5	11.1	13.5		17.4	20.3
ARR Project 5		4.0	8.3	11.3	14.1		17.4	19.8

The McArthur Park catchment assessment was based on the following catchment properties.

Catchment	Year	Total Area* (TA) (ha)	Urban Area^ (UA) (ha)	Total Impervious Area* (TIA) (ha)	Effective Impervious Area# (EIA) (ha)
McArthur Park	1990			23.0	
McArthur Park	2013	143.7	120.2	53.7	35.3

**Determined using the desktop GIS method*

^The Urban Area is classified as the total developed area excluding large open space

#EIA based on analysis in period 1983 to 2004

Table C.11
Adopted Design IFD parameters for the McArthur Park Catchment

Parameter	Value
2 Year ARI 1 hour Intensity	63.0 mm/hr
2 Year ARI 12 hour Intensity	9.80 mm/hr
2 Year ARI 72 hour Intensity	3.00 mm/hr
50 Year ARI 1 hour Intensity	100.0 mm/hr
50 Year ARI 12 hour Intensity	16.0 mm/hr
50 Year ARI 72 hour Intensity	6.0 mm/hr
Location Skew	0.37
F2	4.37
F50	18.5

The adopted IFD parameter values are given in **Table C.11**.

In the case of assessment undertaken using the 1977 ARR runoff coefficients the adopted runoff coefficient curve number was No. 4 (refer Figure 2.3)

For 1977 ARR procedures the adopted parameter values for assessment of time of concentration were:

	L	=	50 m
	n	=	0.015 for impervious surfaces
	n	=	0.04 for pervious surfaces
	S	=	1.5%
Giving	to	=	21.5 mins for pervious surfaces including an estimated 7 mins travel time
	to	=	12.5 mins for pervious surfaces including an estimated 7 mins travel time

For 1987 ARR procedures the adopted parameter values for assessment of time of concentration were:

	L	=	50 m
	n*	=	0.015 for impervious surfaces
	n*	=	0.3 for pervious surfaces
	S	=	1.5 %
Giving	to	=	28 mins for pervious surfaces and 21 mins for impervious surfaces including an estimated 7 mins travel time when assessing the total urban area; and
	to	=	9.6 mins to 8.7 mins for impervious surfaces depending on ARI including an estimated 7 mins travel time when assessing the EIA only.

The peak flows reported in **Table C.12** were estimated using a single node **xprathgl** model of the McArthur Park catchment.

Table C.12 Estimated Peak Flows (m^3/s) in the McArthur Park Catchment

Return Period (years)	1.01	2	5	10	20	25	50	100
Partial Series								
Annual Series	6.8	13.7	17.9	20.8	23.5		27.0	29.7
Composite Series								
1977 ARR Procedures								
TU + TIA – 1990 Values	20.1	26.3	32.3	35.7	40.9		47.9	53.4
TU + TIA – 2013 Values	21.7	28.0	34.1	37.5	42.8		49.9	55.5
EIA# only	9.6	12.2	14.7	16.1	18.2		21.1	23.3
#EIA assumed to be 36.2 ha								
1987 ARR Procedures								
TU + TIA – 1990 Values	14.4	20.2	28.0	32.7	39.7		51.1	59.3
TU + TIA – 2013 Values	15.4	21.6	30.0	35.0	42.5		54.1	62.1
EIA only #	10.8	13.7	14.7	18.3	20.9		24.3	27.0
#EIA assumed to be 36.2 ha								
ARR Project 5								

C.5 DISCUSSION

C.5.1 Canberra and Sydney

It was concluded from a comparison of the various Giralang catchment results that:

- The 1977 ARR procedures give peak flows which match the peak flows adopted for the composite series based on flood frequency analysis (FFA) except for flows based on EIA only;
- The 1987 procedures give peak flows lower than the peak flows adopted for the composite series based on flood frequency analysis with the estimated 100 yr ARI peak flow comparable to the 10 yr ARI peak flow from the FFA;
- For 10 yr ARI and above the 1987 ARR procedures give similar peak flows to the ARR Project 5 procedures for rural catchments.

It was concluded from a comparison of the various Hewitt catchment results that:

- The 1977 ARR procedures give peak flows which are slightly lower than the peak flows adopted for the composite series with the estimated 100 yr ARI peak flow being comparable to the 50 yr ARI peak flow from the FFA.

- The 1987 procedures give peak flows lower than the peak flows adopted for the composite series with the estimated 100 yr ARI peak flow being comparable to the 10 yr ARI peak flow from the FFA.

It was concluded from a comparison of the various Powells Creek catchment results that:

- The 1977 ARR procedures give peak flows which are higher than the peak flows obtained from an annual series analysis of gauged flows with the peak flows estimated for frequent runoff up to 10 yr ARI being significantly higher;
- One approach to improve agreement would be to test Curve No. 6 in comparison with the adopted Curve No. 5;
- The 1987 procedures give peak flows slightly higher than the peak flows adopted for the annual series but in good agreement.

It was concluded in the 1989 Part I study in the ACT that the runoff coefficients and the time of concentration are paired ie. any procedure to estimate each in a gauged urban catchments needs to be undertaken simultaneously not independently as occurred when preparing the 1987 ARR. This conclusion is further supported by these preliminary analyses.

C.5.2 Melbourne

Based on the results presented in Table C.8 it is apparent all Rational Method peak flows are significantly higher than corresponding flood frequency peak flow estimates except where agreement is forced by adjusting the runoff coefficient or the time of concentration.

Based on the work of Pomeroy et al (2013) the Kinkora Road urban catchment shares many characteristics with the Powells Creek urban catchment in Sydney. The peak flows in both catchments appear to derive mostly from the EIA only. This may well encompass only the roads themselves plus very limited amounts of in block hard surfaces.

This also highlights the potential problems of adopting a limited number of long term gauged urban catchments as representative of all urban catchments. The Kinkora Road and Powells Creek catchments are probably representative of many older suburbs which were first developed in the 1950s or 1960s. They are however not representative of newer catchments with high degrees of directly connected impervious areas including the Hewitt catchment in Sydney and the Giralang catchment in ACT.

C.5.3 Darwin

It was concluded from a comparison of the various Moil results that:

- The 1977 ARR procedures give peak flows which are slightly higher than the peak flows adopted for the composite series (based on the adoption of Curve 6 for runoff coefficients);
- The 1987 procedures give peak flows higher than the peak flows adopted for the composite series for events greater than a 10 yr ARI event.

It was found that most predicted peak flows estimated using the 1977 ARR or 1987 ARR procedures gave peak flows considerably higher than the peak flows estimated using FFA of the gauged flows from the McArthur Park.

While one approach to improve agreement would be to test Curve No. 5 in comparison with the adopted Curve No. 4 it was noted however that these FFA results are problematic due to the presence of a large retarding basin located upstream of the gauging station which can modify the runoff response from a significant proportion of the catchment by infiltrating any overland flows into the grassed base of the basin in frequent events and by reducing peak flows in major events.

APPENDIX D

PRELIMINARY PART II STUDY – CANBERRA & SYDNEY

D.1 BACKGROUND

In 1993 a study was undertaken in Canberra to provide practice guidelines when utilising hydrograph based estimation procedures in urban drainage projects in the ACT. The work followed on from the earlier Part I study. It is described in a report titled “Drainage Design Practice Part II”, Willing and Partners (1993) which is attached in **Appendix F.**

The goal of the Part II study was to test several currently available rainfall/runoff computer programs including RAFTS, RORB and IlSAX on Canberra's gauged urban catchments.

In particular, the objectives were to determine appropriate:

(iv) design rainfall loss rate estimation parameters applicable to individual programs,
(v) surface runoff routing parameters for pervious and impervious areas specific to each program tested, and
(vi) design storm event modelling procedures specific to each program tested.

The assessment of RAFTS model parameters was summarised as follows (Willing & Partners, 1993):

The RAFTS analysis in this study involved the use of two approaches to rainfall loss estimation. They were the initial/continuing loss approach and the infiltration/water balance procedure approach which utilizes the Australian Representative Basin Program (ARBM).

The Giralang catchment analysis results were based on a 41 node RAFTS-XP network which is equivalent to an average sub-catchment size of approximately 2.2 hectares. The Mawson catchment analysis results were based on a 180 node network which is equivalent to an average sub-catchment size of approximately 2,3 hectares.

The initial/continuing loss model analysis failed to produce a single set of loss rates which were able to model the full range of flood frequency curve flows on the catchments modelled. The results indicated the peak flows are sensitive to the losses adopted.

Analysis was carried out on the Giralang and Mawson catchments to determine the effect of the level of sub-catchment discretisation adopted. The conclusion to be drawn from this analysis was that the modelled peak flow increases with increasing catchment discretisation. Alternatively the modelled peak flows decrease with decreasing catchment discretisation.

The RAFTS ARBM loss model approach to calibration was to vary the initial catchment wetness conditions until a volume calibration was achieved against the targeted flood hydrograph.

Following this a further calibration against the targeted peak flow was carried out by varying the catchment surface roughness parameters.

The results of the RAFTS ARBM modelling produced a high level of calibration achievement particular on the Giralang catchment which is well gauged. The design storm event modelling against the catchment flood frequency curves also revealed that single set of model parameter values was able to reasonably predict a full range of the ARI flood frequency flows. The results of the design storm analysis should be viewed with some caution due to the uncertainty which exists regarding the catchment flood frequency curves, particularly at the higher magnitude ARI events.

The ACT Department of Urban Services publication titled "Design Standards for Urban Infrastructure, 1 Stormwater" subsequently specified recommended parameters and procedures for hydrograph estimation to be used instead of values and procedures recommended in program documentation and related reports. The guidance was as follows.

Rainfall Loss Rates

The XP-RAFTS program offers a choice between two approaches to rainfall loss estimation. They are the initial/continuing loss model and the infiltration/water balance procedure which utilises the Australian Representative Basins Model (ARBM). The use of the ARBM loss model shall be used in preference to the initial/continuing loss model due to the ability of ARBM to model a range of ARI events with a single set of model parameters.

The values for the ARBM loss model to be adopted are as follows.

Parameter	**Adopted Value**	**Initial Value**
Storage Capacities		
Impervious (IMP)	0.50	0.0
Interception (ISC)	1.00	0.0
Depression (DSC)	1.00	0.0
Upper soil (USC)	25.00	20.00
Lower soil (LSC)	50.00	40.00
Infiltration		
Dry soil sorptivity (SO)	3.00	
Hydraulic conductivity (K0)	0.33	
Lower soil drainage factor (LDF)	0.05	
Groundwater recession;		
constant rate (KG)	0.94	
variable rate (GN)	1.00	
Evapo-Transpiration		
Proportion of rainfall intercepted by vegetation (IAR)	0.70	
Max potential evapo-transpiration;		
upper soil (UH)	10.00	
lower soil (LH)	10.00	
Proportion of evapo-transpiration from upper soil zone (ER)	0.70	
Ratio of potential evaporation to A class pan (ECOR)	0.90	

Surface Runoff Routing

The following surface runoff routing parameters shall be adopted.

Parameter	**Value**
Impervious surface roughness	0.015
Pervious surface roughness	0.040
Non-linearity coefficient (default)	0.285

Since the 1993 study was completed an addition of 20+ years of rainfall and runoff data collected including 3 years of data collected on micro catchments embedded within the Giralang urban catchment. Data from the micro catchments was collected and reported in the PhD thesis submitted by Goyen in 2000. The research reported by Goyen, 2000 further examined the processes within the Giralang catchment as well as the Hewitt urban catchment located near Penrith in Sydney.

A potential problem with Part II study was the recommended initial (and high) values for moisture stores. An embedded approach has been assessed to establish if it performs better than fixed initial values in a vertical water balance loss model.

D.2 GIRALANG

The details of the Giralang urban catchment and its gauging stations are provided by Goyen (2000). An overview of the Giralang catchment is given in **Figure D.1** while details on the paired micro catchments are given in **Figures D.2** and **D.3.**

As part of the preparation of this position paper both the ACT Part II study carried out in 1993 as well as the research carried out by Goyen (2000) was revisited.

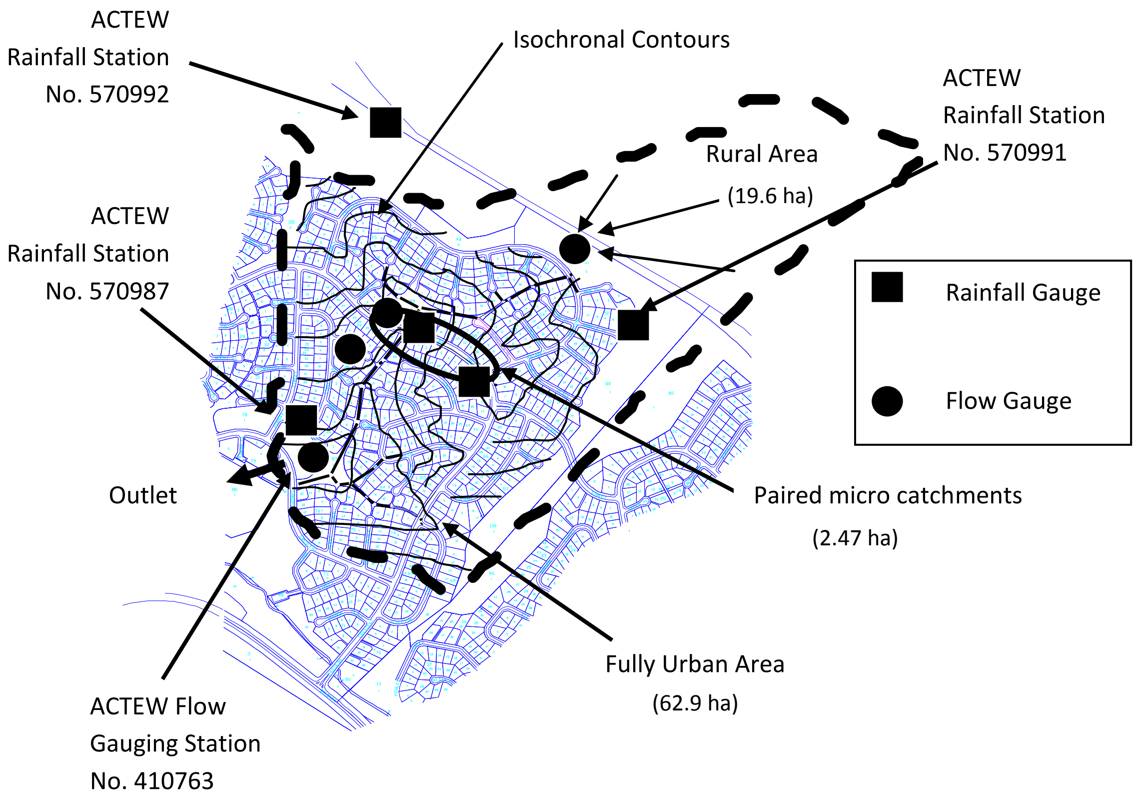


Figure D.1 Giralang Gauged Urban Catchment

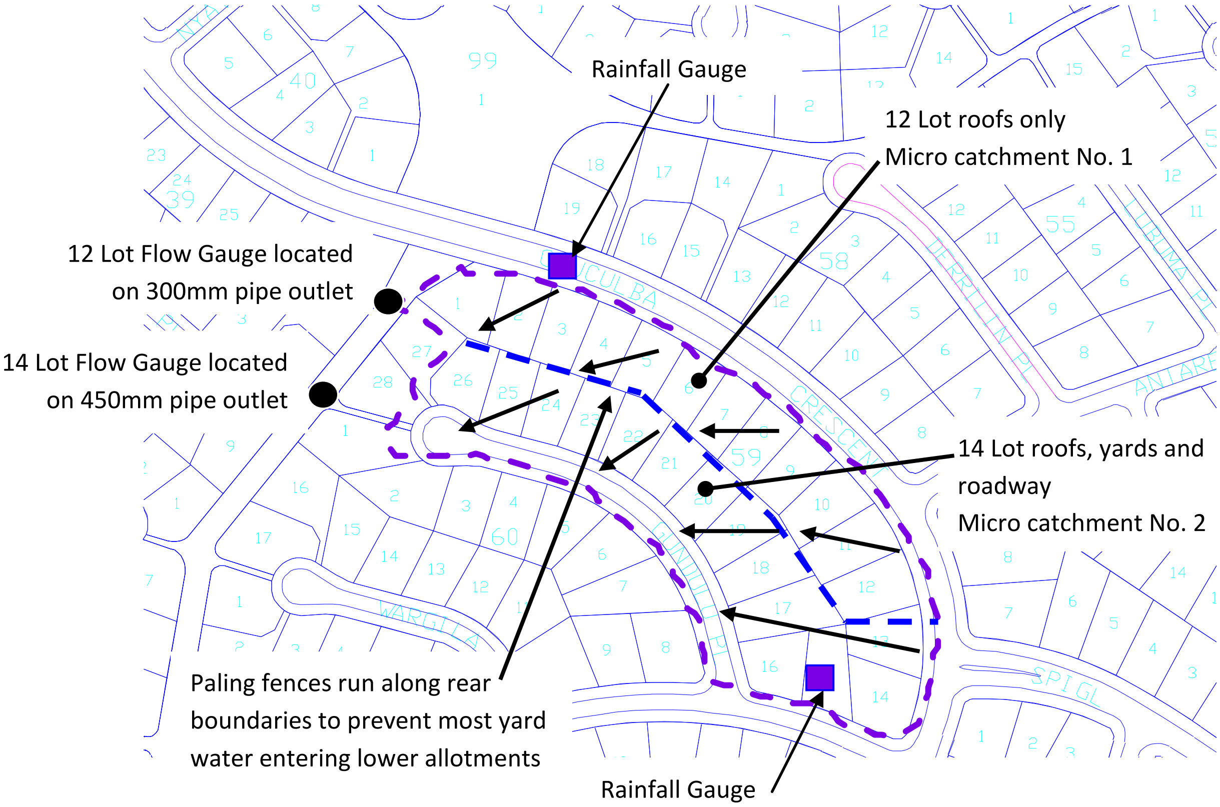


Figure D.2 Giralang Gauged Urban Micro Catchments (after Goyen, 2000)

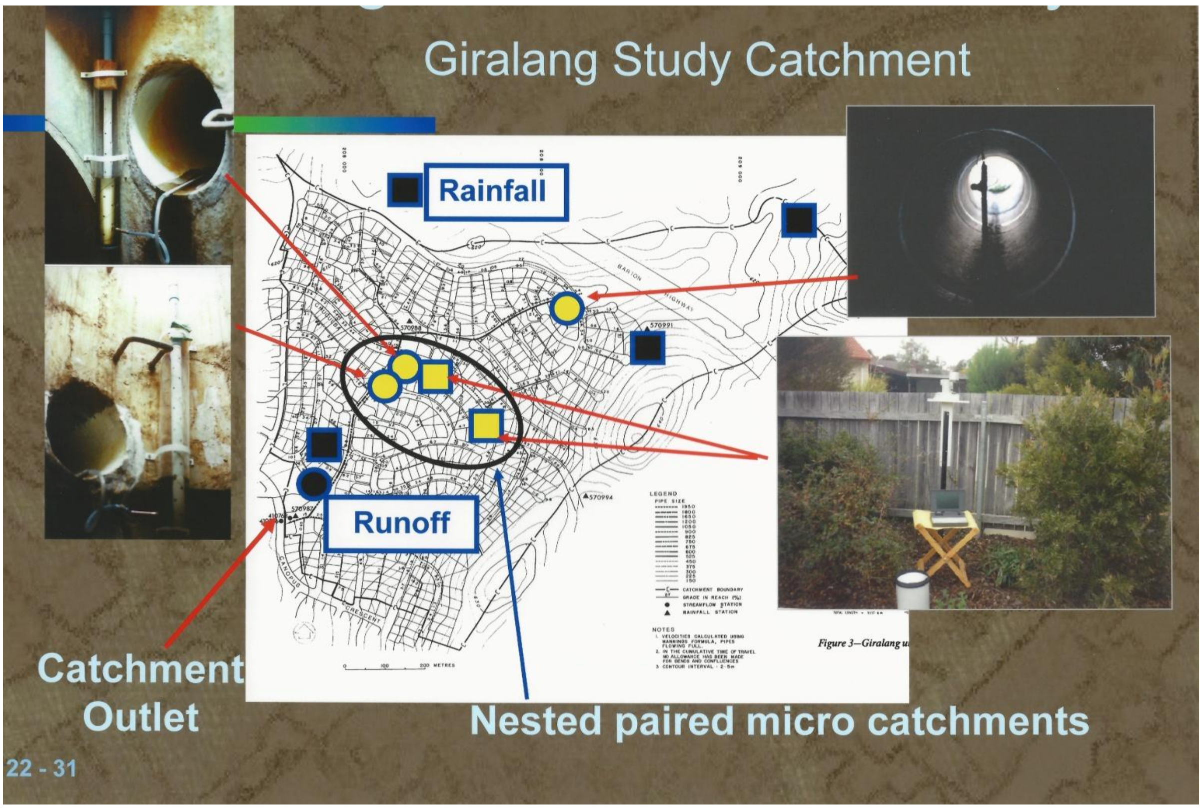


Figure D.3 Giralang Gauged Urban Micro Catchment Gauges (after Goyen, 2000)

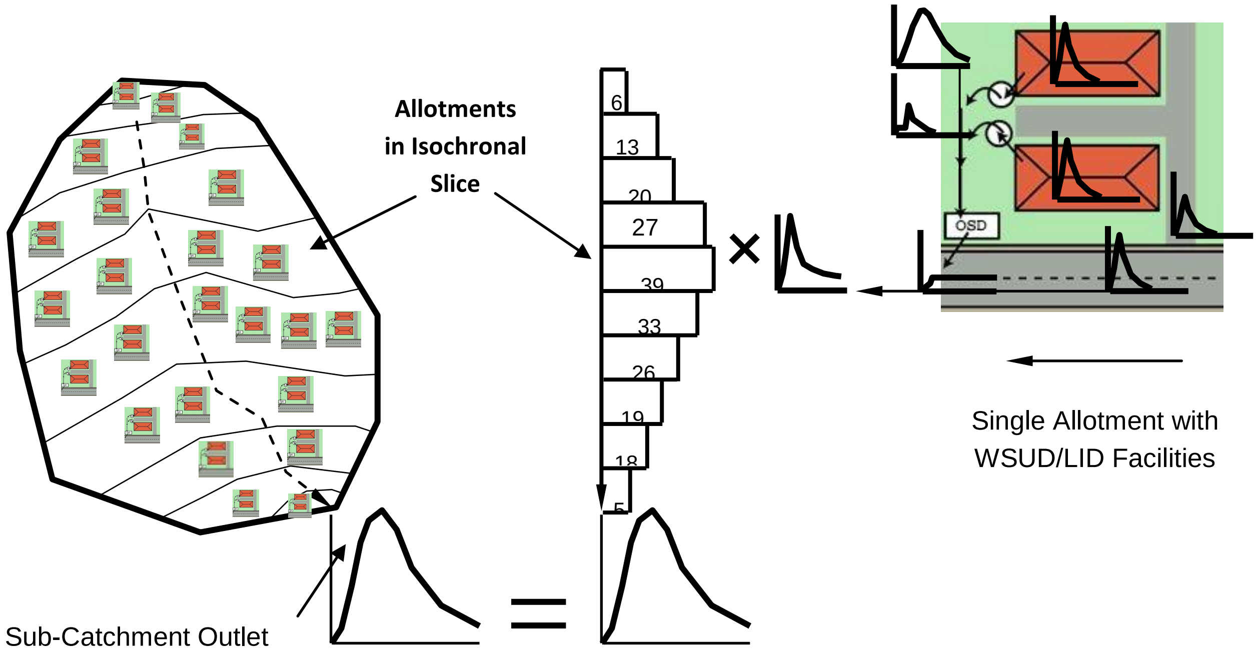


Figure D.4 Conceptual Representation of Isochronal Modelling Approach applied to xprafts

The methods described by Goyen, 2000 have been incorporated into the sub-catchment hydrograph estimation module of **xprafts**. The modifications to the **xprafts** analysis procedure included an alternate sub-catchment analysis procedure that is indicated diagrammatically in Figure D.4.

Runoff is estimated separately for the roof and gutter, adjacent road surface and paving and pervious gardens and lawn areas. A virtual allotment drainage network is constructed to represent lagging, bypass, capture and additional storage routing and infiltration/ evapotranspiration within the various WSUD facilities. The outputs from each structure as well as any bypass flows are combined to give the total runoff hydrograph from a typical allotment.

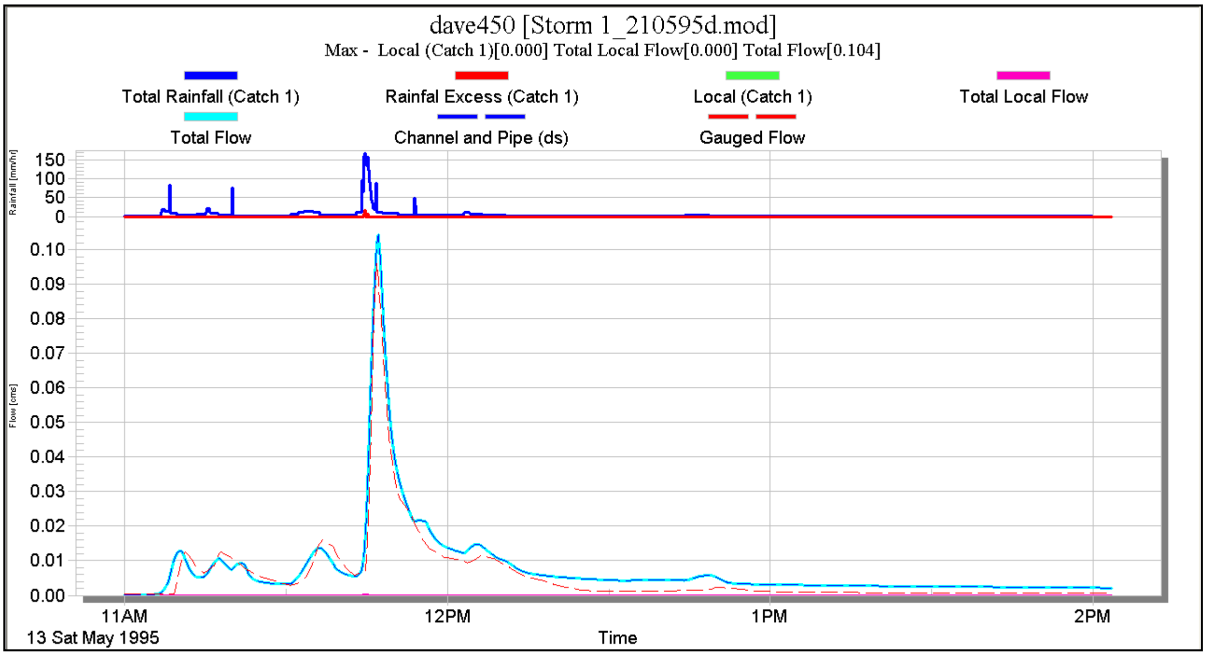
The method allows the definition of a wide range of WSUD/LID facilities including allotment storage devices, infiltration beds and rain water tanks. The procedures allows for variable structure sizes as well as variable capture and bypass percentages. Additional parameters to define the percentage breakdown in impervious surfaces between roofs, paving and road surfaces was also included.

The models as described by Goyen (2000) were adopted without any modification apart from the addition of evaporation from impervious surfaces during the extended duration summer daytime events.

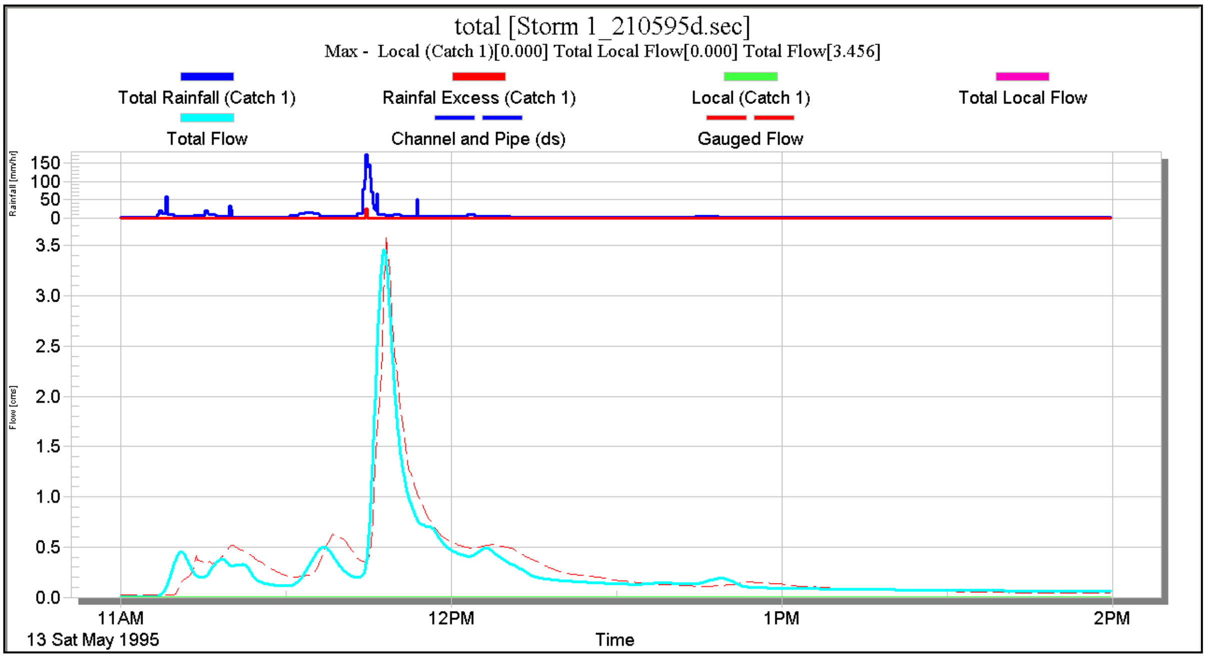
The RAFTS model developed as part of the research, which analysed gauging data collected in the micro catchments in the period 1993-1995, has been used in this analysis together with the updated total catchment flow gauging up to 2013.

A review of the original calibration was carried out using four separate storm events from the 1993 – 1995 dataset. These included events on the 3 January 1993, 6 March 1993, 5 April 1993 and 13 May 1995 that represented a range of events between <1 yr ARI to around 8 yr ARI.

This allowed a review of events that were predominately impervious area runoff only through to events that only had pervious area runoff in the later portions of the event to an event that had a significant proportion of pervious area runoff. **Figures D.5 – D.8** show the fit between the simulated flows and the gauged flows for both the 1.27ha embedded Micro catchment No. 1 (14 Lots), and the 62.9 ha urban catchment (526 Lots).

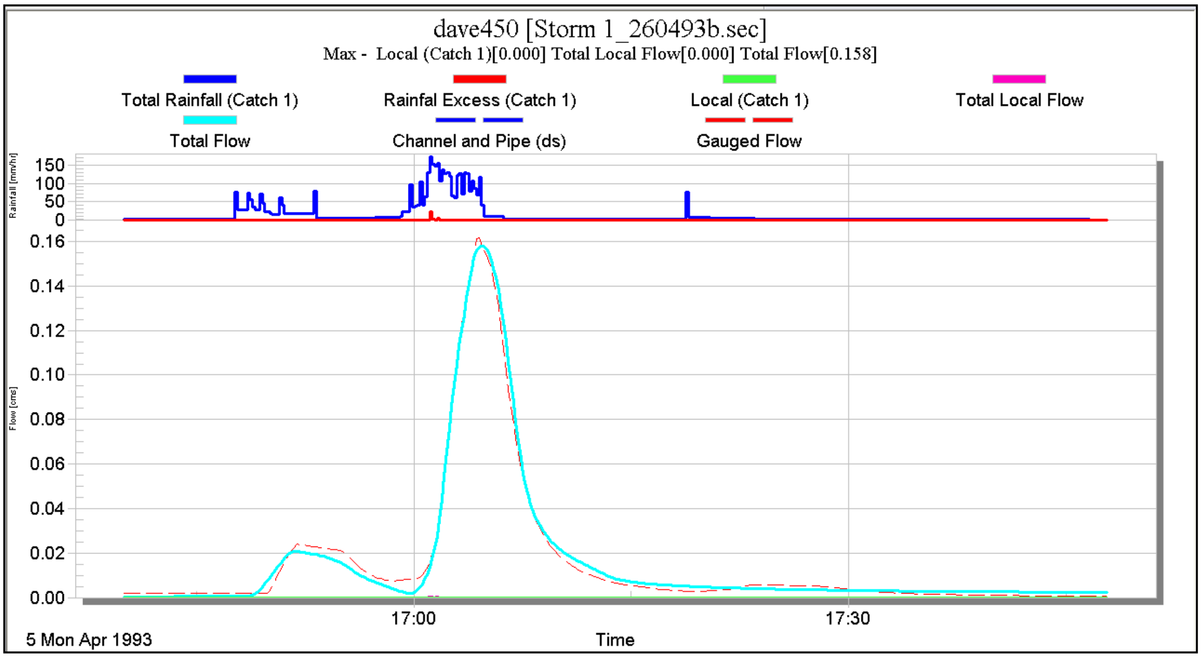


(a) Micro Catchment No. 1 (R^2 = 0.924)



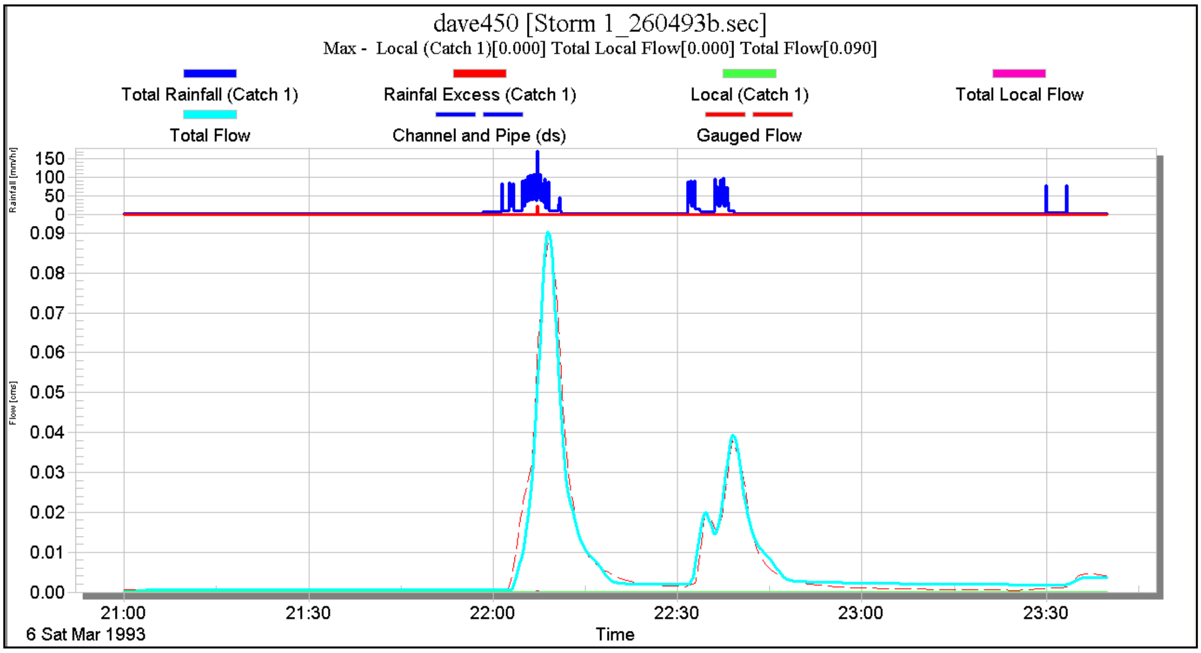
(b) Giralang Urban Catchment ((R^2 = 0.894)

Figure D.5 Observed and Predicted Flows during Storm of 13 May 1995



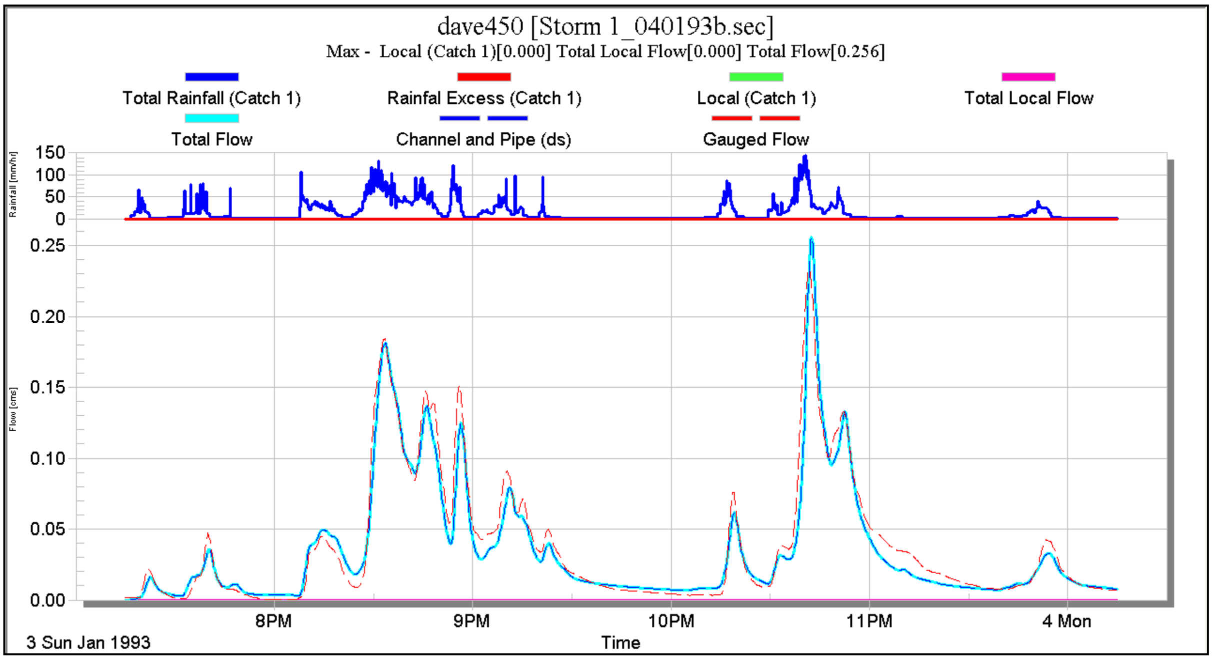
Micro Catchment No. 1 (R^2 = 0.992)

Figure D.6 Observed and Predicted Flows during Storm of 5 April 1993

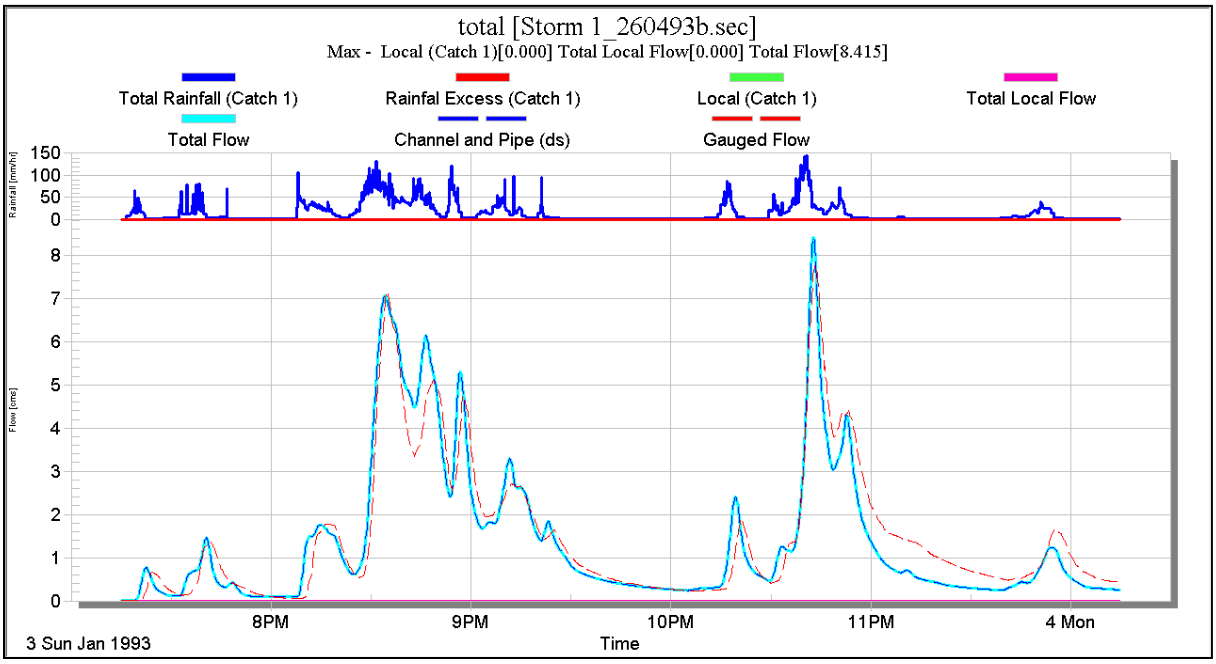


Micro Catchment No. 1 ((R^2 = 0.977)

Figure D.7 Observed and Predicted Flows during Storm of 6 March 1993



(a) Micro Catchment No. 1 (R^2 = 0.945)



(b) Giralang Urban Catchment ((R^2 = 0900)

Figure D.8 Observed and Predicted Flows during Storm of 3 January 1993

The above review and the 2000 analysis used only the micro catchment rainfall gauge that is located near the centre of the overall catchment.

The coefficient of determination (R^2) for Micro catchment No. 1 for all events was in excess of 0.92 with minimal rainfall variation across the micro catchment. The coefficient of determination R^2 for the urban catchment for both events reviewed were greater than 0.9. The differences in level of agreement between observed and predicted flows from Micro catchment No. 1 and the urban catchment were mainly attributed to spatial variations in rainfall across the overall catchment.

Goyen, 200 reported that the area ratio between Micro catchment No. 1 and the urban catchment was 49.5 the ratio of the peak flows observed peak flows varied between 27.9 and 49.4. Of the 8 larger events in excess of a 1 yr ARI event the average ratio was 35.3 while the average ratio for the 24 smaller events was 37.0. The variation was attributed to spatial variance both in rainfall depths as well as variance in the temporal distribution of rainfall.

Based on the 1993 – 1995 gauging period the above variations suggest that when rainfall variation is not explicitly taken into account the errors in estimated peak flows could be up to +/-30%. This spatial variation over small urban catchments was much larger than reported in the literature. While Project 2 Spatial Patterns of Design Rainfall provides guidance on areal reduction factors the minimum catchment area is 100 ha and the minimum storm duration is 1 hour. In the case of the ACT the latest guidance would give a 3.3% reduction of the point rainfall intensity.

Despite the above reservations in respect to rainfall variance over even small urban catchments the above the calibration review above demonstrates that it is possible to calibrate urban catchments to historical events to an acceptable level of accuracy. This has been made possible mainly via the replacement of simple initial loss/continuing loss rate model with a more physically based water balance model with separate testing of infiltration and saturation/drainage at each modelling time step.

This review was of complete storms and could have just as easily been applied to complete continuous rainfall records as has been undertaken in other studies.

In the past it has been difficult to transfer calibrated urban catchment models into a design mode to estimate flow quantiles usually between 1 yr ARI and 100 yr ARI.

The current practice when using rainfall-runoff models to estimate flow quantiles in ungauged catchments is the apply a dimensionless design rainfall burst temporal pattern to the rainfall intensity return period of interest determined in accordance with ARR procedures..

The main issues in this regard have been the lack of recommendations for appropriate rainfall losses to apply when assessing each of the flow quantiles. Additionally, while the rainfall losses which have been applied are intended to represent infiltration they also tend to try and compensate for areal rainfall variance and for any difficulties in the calibration. In the case of the 1993 Part II study it was found that in order to match the flow quantiles obtained from FFA that the initial pervious rainfall loss needed to increase with increasing ARI ie. the 2 yr ARI peak flow was best fitted by a 5.0 mm initial pervious area rainfall loss while the 100 yr ARI peak flow was best fitted by a 15.0 mm initial pervious area rainfall loss.

Since the 1993 Part II study in the ACT further development has occurred within the RAFTS modelling system as described by Goyen (2000). In particular development of the ARBM based rainfall loss and water balance model was altered to allow a distribution of porosity across the pervious surfaces to better reflect the variation experienced across urban allotments. This behaviour was previously postulated as far back as 1965 by Lindsay and Crawford in their development of the Stanford Watershed Model.

In the 1993 Part II study the ARBM did not contain the distribution capability and as such only a single set of values were provided (refer **Appendix D.1**). The distribution of porosity has allowed a more realistic calibration tool that now better simulates the commencement of pervious runoff by varying infiltration rates across pervious areas.

The design storm concept in this discussion paper was further tested using the calibrated Giralang catchment (enhanced) RAFTS model and embedding the ARR design storm burst temporal patterns in the storm recorded on 3 January 1993. The 25 minute ARR design burst pattern was embedded first at the commencement of the event and then separately at the commencement of the first major peak some 1 hour and 15 minutes into the event.

The results from this analysis of the Giralang urban catchment (62.9 ha) are summarised in **Table D.1**.

Table D.1 Estimated Peak Flows for Embedded Storm of 3 January 1993 on Giralang Catchment

ARI (yrs)	Rainfall Intensity (mm/h)	Rainfall Depth (mm)	5 minute Temporal Partition					Peak Flow		
			No. 1 (mm)	No. 2 (mm)	No. 3 (mm)	No. 4 (mm)	No. 5 (mm)	FFA (m^3/s)	Early* (m^3/s)	Middle** (m^3/s)
1	27.1	11.3	1.92	3.16	4.41	1.02	0.79	4.3	3.10	4.15
2	35.5	14.8	2.51	4.14	5.77	1.33	1.04	6.7	4.38	5.56
5	46.9	19.5	3.32	5.47	7.62	1.76	1.37	8.2	6,78	8.12
10	54.2	22.6	3.84	6.33	8.81	2.03	1.58	9.2	7.55	9.44
20	64.0	26.6	4.53	7.46	10.39	2.40	1.87	11.3	10.06	12.11
50	77.4	32.3	5.81	8.39	11.29	3.55	3.22	12.7	12.51	13.82
100	88.3	36.8	6.62	9.56	12.87	4.04	3.68	14.2	16.05	17.16

** "Early" refers to embedment of the ARR storm burst at the start of the storm*

***"Middle" refers to the embedment of the ARR storm burst at the commencement of the first major peak some 1 hour and 15 minutes into the event*

It was also found that when the design storm was embedded under the second major historical peak around 3 hours and 15 minutes after the commencement of the storm the resulting 10 yr ARI peak flow was found to be 9.40 m^3/s or slightly less than the embedment under the first major peak. This was due to the intervening dry period that allowed the re-establishment of the infiltration capacity and the degree of saturation of the soil upper store.

It was concluded that "middle" embedment gave peak flows which were the best overall fit to the peak flows calculated by FFA.

While it was expected that the estimated peak flows would be higher than the FFA estimates due to the use of point IFD data it was found that the estimated peak flows for events up to the 5 yr ARI were slightly lower than the FFA estimates while the estimated peak flows for events greater than or equal to 10 yr ARI were slightly higher than the FFA estimates. The steeper slope of the estimated flood frequency curve may be due to the design storm burst temporal pattern. It may also be due to the expectation that analysing the storm burst of a given ARI yields a peak runoff of the same ARI which may not be correct on average.

The only way that estimated peaks could better match the peak flows estimated by FFA would be to abandon the design storm approach and instead directly model the annual maximum runoff events as well as additional events to create partial series data ie. to create a synthetic database of peak flows which could be analysed using FFA.

If the simulation of historical storms is able to match the observed events as closely as achieved in the Giralang catchment in the period 1993 – 1995 then the resulting flow quantiles obtained from FFA of the simulated peak flows should closely match the flow quantiles derived from the gauged peak flows.

D.2 HEWITT

The details of the Hewitt urban catchment and its gauging stations are provided by Goyen (2000). An overview of the Hewitt catchment is given in **Figure D.9.**

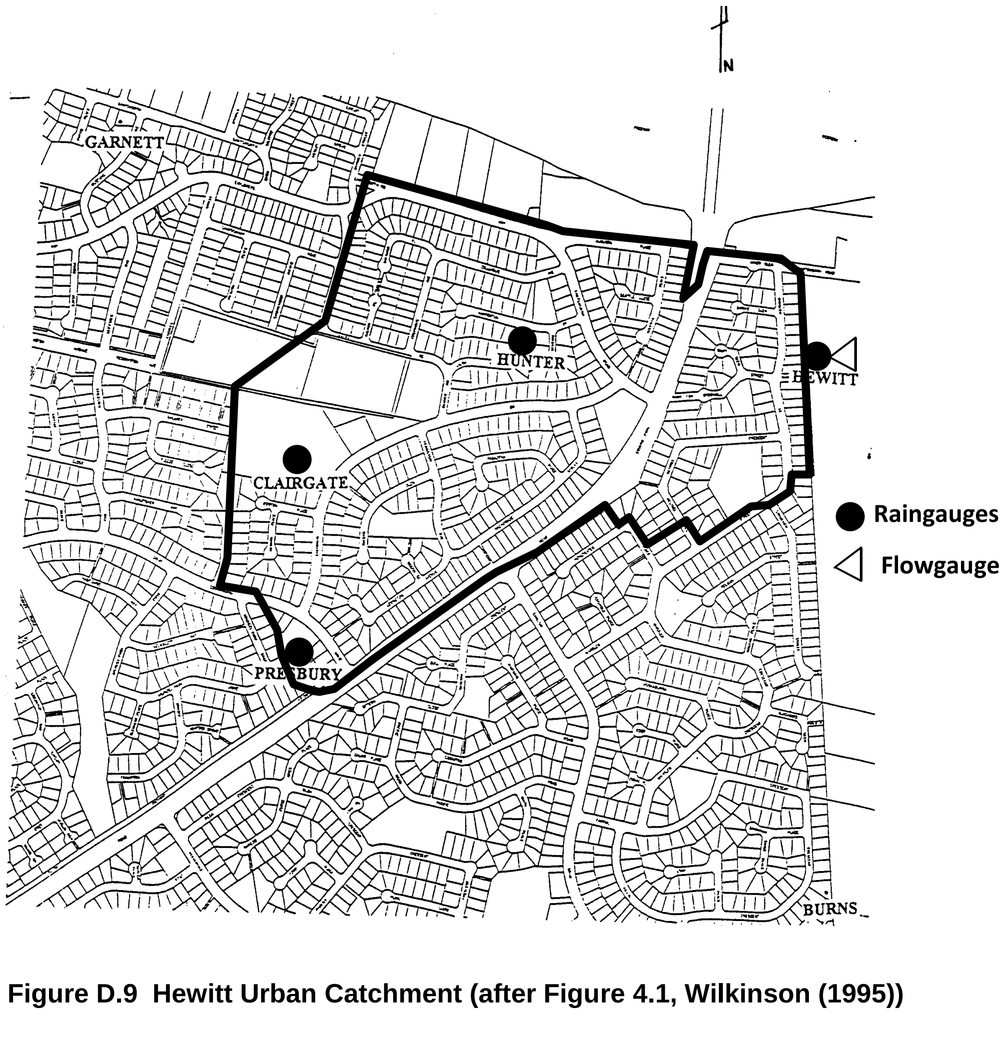
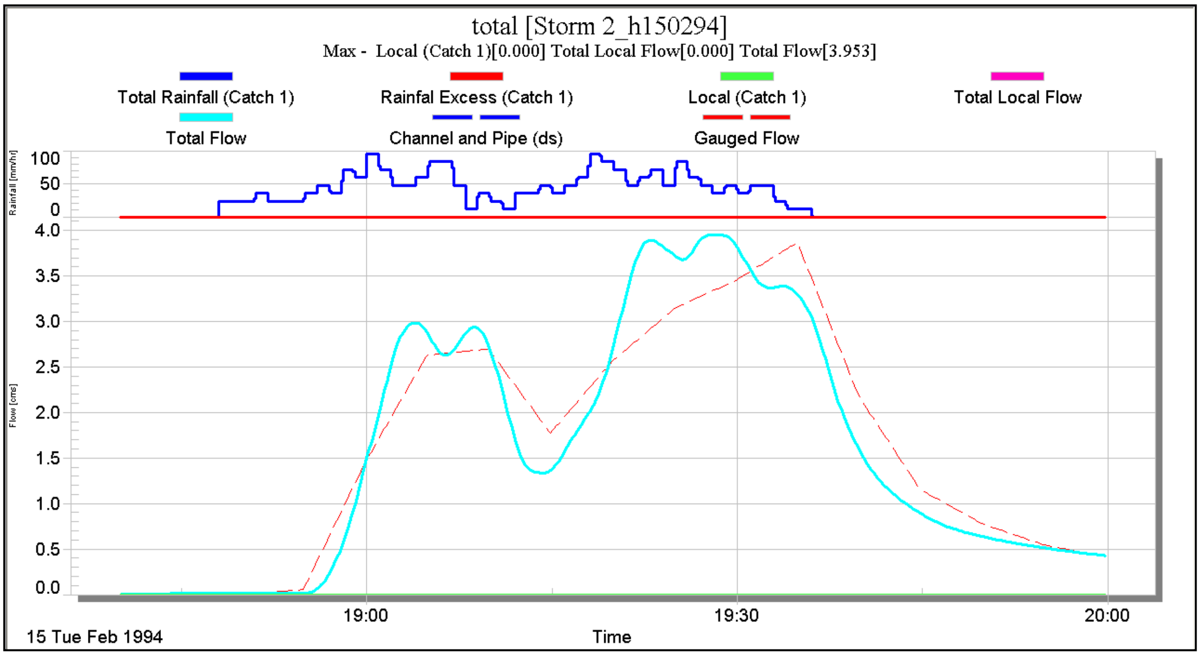


Figure D.9 Hewitt Urban Catchment (after Figure 4.1, Wilkinson (1995))



Hewitt Urban Catchment ((R^2 = 0.903)

Figure D.11 Observed and Predicted Flows during Storm of 15 February 1994

The original calibration was carried out using three separate storm events including the storm of 15 February 1994. **Figure D.11** shows the fit between the simulated flows and the gauged flows for the 62 ha urban catchment (556 Lots).

The design storm concept in this discussion paper was further tested using the calibrated Hewitt catchment (enhanced) RAFTS model and embedding the ARR design storm burst temporal patterns in the storm recorded on 15 February 1994. The results from this analysis of the Hewitt urban catchment (62 ha) are summarised in **Table D.2**.

Table D.2 Estimated Peak Flows for Embedded Storm of 3 January 1993 on Hewitt Catchment

ARI (yrs)	Rainfall Intensity (mm/h)	Rainfall Depth (mm)	5 minute Temporal Partition					Peak Flow		
			No. 1 (mm)	No. 2 (mm)	No. 3 (mm)	No. 4 (mm)	No. 5 (mm)	FFA (m^3/s)	Early* (m^3/s)	Middle** (m^3/s)
1	37.2	15.5	2.64	4.34	6.05	1.40	1.08		3.85	4.49
2	48.1	20.0	3.41	5.61	7.82	1.80	1.40		5.00	6.22
5	62.4	26.0	4.42	7.29	10.15	2.34	1.82		7.57	10.07
10	70.9	29.5	5.02	8.27	11.52	2.66	2.07		8.95	11.76
20	82.0	34.2	5.81	9.56	13.32	3.07	2.39		13.38	15.80
50	96.6	40.2	7.24	10.46	14.08	4.43	4.02		16.22	18.32
100	107.8	44.9	8.08	11.67	15.72	4.94	4.49		20.96	22.80

It should be noted that the depth of rainfall in the Hewitt catchment is between 22% and 37% higher than Giralang. This not only increases flow peaks due to increased rainfall intensity it also potentially saturates the upper soil stores within pervious areas during an event. This could have the effect of increasing flow peaks if the primary storm burst is embedded later in the event.

APPENDIX E

WILLING AND PARTNERS (1989)
DRAINAGE DESIGN PRACTICE FOR LAND DEVELOPMENT IN THE ACT.
PART I: RATIONAL FORMULA PROCEDURES

National
Capital
Development
Commission

DRAINAGE DESIGN PRACTICE FOR LAND DEVELOPMENT IN THE A.C.T.

PART I: RATIONAL FORMULA PROCEDURES

WILLING & PARTNERS
WP RESEARCH

National
Capital
Development
Commission

DRAINAGE DESIGN PRACTICE FOR LAND DEVELOPMENT IN THE A.C.T.

PART I: RATIONAL FORMULA PROCEDURES

JANUARY 1989

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1 INTRODUCTION

The following report was prepared by Willing & Partners Pty Ltd in response to Brief No. E34/88 issued by the National Capital Development Commission on the 6th of May 1988.

To assist the National Capital Development Commission and ACT Water prepare a detailed ACT Urban Stormwater Design Handbook for land development projects, this brief sought specialist advice on particular aspects of runoff estimation procedures.

The aim of the study was to make use of locally gauged urban catchment rainfall and runoff data to derive runoff estimation parameters for a range of suitable rainfall/runoff simulation procedures.

In particular, the study was to report on the suitability of the recently published guideline "Australian Rainfall and Runoff - A Guide to Flood Estimation, 1987" (IEAust., 1987) for urban runoff estimation within the ACT.

Since the Handbook is primarily intended for designers of small subdivisions the emphasis in this study was the derivation and testing of parameters for use in the Rational Formula method. It is intended that a subsequent study will investigate acceptance criteria and rainfall loss rates and other rainfall/runoff model parameters for hydrograph estimation procedures.

In summary the objectives of this study were to:

(i) Develop design parameters for the Rational Formula method for inclusion in the proposed ACT Urban Stormwater Design Handbook, including-

- Runoff coefficients
- Time of concentration

(ii) Develop acceptance criteria for suitable urban runoff estimation procedures including the provision of representative catchment and runoff data to test proposed rational formula methods.

2 SUMMARY AND RECOMMENDATIONS

2.1 Summary

The current study has compared the methodologies for using the Rational Formula drainage design as recommended in both the 1977 and 1987 editions of the Institution of Engineers, Australia publication "Australian Rainfall and Runoff" (I.E.Aust., 1977 & 1987).

The two documents differ significantly in their specific recommendations for estimating both the subarea time of concentration for overland flow (t_c) and the appropriate subcatchment runoff coefficient (C).

To test the acceptability of either the 1977 or 1987 recommendations, simulations were undertaken of both the Giralang and Mawson gauged urban catchments and compared with the gauged data. Both gauged catchments have in excess of twelve years of runoff records.

The Giralang runoff data was first used to test the results of a previous study carried out in 1983 which utilised the AR&R, 1977 recommendations. Comparisons of the gauged and predicted flood frequency curves over the normal design range were carried out to assess the accuracy of the 1983 analysis. Even though the results were close, the runoff coefficient values from the 1983 study were modified by re-classifying pervious areas from Curve 5 to Curve 4 within the AR&R, 1977 curves (refer Figure 1). This further improved the fit to the gauged flood frequency curve and was adopted as the calibrated run.

The data set was subsequently modified to reflect the recommended procedures of AR&R, 1987 for the estimation of both runoff coefficients and times of concentration for overland flow respectively. The results of this action was to greatly reduce the estimated flows (refer Figure 4). Neither the runoff coefficient nor times of concentration for overland flow procedures individually or in concert provided acceptable results. To verify these findings, similar simulations were undertaken of the second gauged urban catchment at Mawson. This catchment is some four times the size of the Giralang catchment.

The results of a previous 1980 study were first checked against the gauged flood frequency data. Subsequently, the runoff coefficient values were modified in accordance with the AR&R, 1987 procedure and the revised data set was run. The Mawson results were found to be very similar to the results of the Giralang analysis (refer Figure 6).

It was only possible to obtain surface flow times of concentration using the AR&R, 1987 procedure which were similar to the values obtained using the AR&R, 1977 procedure by greatly reducing the value of the surface roughness (n^*) for various rainfall intensities, ground slopes and flow lengths. No guidelines are provided in AR&R, 1987 to select appropriate roughness values from the recommended ranges of roughness values. Likewise, the calibrated roughness values were outside the recommended ranges for roughness values. Hence, it was not possible to achieve consistently acceptable results using the AR&R, 1987 procedure for the estimation of surface flow times of concentration.

A number of lag times were also determined from recorded hydrographs from the Giralang and Mawson gauging stations. These lag times lend further support to the acceptability of the AR&R, 1977 procedure for the estimation of surface flow times of concentration. The times of concentration derived from these lag times generally agreed with the estimates gained from the AR&R, 1977 procedures.

2.2 Conclusions

The results from this study lend further support to the continued use of the Rational Formula for drainage design in small to medium sized urban catchments.

The procedures outlined in the AR&R, 1977 for estimating runoff coefficients and surface flow times of concentration were found to be reliable when using the Rational Formula to estimate a flood frequency curve of flood peaks for design purposes.

In the Giralang analysis in particular, it was shown that it is essential to estimate peak flood flows from partial areas. The peak flood flow at the catchment outlet was underestimated by 33% when only the total area was considered.

It was also found that the adoption of the AR&R, 1987 procedures for both estimating runoff coefficients and surface flow times of concentration resulted in a flood frequency curve which was 40 - 60% lower than the gauged curve. In effect, the 5 Yr ARI peak flood discharge predicted using the AR&R, 1987 procedures was in fact equivalent to the gauged 1 Yr ARI peak flood discharge.

2.3 Recommendations

From the results of this study, and in accordance with the statement made in AR&R, 1987 : "where circumstances warrant, designers have liberty, and perhaps a duty to use other procedures and data. The use of new or improved procedures is encouraged especially where these are more appropriate than the method described in this publication (AR&R, 1987)", the following recommendations are made:

(i) For all urban drainage design within the ACT using the Rational Formula, the following relations are recommended for the determination of surface flow times of concentration (t_o) and corresponding runoff coefficient (C) values.

Time of overland flow

$$t_o = 107 \frac{n\ L^{0.333}}{S^{0.2}}$$

where

t_o	=	overland flow travel time (minutes)
L	=	flow path length (m)
n	=	Horton's roughness value for the surface
S	=	slope of surface (%)

Surface Type	Recommended Horton roughness value
Paved surface	0.015
Bare soil surface	0.0275
Poorly grassed surface	0.035
Average grassed surface	0.045
Densley grassed surface	0.060

Runoff coefficient

$$C_i = 0.90$$

$$C_p = 0.91 - 3.14\ I^{-0.594}$$

where

C_i	=	runoff coefficient for impervious surfaces
C_p	=	runoff coefficient for pervious grassed surfaces
I	=	rainfall intensity (mm/h)

(ii) For all urban drainage analyses, full partial area effects be taken into account

(iii) Further studies be undertaken to further examine possible modifications to the recommended AR&R, 1987 procedures to improve the estimation of surface flow times of concentration and corresponding runoff coefficients. In particular, further studies should aim to determine appropriate surface roughness (n^*) values for use in the kinematic wave formulation for overland flow in Australia.

3 REVIEW OF AR&R RATIONAL FORMULA PROCEDURES

The majority of stormwater drainage provisions associated with land development within the ACT have been either designed or analysed and modified where necessary using the procedures laid down in either AR&R,1958 or AR&R, 1977.

AR&R,1987 has departed from AR&R, 1977 in a number of respects including in the area of urban runoff estimation procedures.

All editions of AR&R have recommended the Rational Method for the estimation of urban peak flows to size both underground pipes systems as well as channels and floodways.

Only when full hydrographs have been required, for example to size retarding basins or assess flood wave routing along large floodways or rivers, have AR&R, 1958 and 1977 recommended the use of hydrograph generation procedures. AR&R, 1987 has departed from this position by discussing, along with the Rational Method for urban piped drainage systems, a range of computer based numerical procedures to compute hydrographs and in some instances to carry out complex hydraulic simulations.

Although no particular computer program is recommended, the ILSAX program is described in detail to indicate some of the general capabilities of urban drainage computer models.

The RAT-HGL program (Messner & Goyen, 1985) is the only Rational Method computer program mentioned in the urban drainage section of the AR&R, 1987. Other simulation programs which simulate hydrographs and analyse steady as well as unsteady flow pipe hydraulics are also cited. These models include SWMM (Huber et al, 1981) and WASP (Price, 1981).

3.1 The Rational Method

The Rational Formula method has remained the most widely used peak flow estimation procedure for urban drainage design in Australia for over 30 years.

The simplicity of its formulation, as shown in Equation 1, has ensured its continued use even though many more explicit and complex computer simulation techniques, that theoretically should perform better, are now available.

$$Q = \frac{1}{360} C.I.A \qquad (1)$$

where Q = design peak flowrate (m^3/s),
C = a dimensionless runoff coefficient,
I = rainfall intensity (mm/h),
A = catchment area (ha).

The rainfall intensity, I, is obtained from Intensity - Frequency - Duration (IFD) data for the duration equal to the time of concentration, t_c.

The three editions of AR&R (IEAust., 1958, 1977 and 1987) have each described the use of the Rational Formula method. The main differences between the three editions have been the need to assess partial area effects and the recommended procedures to estimate runoff coefficients and overland flow times of concentration. Each of these aspects is discussed below.

3.1.1 Partial Area Effects

AR&R, 1958 provided a comprehensive procedure known as the "Tangent Check" to determine the critical time for an area and the appropriate partial area to be applied in the Rational Formula procedure. It was argued that a portion of the catchment area when multiplied by the higher rainfall intensity resulting from a shorter time of concentration could provide a higher peak flowrate than the peak flowrate contributed by the total area. AR&R, 1977 subjectively recommended against the use of partial area assessments including the "Tangent Check" on the premise that the Rational Method was not accurate enough to warrant such a check.

AR&R, 1987 re-assessed the partial area question and recommended a single partial area check by calculating a partial area based on the times of concentration of impervious zones directly connected to the pipe system. Hence, AR&R, 1987 falls significantly short of the AR&R, 1958 recommendations for the checking of partial areas. In view of the easy access to computer based techniques, as described in Section 3.2, it is hard to reconcile the availability of such programs with the criteria of "simplicity" which was used in AR&R, 1987 to justify a requirement that only a nominal partial area check be undertaken when using the Rational Method to design piped stormwater systems.

This deficiency is particularly important since it has been previously reported (Willing & Partners, 1983) that peak flowrates in urban stormwater systems can be seriously underestimated by ignoring partial area effects. This issue is further discussed in Section 4.

3.1.2 Runoff Coefficients

AR&R, 1958 and 1977 both provided the same runoff coefficient estimation procedures which were based on series of curves representing different land uses and types. The curves, which are presented in Figure 1 and described in Table 1, were reproduced directly from the empirical curves published by Ordon, 1954.

Curve No.	Description
1	Impervious Roofs, Concrete City Areas Full and Solidly Built Up
2	Surface Clay, Poor Paving, Sandstone Rock Commercial & City Areas Closely Built Up
3	Semi Detached Houses on Bare Earth
4	Bare Earth, Earth with Sandstone Outcrops Urban Residential Fully Built Up with Limited Gardens
5	Bare Loam, Suburban Residential with Gardens
6	Widely Detached Houses on Ordinary Loam Suburban Fully Built Up on Sand Strata
7	Park Lawns and Meadows
8	Cultivated Fields with Good Growth Sand Strata

Table 1 Runoff Coefficient Curve Number Descriptions

The 1987 edition of AR&R varies from the previous two editions in its presentation of runoff coefficients for design purposes. This edition presents a:

> "composite relationship reflecting experience of drainage authorities and evidence from the few gauged urban catchments with suitable lengths of record ..." (IEAust., 1987).

It is stated that:

> "it should be used in preference to the runoff coefficient relationships given in previous editions ..." (IEAust., 1987).

The 10 Year ARI runoff coefficients recommended in the AR&R, 1987 are presented in Figure 2. Also shown for comparison are the data used to define the upper and lower bounds of the interpolation zone. The location of the gauged catchments, their size and representative rainfall intensity are given in Table 2.

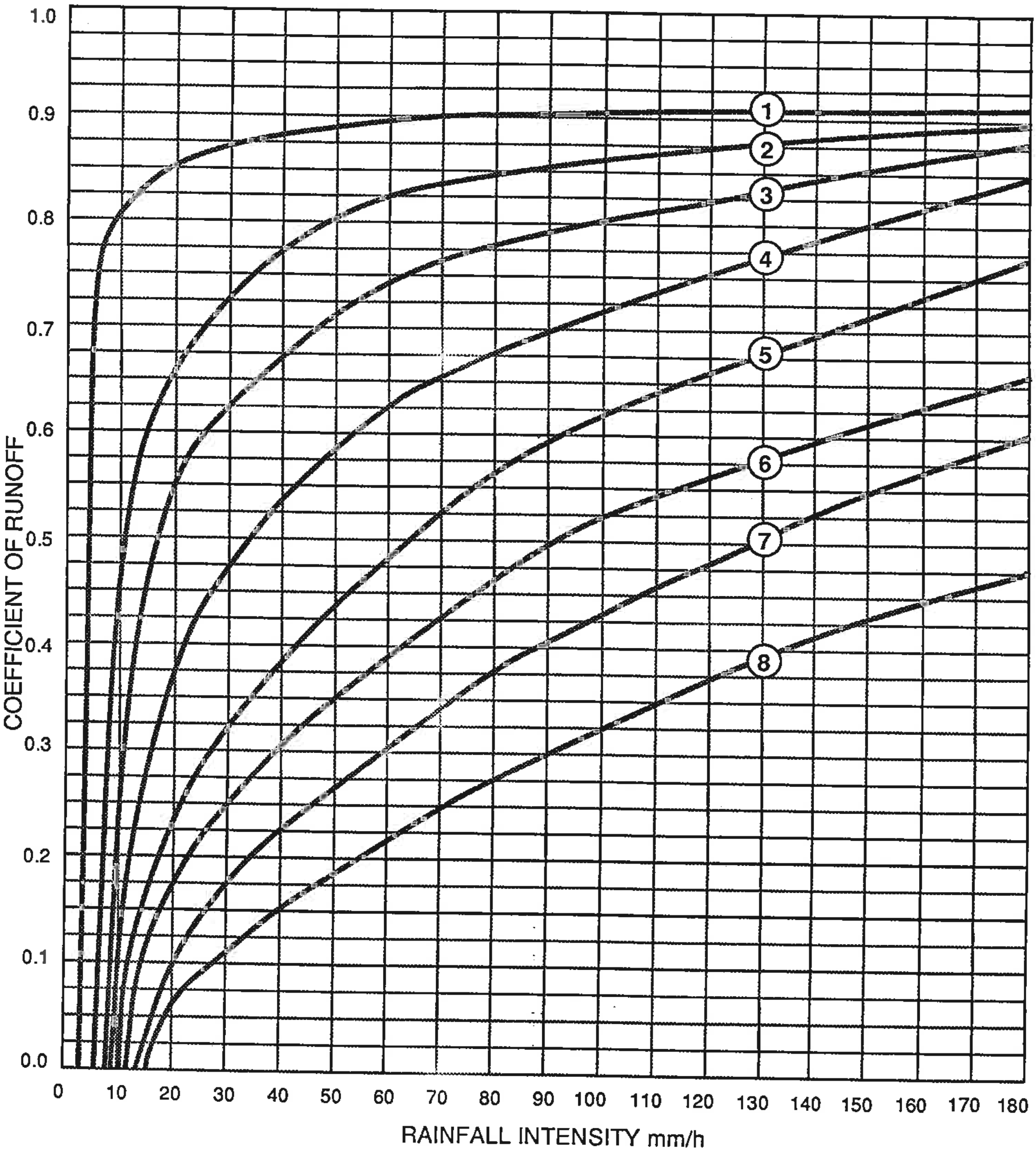

Figure 1 Runoff Coefficients for Urban Catchments (after IEAust., 1958 & 1977)

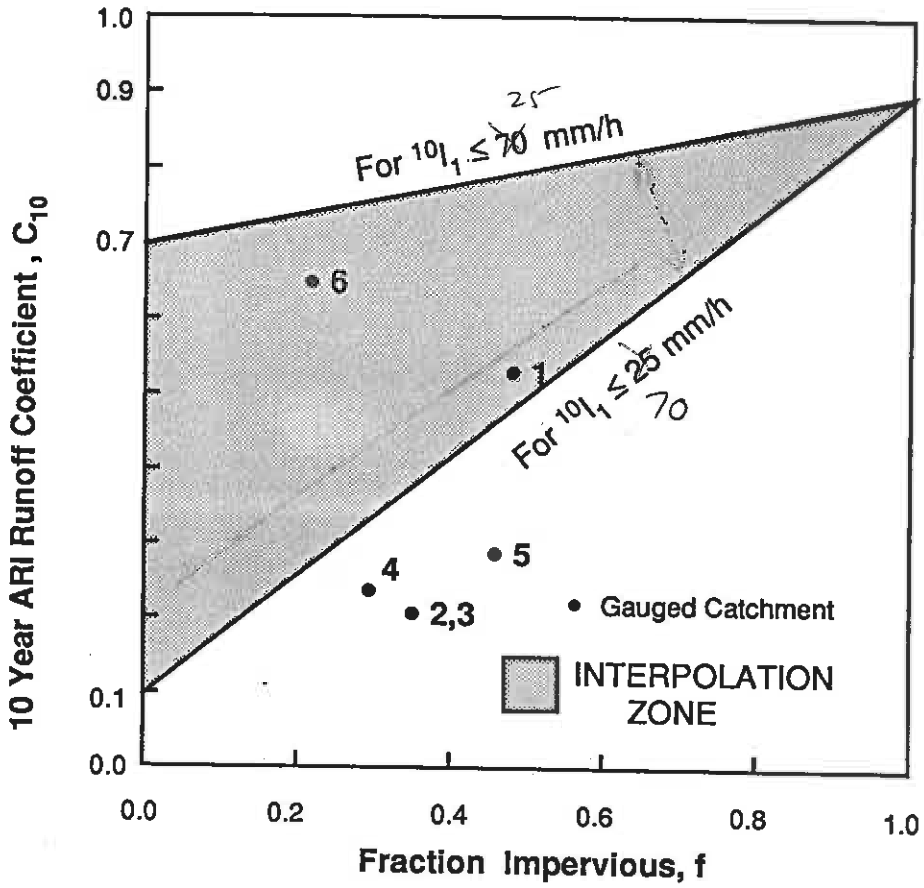

Figure 2 10 Year ARI Runoff Coefficients (after IEAust., 1987)

Gauged Urban Catchment No.	Location	Catchment Area (ha)	$^{10}I_1$ (mm/h)
1	Powells Creek, Strathfield, Sydney	231	48.9
2	Box Hill Main Drain, Box Hill, Melbourne	113	28.0
3	Vine Street Main Drain, Braybrook, Melbourne	70	29.0
4	Ashmore Ave Main Drain, Mordialloc, Melbourne	53	26.5
5	Gardenia Road Main Drain, Doncaster, Melbourne	80	28.1
6	Yarralumla Creek, Mawson, Canberra	382-400	32.2

Table 2 Gauged Urban Catchment Descriptions

The graphical relationship is further supplemented by the following numerical relationships:

$$C_{10} = 0.9f + C^1_{10}(1 - f) \qquad (2)$$

and

$$C^1_{10} = 0.1 + 0.0133\,({}^{10}I_1 - 25) \qquad (3)$$

where C_{10} = 10 Year ARI runoff coefficient
C^1_{10} = pervious area 10 Year ARI runoff coefficient
f = fraction impervious (0.0 to 1.0)
${}^{10}I_1$ = 10 Year ARI, 1 hour rainfall intensity

For average recurrence intervals other than 10 years the C_{10} value is multiplied by a frequency factor from Table 3. Hence:

$$C_y = F_y C_{10} \qquad (4)$$

where F_y = frequency factor.

ARI (Years)	Frequency Factor, F_y
1	0.80
2	0.85
5	0.95
10	1.00
20	1.05
50	1.15
100	1.20

Table 3 Frequency Factors for Rational Method Runoff Coefficients (after IEAust., 1987)

In view of the absence of data to support the urban runoff coefficient estimation procedure proposed in AR&R, 1987 that the comment of Munro (1956) may still apply:

"The literature abounds with tabulations of graphs of C for various conditions, but few are observed from reliable evidence .... Apparently, Horner and Flynt (1936) are the only ones to have carried out a really comprehensive set of measurements."

3.1.3 Times of Concentration

The time of concentration (t_c) is estimated as the time of flow from the most remote edge of the subcatchment under consideration to its outlet. This often includes estimating an overland flow time over a pervious surface and then adding this time to the remaining channel flow time for flow usually within a gutter or pipe.

While the procedures for estimating travel times in channels or pipes have remained relatively unchanged over the period of the last 30 years, the estimation of overland flow times proposed in AR&R, 1987 differs from the earlier editions.

Both AR&R, 1958 and 1977 recommended the use of a nomograph for the determination of the time of overland flow. The AR&R, 1958 nomograph also presented a formula for the calculation of the overland flow time which was attributed to Friend, 1954. This equation is as follows (S.I. units):

$$t_o = 107 \frac{n\, L^{0.333}}{S^{0.2}} \qquad (5)$$

where
t_o = overland flow travel time (minutes)
L = flow path length (m)
n = Horton's roughness value for the surface
S = slope of surface (%)

AR&R, 1987 has departed from the empirical relationship given in Equation 5. Instead it recommended the use of the "kinematic wave" equation for overland flow time previously described by Ragan & Duru (1972). This equation is as follows:

$$t_o = 6.94 \frac{(L\, n^*)^{0.6}}{I^{0.4}\, S^{0.3}} \qquad (6)$$

where
t_o = overland flow travel time (minutes)
L = flow path length (m)
n^* = surface roughness
I = rainfall intensity (mm/h)
S = slope (m/m)

While the later equation for estimating overland flow times is based on a rigorous solution of the shallow overland flow equations, the appropriate values particularly for the surface roughness, n^*, are not well defined. The reported roughness values for pervious surfaces range between 0.05 and 0.70.

Reported values for Horton's roughness values in Equation 5 are similar to Manning 'n' roughness values and range between 0.015 for paved surfaces up to 0.06 for densely grassed surfaces.

As is further discussed in Section 4, the estimation of overland flow times can have a significant effect on the predicted peak flowrate due to its influence on the value of rainfall intensity input into the Rational Formula.

Section 4 discusses in detail the background and appropriateness of various estimating procedures for the overland flow time and recommends a procedure for adoption in the ACT.

3.2 RAT-HGL - A computer based Rational Method procedure

The RAT-HGL computer program (Messner & Goyen, 1983) is an Australian developed program for the analysis and design of urban piped stormwater systems. It is also the only reported model that complies with the Rational Formula procedures recommended in both AR&R,1977 and 1987.

The hydrological component of the program computes total area as well as critical area flowrates at every node within a drainage network.

A unique feature of the RAT-HGL program is its ability to thoroughly check all combinations of subareas above each node. The program tests the area contributions above each node by scribing isochrones equal to the times of travel between the node in question and the front and back of each contributing subarea. The equivalent impervious area contributions are then summed for each time of concentration and the resultant peak flowrate determined. The peak flowrate along with the associated time of concentration, rainfall intensity and critical area and total area results are subsequently output together.

The program allows up to seven return periods to be computed in one run.

Rainfall IFD data can be input via polynomial curve coefficients in accordance with AR&R, 1977 or as rainfall intensity and location data in accordance with AR&R, 1987.

Runoff coefficients are internally estimated using either the AR&R, 1977 runoff curves given in Figure 1 or the AR&R, 1987 procedure given in Figure 2 and Equations 2 to 4.

The hydraulic component of the RAT-HGL model includes comprehensive hydraulic grade line analysis algorithms in line with AR&R, 1987. The program allows for hydraulic energy losses due to pipe friction, junction pits and other arbitrary user defined losses. The re-routing of overland flow due to a limited inlet capacity and/or pipe surcharging is also considered in conjunction with the complete balancing of the energy line throughout the pipe network.

4 DEVELOPMENT OF A RATIONAL FORMULA PROCEDURE FOR THE ACT

A review of the differences between the procedures to estimate both runoff coefficients and times of concentration in the 1977 & AR&R, 1987s indicates that it is likely that different flowrates could be determined using the two documents.

To test the applicability of either procedure in the ACT this study sought to use the statistical interpretation of the Rational Formula to test estimated flowrates against recorded flowrates at the outlets of two urban gauged catchments in Canberra.

The comparison was undertaken by estimating flood frequency curves for the stormwater outflows from one urban catchment for a range of runoff coefficients and times of concentration determined using the various proposed procedures. The predicted flood frequency curves were then tested against the gauged frequency curve.

The adopted runoff coefficient and time of concentration procedures were then tested on an independent urban gauged catchment to verify their general applicability.

In Australia, the Rational Method has been mainly applied in a statistical manner where the Rational Formula has been rewritten in the following form:

$$q_p(Y) = \frac{1}{360} C \cdot I(t_c, Y) \qquad (7)$$

where

q_p	=	peak runoff rate per unit area for return period Y (Years)
C	=	coefficient of runoff
I	=	rainfall intensity (mm/h) for a return period Y Years and storm duration equal to the time of concentration, t_c
t_c	=	rainfall intensity average time
Y	=	return period (Years).

Over time there has been widespread criticism of the use of the Rational Formula. However, this criticism has been usually directed at the Rational Formula when it has been applied in its deterministic form (refer Equation 1). While the criticisms of the simplistic form of the model, including the assumption of uniform rainfall and the discounting of storage effects, etc (Aitken, 1975) are valid, the statistical formulation tends to significantly reduce these problems.

Using the procedures described by Aitken (1975), it is possible to estimate runoff coefficients from gauged flows in conjunction with Intensity-Frequency-Duration data in the following manner:

$$\text{INPUT} = \frac{1}{360} \, I(t_c, Y) \, A \qquad (8)$$

where

INPUT	=	calculated peak runoff for ARI of Y (Years) in m^3/s
$I(t_c, Y)$	=	rainfall intensity (mm/h) for ARI of Y Years and storm duration equal to the time of concentration
t_c	=	time of concentration (minutes).
A	=	catchment area (ha)

The value of the coefficient of runoff, C, is calculated from:

$$C = Q(Y) \,/\, \text{INPUT} \qquad (9)$$

where

$Q(Y)$	=	peak runoff for ARI of Y (Years) in m^3/s (abstracted from the gauged flood frequency curve).

Using this method it is possible to use stations with 10 years or more of recorded runoff data to derive statistical 'C' values.

It must be stressed, however, that the 'C' values calculated by the above method are directly related to the method adopted to calculate times of concentration, t_c. The estimation of time of concentration is further discussed in Section 4.2.

4.1 Estimation of Canberra Runoff Coefficients

Over the last 10 years the Rational Formula has been used extensively throughout Canberra to both estimate urban runoff design flow peaks as well as to analyse the effectiveness of existing systems. The Canberra stormwater system review of the capacity of the existing system in Belconnen is one example of the stormwater system studies conducted in recent years (Willing & Partners, 1983).

In all instances, the estimates of both the time of concentration, t_c, and runoff coefficient, C, were determined using the recommended procedures in the AR&R, 1958 and 1977.

The validity of both the past analyses and the revised procedures prescribed in AR&R, 1987 was tested by undertaking simulations of two urban catchments with gauging stations at their outlet. These catchments had been previously analysed using the RAT-HGL model. The two selected urban catchments were the Giralang and Mawson catchments.

The gauged flood frequency curves for these catchments were compared with flood frequency curves assembled from both the results of the previous studies as well as the results of the current study which tested the new Rational Formula procedures.

The Giralang urban catchment is shown on Figure 3. It has a total area of 85 hectares to the gauging station installed within the stormwater pipe system. It includes 19 hectares of impervious surfaces and 32 hectares of predominantly indigenous soil with unirrigated grassland. The residual 34 hectares consists of urban residential pervious area comprising lawns and gardens which are predominantly on imported topsoils. The overland flow from a further 8.5 hectares also discharges through the surface flow gauging station under Canopus Crescent. This additional outflow is only gauged once the pipe capacity is exceeded at events greater than approximately a 5 Yr ARI event.

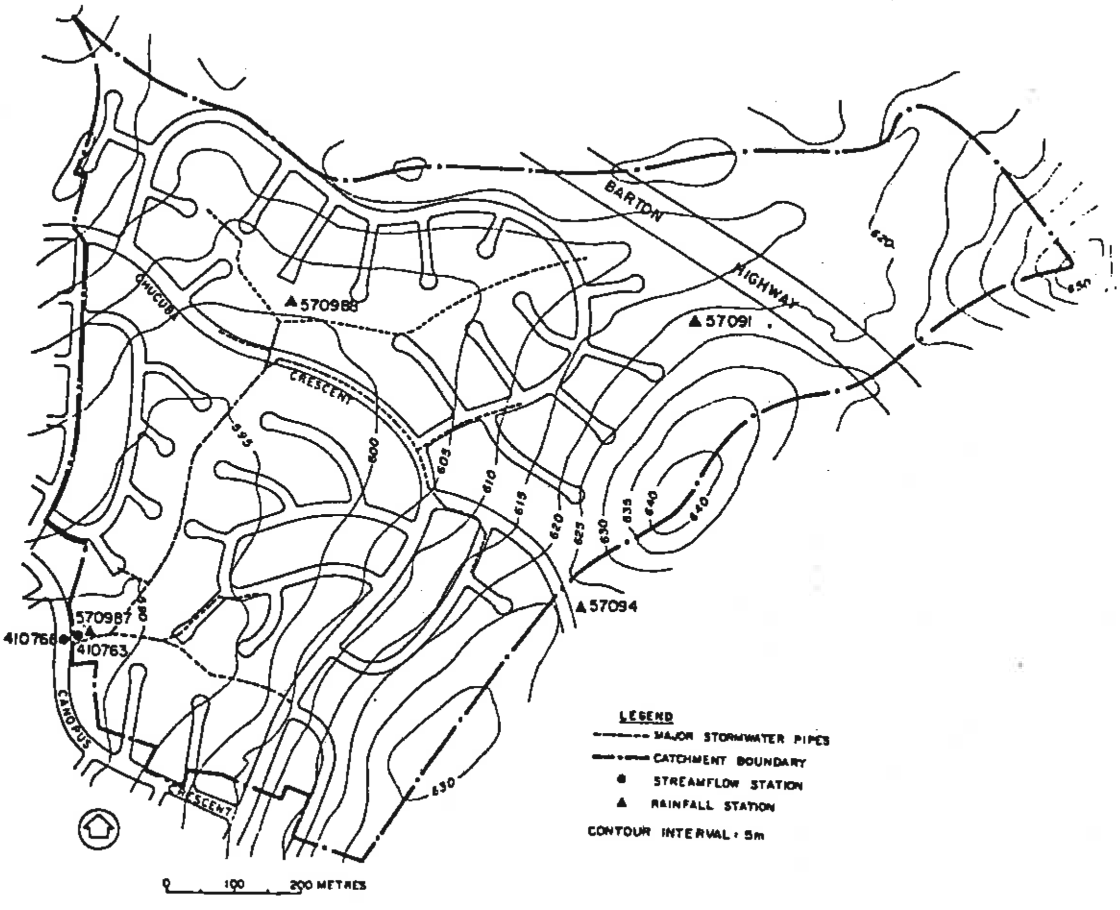

Figure 3 Giralang Urban Catchment

The first flood frequency curve considered was a curve based on the results of the original 1983 Belconnen study (Willing & Partners, 1983). Hence, the predicted flood frequency curve was based on the procedures laid down in AR&R, 1977. In the 1983 analysis, the RAT-HGL computer program (Messner & Goyen, 1985) was used to estimate the 1, 2, 5, 10, 20, 50 and 100 Yr ARI flood peaks throughout Belconnen including at the Giralang gauging station. The total catchment was discretised into 41 individual subareas to correctly represent the combined overland, gutter and pipe flow paths. Individual subareas were further subdivided into their respective impervious and pervious portions. Times of concentration of overland flows were estimated using Friend's equation (refer Equation 5). These times were added to appropriate gutter and pipe flow times which were determined using Manning's equation to obtain the total flow time through each of the 41 contributing subareas in the system.

A full description of the total watershed together with the link / node layout and input data for this catchment is provided in the 1983 study.

Flow times through the pipe system were estimated using the Darcy-Weisbach friction loss equation and the Colebrook-White equation for the determination of friction factors.

Runoff coefficients were computed by RAT-HGL and were, in the case of the AR&R, 1977 procedure, a function of the rainfall intensity (refer Figure 1) which was in turn a function of the total time of concentration to the node in question or, in the case of the AR&R, 1987 procedure, simply a function of the reference rainfall intensity, ${}^{10}I_1$, and the flood ARI.

The RAT-HGL model performed both a total area analysis as well as a critical area analysis above every node. All area subsets above the node were computed to estimate the maximum runoff peak. The critical analysis results were adopted for comparison with the gauged flood frequency curve.

In the case of the Giralang catchment, the critical (partial) area peak flowrates at the gauging station were approximately 33% higher than the total area peak flowrates. This phenomenon is probably due to the presence of a rural area at the top of the catchment.

The flood frequency curve based on partial and annual series analyses, using the Log Pearson Type III distribution, of the flowrates recorded at the Giralang station over the 10 years of record is presented in Figure 4. Also shown are a series of predicted flood frequency curves based on a variety of Rational Formula analyses which tested various combinations of the various runoff coefficient and time of concentration procedures.

Curve 1 on Figure 4 indicates the composite flood frequency curve based analyses of the gauged data using a Log-Pearson Type III distribution. The results of the partial series analysis was used to represent the values up to a 2 Yr ARI event while the results of the annual series analysis was adopted above the 5 Yr ARI event.

Curve 2 on Figure 4 represents the results of the 1983 study. This study was based predominantly on the classification of typical residential pervious areas as Curve 5 from Figure 1. Impervious runoff coefficients were set constant and equal to 0.9. Times of concentration for overland flow were computed using Friend's equation (refer Equation 5).

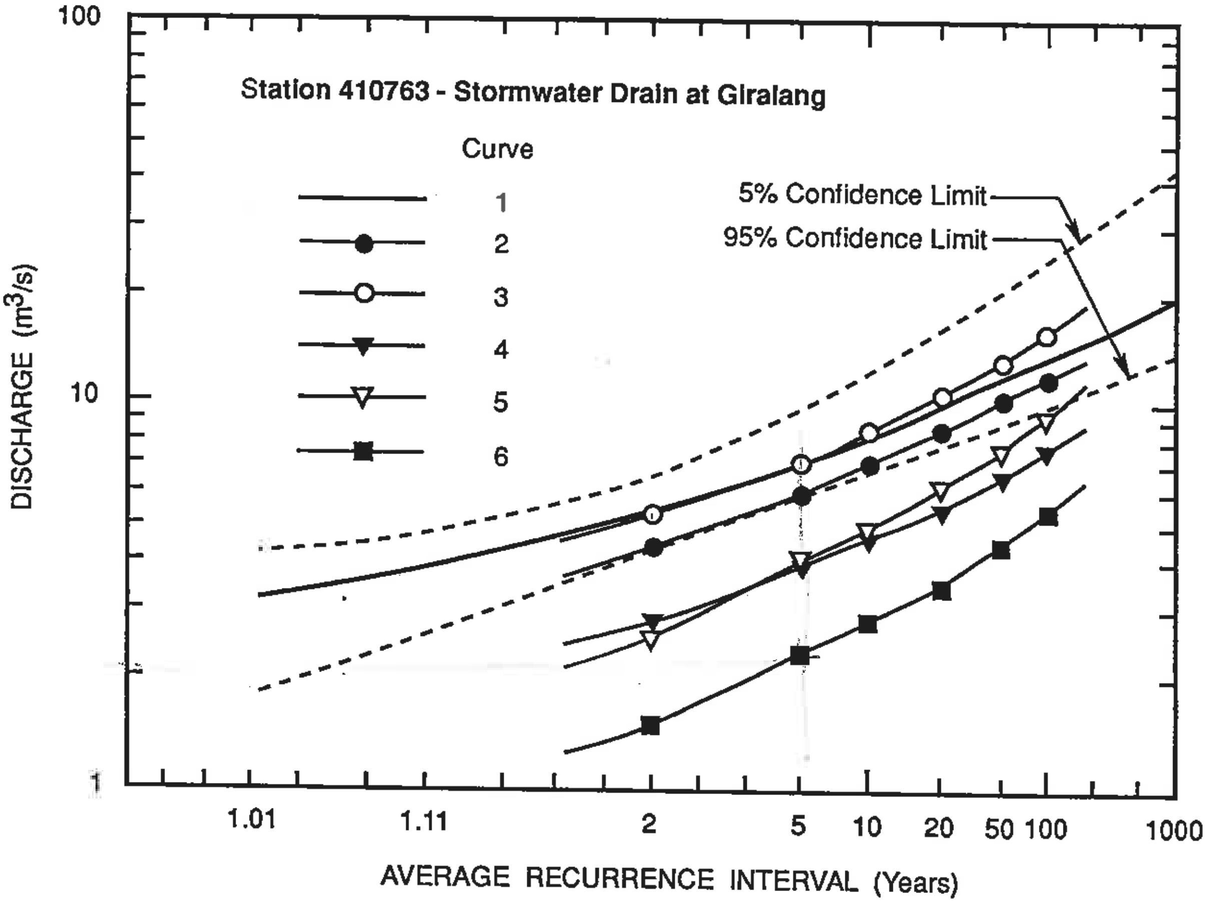

Figure 4 **Giralang Flood Frequency Curves**

Curve No.	Description	IFD procedure	Runoff coefficient procedure	Time of concentration procedure
1	Gauged data fitted to a Log Pearson Type III distribution			
2	1983 Study results	AR&R 1977	AR&R 1977 (Curve 5)	Friend's Eq
3	1983 Study results	AR&R 1977	AR&R 1977 (Curve 4)	Friend's Eq
4	1989 Study results	AR&R 1987	AR&R 1987	Friend's Eq
5	1989 Study results	AR&R 1987	AR&R 1977 (Curve 5)	Kinematic wave Eq
6	1989 Study results	AR&R 1987	AR&R 1987	Kinematic wave Eq

Table 4 **Giralang Flood Frequency Curve Number Descriptions**

Curve 3 on Figure 4 represents the 1983 study results with all pervious area runoff coefficients being computed using runoff coefficient Curve 4 rather than runoff coefficient Curve 5 (refer Figure 1).

Curve 4 employed the same catchment network layout as the 1983 study; runoff coefficients for pervious areas were estimated using the AR&R, 1987 procedure while overland flow times were determined using Friend's equation (refer Equation 5).

Curve 5 and 6 were obtained using the "kinematic wave" equation to calculate times of overland flow. A median surface roughness value of n* = 0.3 was adopted for pervious grassed areas in both simulations. Runoff coefficients were determined using the AR&R, 1977 and 1987 procedures for Curves 4 and 5, respectively.

The results presented in Figure 4 strongly support the continued use of the AR&R, 1977 runoff curve procedure with runoff coefficient Curve 4 being adopted for typical urban pervious (grassed) areas and the runoff coefficient from impervious areas being set constant and equal to 0.9.

The fully pervious runoff coefficients estimated using the AR&R, 1987 procedure (refer Figure 2) were based on a 10 Yr ARI, 1 hour duration rainfall intensity of 32.2 mm/h. These runoff coefficients predicted flowrates which were approximately 50% of the gauged flow rates (refer Curve 4 in Figure 4). This reduction in the predicted peak flowrate was even greater when the Kinematic Wave equation was used to determine overland flow times (compare Curves 4 and 6). In effect, the 5 Yr ARI peak flood discharge predicted using the AR&R, 1987 procedures was in fact equivalent to the gauged 1 Yr ARI peak flood discharge.

Based on the data presented in the "draft" AR&R, 1987 (the gauged catchment data points presented in Figure 2 were not published in the "final" AR&R, 1987) the data point for Yarralumla Creek at Mawson, Canberra indicates a fully pervious 10 YR ARI runoff coefficient of 0.60 rather than the value of 0.2 which was determined from the final version of Figure 2 presented in the AR&R, 1987.

The significant difference between the runoff coefficients determined using AR&R, 1977 and 1987 would in the main account for the 50% dicrepancy in flowrates highlighted in Figure 4 (refer Curves 3 and 4). Other small differences may have been caused by slight changes in IFD data.

It would appear that Figure 2 is not supported by either the previously reported Mawson data point or the results of the Giralang analysis carried out as part of this study.

This conclusion is supported by the independent research carried out by Goyen (1981) on both the Giralang and Mawson catchments which indicated a weighted runoff coefficient of around

0.5. This analysis was based on a single catchment statistical analysis which was based on total areas only. The reported areas for Giralang and Mawson were 94 and 445 hectares respectively which are some 10 to 13% higher than the current study area measurements. Making adjustments for the revised areas would give a weighted runoff coefficient of approximately 0.56 which aligns closely with the current analysis.

The results of the Giralang analysis which adopted Friend's equation to calculate overland flow time and runoff coefficient Curve 4 (from Figure 1) to estimate pervious area runoff coefficients are summarised in Table 5. In this analysis, all impervious subarea runoff coefficients were set equal to 0.9.

The Giralang analysis also indicated the importance of considering all potential critical area contributions above any particular node. Although the total catchment area above the in-pipe gauging station was 85.23 ha, the critical area was typically found to be between 70 and 72 ha depending on flood frequency. The total times of concentration of the critical area and the total area were approximately 19 and 38 minutes, respectively.

At the same time, the weighted runoff coefficient for the critical area and the total area were also determined and were found to be similar in magnitude (refer Tables 5 and 7). The values of the critical and total area weighted runoff coefficient given in Table 5 were determined by dividing the critical and total equivalent areas (CA) by the critical or total area derived using the RAT-HGL model. Hence, it was concluded that the partial area times of concentration have the greatest effect on the predicted peak flowrate rather than any differences between weighted runoff coefficients for partial or total areas.

ARI (Years)	Critical Area ΣC_p	ΣC_i	ΣC	Total ΣC	Critical Imp Area (ha)	Critical Area (ha)	Total Area (ha)	Crit t_c (mins)	Total t_c (mins)	Est Q (m^3/s)	Est Q_{tot} (m^3/s)	Gauge Q (m^3/s)
1	.51	.9	.61	.52	17.62	68.38	85.23	18.0	37.5	3.71	2.68	-
2	.57	.9	.65	.57	17.95	70.16	85.23	18.8	37.5	5.22	3.88	5.21
5	.62	.9	.69	.63	18.29	72.05	85.23	19.5	37.5	7.41	5.22	7.18
10	.64	.9	.70	.64	18.90	79.10	93.50	19.5	37.5	9.49	6.58	8.62
20	.67	.9	.72	.67	18.90	79.10	93.50	19.5	37.5	11.52	8.02	10.0
50	.69	.9	.74	.69	18.90	79.10	93.50	19.5	37.5	14.37	10.02	12.1
100	.71	.9	.75	.71	18.90	79.10	93.50	19.5	37.5	16.67	11.64	13.8

Table 5 Summary of Giralang Catchment Rational Method Results

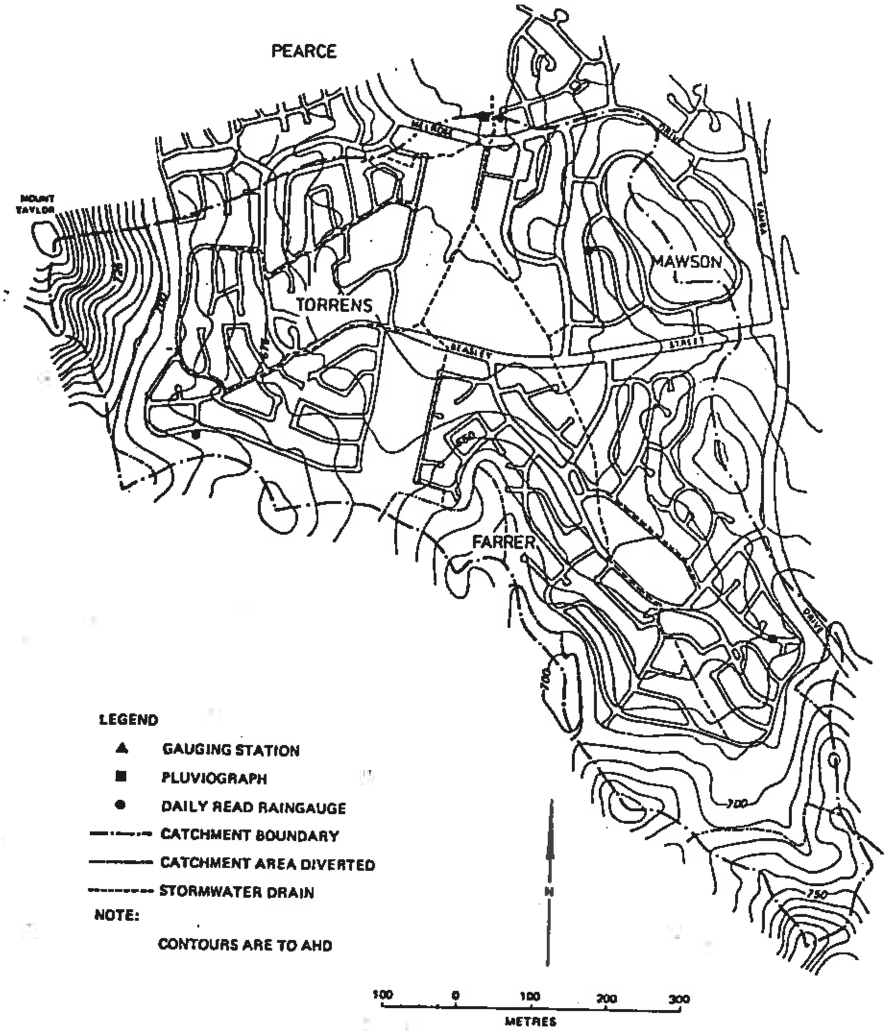

Figure 5 Mawson Urban Catchment

To further test the runoff coefficient estimating procedure recommended in AR&R, 1977 the same analysis procedure was applied to estimate flowrates from the gauged Mawson urban catchment.

The Mawson urban catchment, which is indicated in Figure 5, has a catchment area above the flow gauging station of between 382 and 400 hectares, depending on flood frequency. The impervious area represents approximately 25% of the catchment area. An additional catchment area also exists above the cutoff drains although this would only contribute flows to the gauging station during extremely rare events.

The average catchment slope is 2.5% and the general soils and grass cover superficially appear to be similar to the Giralang catchment.

The results from the analysis of the Mawson urban catchment are presented in Figure 6 and are summarised in Table 7.

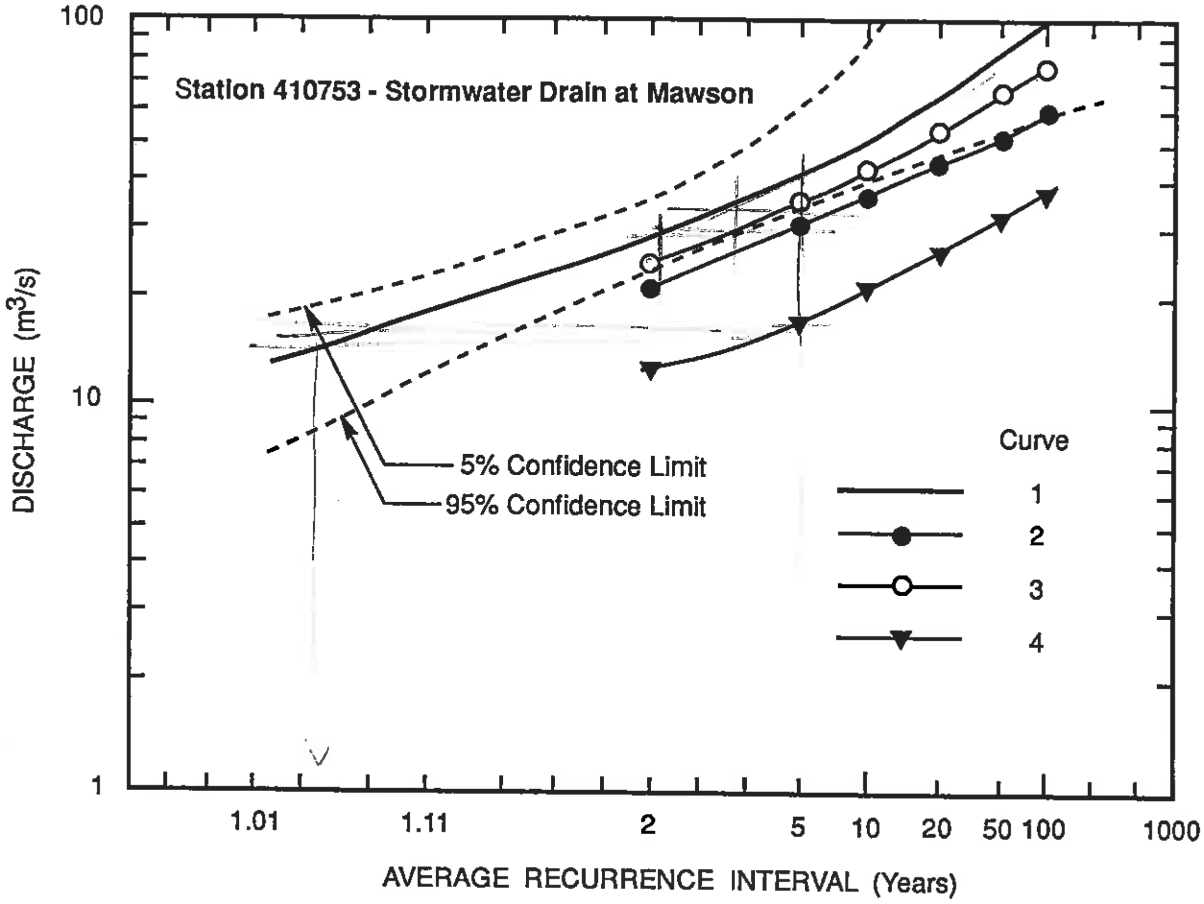

Figure 6 Mawson Flood Frequency Curves

Curve No.	Description	IFD procedure	Runoff coefficient procedure	Time of concentration procedure
1	Gauged data fitted to a Log Pearson Type III distribution			
2	1980 Study results	AR&R 1977	AR&R 1977 (Curve 5)	Friend's Eq
3	1980 Study results	AR&R 1977	AR&R 1977 (Curve 4)	Friend's Eq
4	1989 Study results	AR&R 1987	AR&R 1987	Friend's Eq

Table 6 Mawson Flood Frequency Curve Number Descriptions

ARI (Years)	Critical Area ΣC_p	ΣC_i	ΣC	Total ΣC	Critical Imp Area (ha)	Critical Area (ha)	Total Area (ha)	Crit t_c (mins)	Total t_c (mins)	Est Q (m^3/s)	Est Q_{tot} (m^3/s)	Gauge Q (m^3/s)
1	.50	.9	.59	.54	78.08	340.9	381.6	22.5	35.5	16.95	13.45	-
2	.56	.9	.63	.59	78.87	345.5	381.6	23.0	35.5	23.82	19.28	19.6
5	.61	.9	.68	.64	79.63	350.1	381.6	23.5	35.5	33.95	27.91	35.0
10	.63	.9	.69	.66	79.85	351.5	381.6	23.6	35.5	40.59	33.70	47.6
20	.66	.9	.71	.68	79.94	352.0	381.6	23.7	35.5	51.40	43.30	60.0
50	.68	.9	.73	.70	80.09	353.0	381.6	23.8	35.5	66.34	56.80	82.1
100	.70	.9	.74	.72	80.19	353.6	381.6	23.9	35.5	75.60	64.80	99.8

Table 7 Mawson Results Summary

As can be seen from Table 7 and Figure 6, the predicted results for Mawson are similar to the gauged flowrates and are well within the indicated confidence limits.

The Mawson results lend further support to the adoption of runoff coefficient Curve 4 from AR&R, 1977 for typical urban pervious (grassed) areas together with a constant runoff coefficient of 0.9 for the remaining impervious areas.

Alternatively, it may be possible to overcome the very poor performance of the AR&R, 1987 procedure by adopting an amended line which passes through the Mawson data point plotted in Figure 2. This alternative is not recommended at the current time because of the uncertainty which has arisen regarding the AR&R, 1987 runoff coefficient procedure due to both the poor agreement between the proposed interpolation procedure and the previously reported (but unpublished) six runoff coefficients and the confirmation of the plotted Mawson runoff coefficient by the Giralang catchment analysis. This study has highlighted the need to confirm the reported runoff coefficients for both Melbourne and Sydney and the need to obtain further data to better calibrate the AR&R, 1987 procedure.

As already discussed, the preceding analysis has been based on estimating times of overland flow using Friend's equation. AR&R, 1987 however, recommends the use of the 'kinematic wave' equation to calculate the time of overland flow instead of the empirical AR&R, 1977 procedure. The impact of the adopted procedure for the determination of the time of overland flow was also investigated and is discussed below.

4.2 Estimation of Time of Concentration

In order to apply the Rational Formula it is necessary to determine the time of concentration to be used to determine the rainfall intensity to be input into the Rational Formula calculation. AR&R, 1958 and 1977 provided a nomograph of time of overland flow to allow drainage engineers to determine the time of concentration for the overland phase of runoff in urban areas (IEAust., 1958 and 1977).

A formula and values of "n" were added by JA Friend in 1954 to the source data attributed to the US Department of Agriculture in 1942 (refer Equation 5). Reported "n" values were constant for each category of surface and ranged from 0.015 for a paved surface to 0.060 for a densely grassed surface.

If a natural surface longer than 1000 m contributes to the runoff from an urban area then AR&R, 1977 recommended that the Bramsby-Williams formula be used. This equation is given by:

$$t_o = \frac{FL}{A^{0.1}S^{0.2}} \qquad (10)$$

where

	F	=	a factor of proportionality related to the units of catchment area
		=	58.5 for A in km^2
		=	92.7 for A in ha
	L	=	mainstream length (km)
	A	=	catchment area
	S	=	mainstream slope (m/km).

In this case the time of overland flow includes travel time in natural channels and in some instances may be equivalent to the time of concentration (I.EAust., 1977).

AR&R, 1987 however, dispensed with the previously adopted nomograph for time of overland flow and instead proposed the 'kinematic wave' equation for overland flow times described by Ragan and Duru (1972) (refer Equation 6). Typical values of surface roughness reported by Woolhiser (1975) were also provided but no guidance was given on the selection of appropriate roughness values. These typical values are presented in Table 8. These typical values are similar to the roughness coefficients reported by Engman (1986).

In contrast, Argue, 1986 simply recommends the adoption of $n^* = 0.015$ for paved surfaces, $n^* = 0.25$ for lawns and $n^* = 0.50$ for thickly grassed surfaces.

Surface Type	Roughness Coefficient, n*
Concrete or Asphalt	0.010 - 0.013
Bare Sand	0.010 - 0.016
Gravelled Surface	0.012 - 0.030
Bare Clay - Loam Soil (eroded)	0.012 - 0.033
Sparse Vegetation	0.053 - 0.130
Short Grass Prairie	0.100 - 0.200
Lawns	0.170 - 0.480

Table 8 Surface Roughness Values

Since Equation 6 includes rainfall intensity as a variable, it must be solved iteratively using the local intensity-frequency-duration (IFD) curves. It should also be noted that the definition of slope is different in each of the overland flow equations.

4.2.1 Comparison of Predicted Overland Flow Times

A comparison of the times of overland flow obtained using the previous nomograph (Equation 5) and the new 'kinematic wave' equation (Equation 6) for urban catchments in **Canberra** has been undertaken. The results of a comparison of overland flow times for a paved surface and grassed surfaces are presented in Figures 7 and 8.

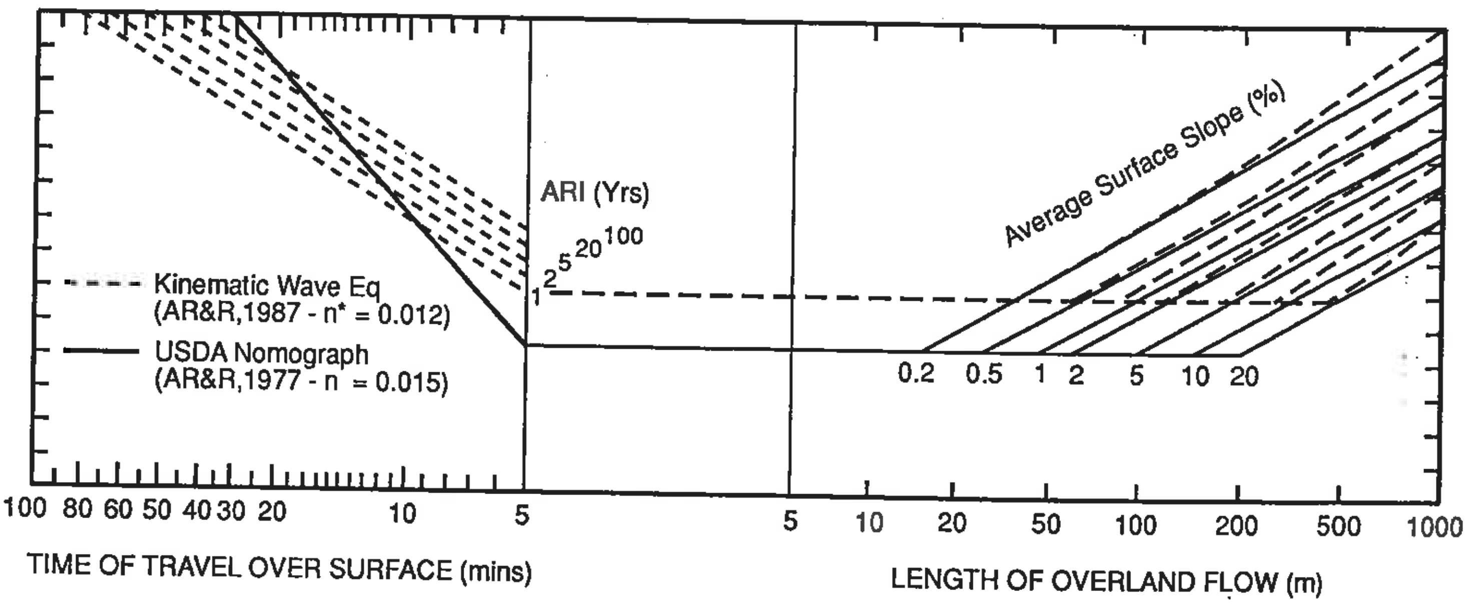

Figure 7 Comparison of Predicted Times of Overland Flow for Paved Surfaces in Canberra

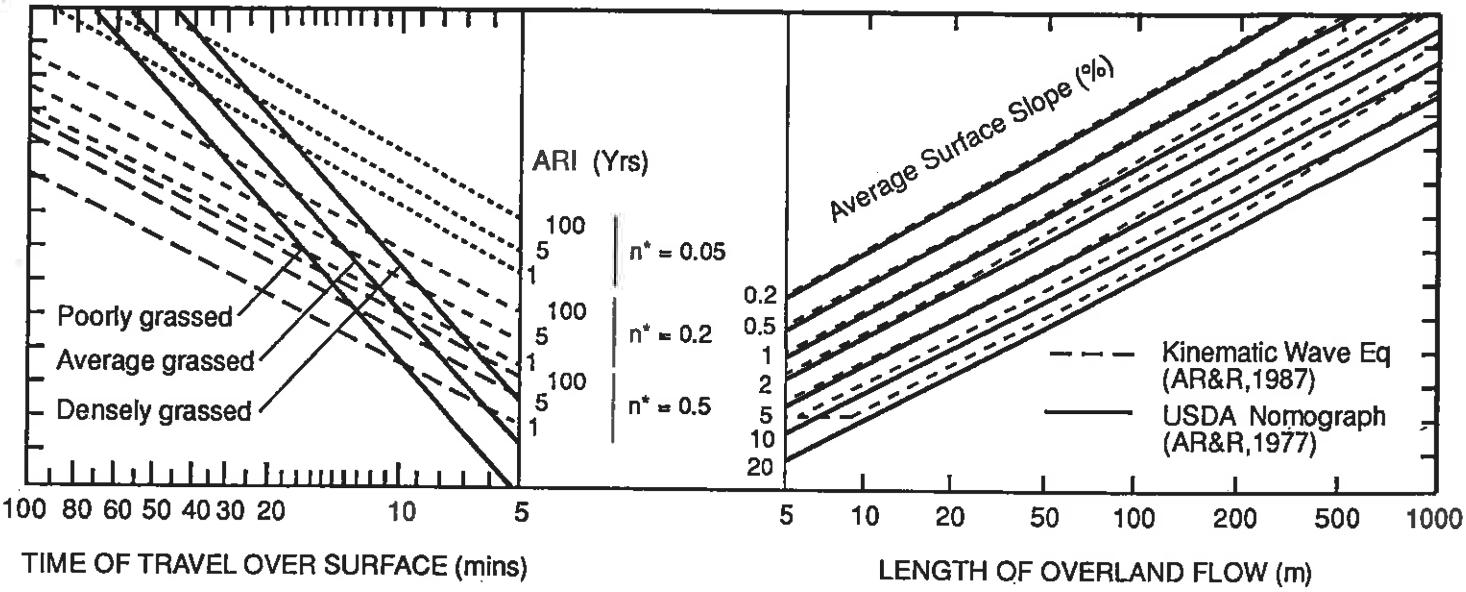

Figure 8 **Comparison of Predicted Times of Overland Flow for Grassed Surfaces in Canberra**

It is evident in both Figures 7 and 8 that the "kinematic wave" equation and a constant value of surface roughness may give overland flow times which are significantly different from the overland flow times predicted using Friend's equation.

In the case of densely grassed surfaces, the time of overland flow predicted by the "kinematic wave" equation (refer curves for n* = 0.5) is likely to be significantly longer than the time of overland flow predicted by Friend's equation (refer "densley grassed" curve). Conversley, the time of overland flow predicted by the "kinematic wave" equation (refer curves for n* = 0.05) for poorly grassed surfaces may be shorter than the time of overland flow predicted by Friend's equation (refer "poorly grassed" curve).

In the case of pervious grassed surfaces (lawns) in the Giralang catchment, a median surface roughness value of n* = 0.3 was adopted when using the "kinematic wave" equation to determine times of overland flow (refer Table 8 - lawns). It was found that the times of overland flow predicted by the "kinematic wave" equation and the adopted value of the surface roughness value were significantly longer than the times of overland flow predicted by Friend's equation for low return periods. At high return periods, the flow times estimated using the two procedures converged to similar travel times. When used to assess rainfall intensities for the Rational Formula calculations, these increased times of overland flow gave rise to peak flowrate estimates which were significantly lower than the peak flowrates predicted when using Friend's equation (compare Curves 2 & 5 and Curves 4 & 6 in Figure 4).

4.2.2 Surface roughness values for overland flow

In the same way that flow in pipes is characterised by three flow regimes, overland flow also exhibits three distinct flow regimes (Morgali, 1970). These regimes are laminar flow, transitional (turbulent) flow and fully turbulent flow. It is expected that a turbulent flow regime would correspond to a constant Manning roughness value. This is supported by Ragan and Duru (1972). For paved surfaces and grassed surfaces, Ragan and Duru (1972) report constant 'n' values of 0.013 and 0.05 respectively where the product of rainfall intensity (I) and overland flow length (L), IL (imperial units), is greater than 500. In the case of a well grassed surface the reported roughness value of 0.05 (for IL>500) is significantly lower than the range of roughness values for lawns reported in Table 8.

Conversely, the surface roughness value (n^*) is expected to vary under a laminar flow regime. It is expected that at low Reynolds Numbers (equivalent to low IL values) that the surface roughness value will be high. It is also expected that the surface roughness will decrease as Reynolds Number increases until a turbulent flow regime is established and a constant surface roughness value is approached. It is also expected that the range of roughness values for grassed and irregular surfaces will be far greater than for paved surfaces.

An indication of the expected trend in surface roughness values for well grassed surfaces may be derived from Figure 8. Using the "densley grassed" nomograph curve as a guide, it is evident that at short times of concentration that high surface roughness values are needed to match the curve (ie. laminar flow regime) while at longer times of concentration a lower value of surface roughness is required if the time of overland flow predicted by the 'kinematic wave" formulation is to approach the nomograph curve.

At present it is not possible to quantify the variation of surface roughness as a function of catchment and rainfall properties due to the difficulty in obtaining local or overseas data. It is also likely that theoverland flow regimes which will be typically encountered when designing and analysing urban drainage in Canberra will be characterised by varying surface roughnesses. Hence, it is very difficult to give guidance on the selection of surface roughness values which are expected to be a function of the type of surface, rainfall intensity and the length of overland flow. Likewise, Argue, 1986 noted that it is a matter of field observation that sheet flow rarely progresses more than 200 m before entering a runnel or rill giving rise to channel flow.

In view of the significant impact assumed values of the surface roughness can have on the calculated times of concentration and in turn on the predicted peak flowrates and the lack of guidance on the limit of applicability of the "kinematic wave" formulation, it is recommended that the AR&R, 1977 procedure be adopted until further guidance is given as to the selection of appropriate surface roughness values for the "kinematic wave" equation.

4.2.3 Determination of Time of Concentration from gauged rainfall/runoff data

In hydrograph analysis, the lag time is defined as the time from the centre of mass of the excess rainfall to the peak rate of runoff. This time difference can be determined by analysing hydrographs from historical storm events.

From the literature (U.S.S.C.S., 1975) and based on studies of many storm events for a range of watershed conditions, an empirical relationship has been found which links the time of concentration with the lag time. This equation is as follows:

$$\text{Lag} = 0.6\ t_c \qquad (11)$$

Using this relationship a number of historical storm events were examined from both the Giralang and Mawson catchments. The results determined from this analysis are presented in Table 9.

As is shown in Table 9, the times of concentration for the Giralang and Mawson catchments were approximately 20 minutes and 21 minutes respectively.

There were difficulties in determining lag times with any great accuracy due to the coarseness of the gauge plots in relation to the short lags being examined. As is indicated in Table 9, a number of the events from the gauge records exhibited a zero or even negative lag. These particular events also highlighted the difficulty in obtaining lags from the records of dynamic storm events on small to medium sized urban catchments recorded by a sparse network of gauges. These anomalous events may also reflect timing problems with recorders which can occur from time to time.

GIRALANG			MAWSON		
Event Date	Estimated Lag (mins)	t_c (mins)	Event Date	Estimated Lag (mins)	t_c (mins)
09.11.80	12	20	07.04.77	0	
03.12.80	10	20	09.01.78	-6	
18.02.81	10	17	20.03.78	14	23
24.03.82	-12		23.03.78	12	20
13.12.83	0		06.01.81	12	20
25.03.84	12	20			

Table 9 Estimated and Predicted Times of Concentration

The deduced times of concentration presented in Table 9 were compared with estimated times of concentration. It was found that the deduced times of concentration were in best agreement with the"critical" area times of concentration reported in Tables 5 and 7; these times of concentration were obtained using Friend's equation.

Based on a comparison of deduced and predicted times of concentration, it is recommended that Friend's equation (Equation 5) be adopted for the determination of the time of overland flow pending further clarification of the use of the kinematic wave approach proposed in AR&R, 1987. In particular, clarification is required of the sensitivity of surface roughness values to the type of surface, the rainfall intensity and the length of overland flow.

5 ACCEPTANCE CRITERIA FOR RATIONAL FORMULA PROCEDURES WITHIN THE ACT

5.1 General

To test the acceptability of flow estimating procedures including Rational Formula procedures which may be proposed for use within the ACT, it will be necessary to develop a set of criteria on which to base the acceptance or rejection of proposed procedures to estimate design flows for new urban developments.

At the same time, it will be advisable to inform developers and drainage engineers in the ACT of the already accepted modelling procedures along with guidelines to the relevant parameters suitable for Canberra. This study has sought to develop guidelines for Rational Formula procedures. It is also intended that a subsequent study will investigate acceptance criteria and rainfall loss rates and other rainfall/runoff model parameters for hydrograph estimation procedures.

It is suggested that a typical urban catchment, eg. the Giralang catchment ,be described in detail and that relevant topographical and hydraulic details be made available to enable potential models including Rational Formula models to be applied in the estimation of a flood frequency curve for the catchment.

This data could be supplied for appropriate model testing when and if required. It is suggested that once the model is calibrated to the supplied catchment data that the model under examination be applied to the Mawson catchment to validate the adopted modelling procedures and model parameters. With this aim in mind, a detailed description of the Giralang urban catchment is presented below.

5.2 Description of Giralang Catchment for Numerical Modelling Purposes

The location of the Giralang urban catchment is shown on Figure 9.

A detailed description of the catchment is further shown on Figure 10 indicating the road and block layout together with the pipe drainage system.

It has a total area of 85 hectares to the gauging station installed within the stormwater pipe system. It includes 19 hectares of impervious surfaces and 32 hectares of predominantly indigenous soil with unirrigated grassland. The residual 34 hectares consists of urban residential pervious area comprising lawns and gardens which are predominantly on imported topsoils. The overland flow from a further 8.5 hectares also discharges through the surface flow gauging station under Canopus

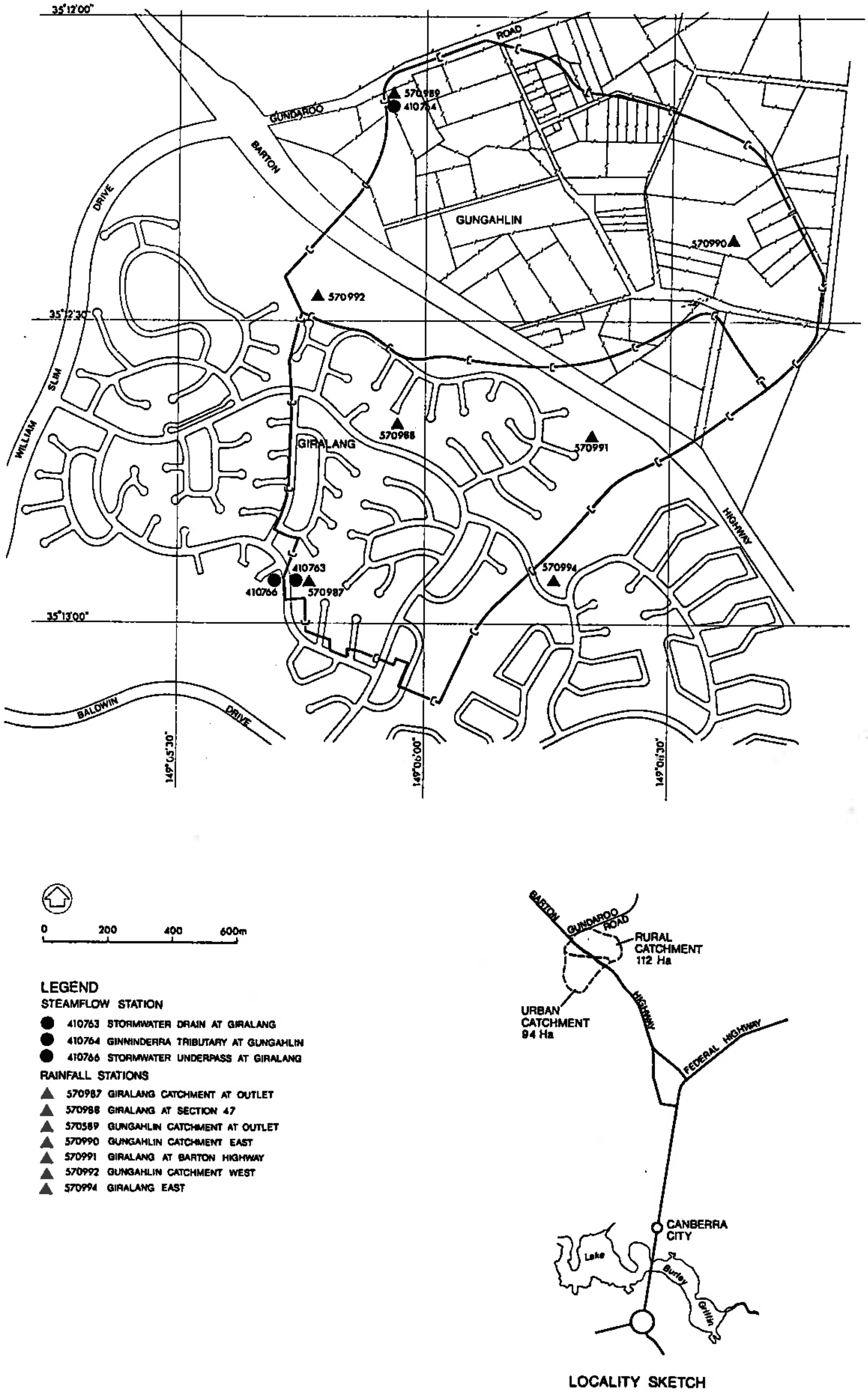

Figure 9 Location of Giralang Urban Catchment

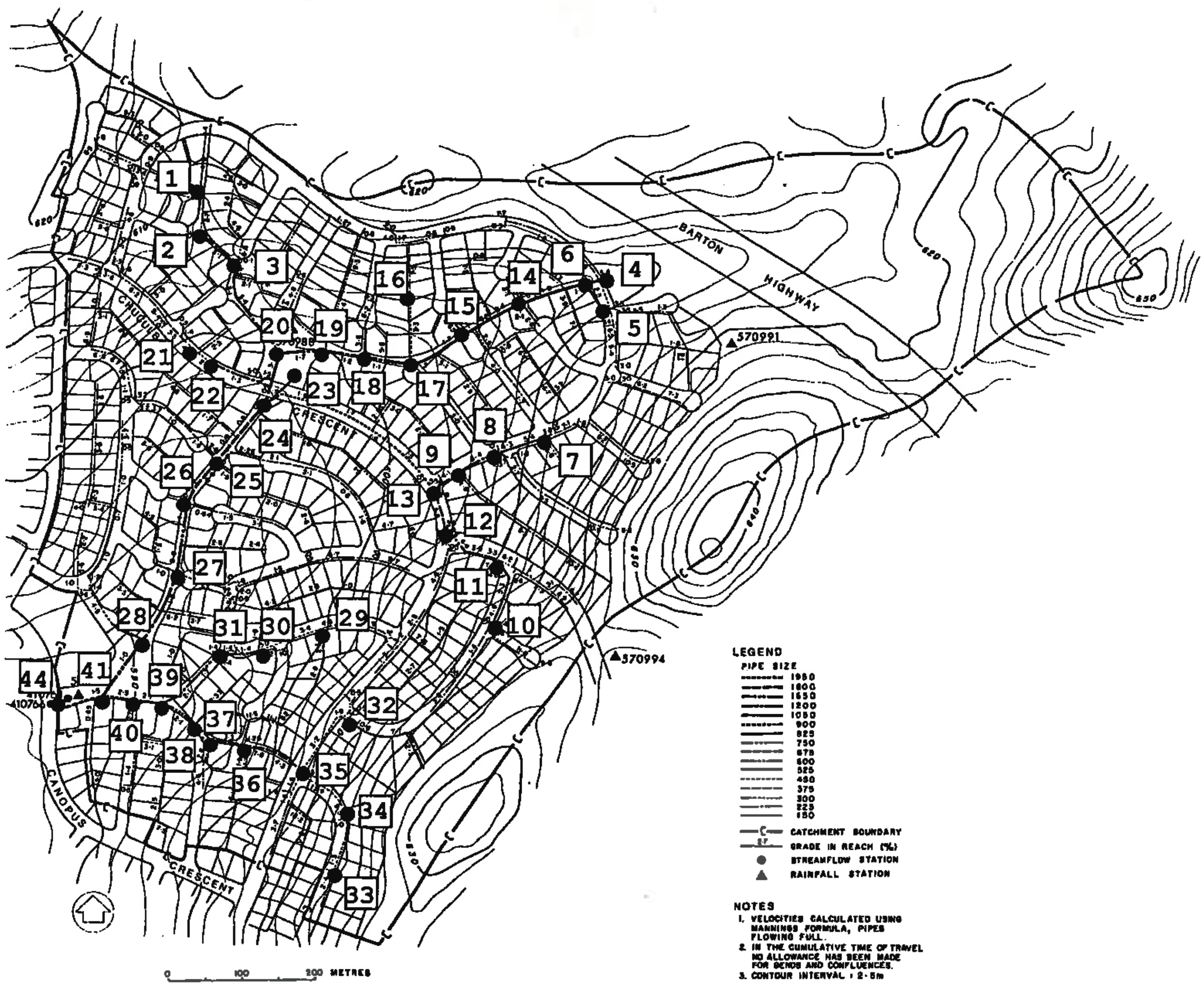

Figure 10 Giralang Urban Catchment

Crescent. This additional outflow is only gauged once the pipe capacity is exceeded at events greater than approximately a 5 Yr ARI event.

Indigenous areas have native grass cover similar to the paired Gungahlin rural catchment with some imported species, predominantly chewings fescue and ryes sown in the areas adjacent to residential lots. Grasses common to residential lots are chewings fescue, couch, kentucky blue and various species of clover. Root depths are usually limited to less than 50 mm.

Native soil types consist basically of red podzolic on the upper slopes and yellow podzolics on the lower slopes of the catchment. The 'A' horizon of the native topsoils or root zone consists mainly of sandy clay to clayey-sand of low plasticity which varies in thickness from virtually zero on some of the upper slopes where the weathered rock is exposed to 400 mm at the bottom of the catchment close to ephemeral watercourses. Imported topsoils vary considerably in nature, from light sandy-clays to puggy organic clays depending on the area of procurement.

A detailed description of the Giralang gauging network is given in Technical Paper No. 29 published by the National Capital Development Commission in 1980. Instrumentation, as described in this above document, consists of a runoff recording station at the outlet of the catchment comprising a sloping crest crump weir to measure flows in the pipe system and an additional cut throat flume incorporated in a walkway underpass to measure excess overland flows up to at least the 100 Year ARI flow.

Also included are five rainfall stations either in or just outside the catchment area. They consist of various types of pluviographs which are capable of monitoring variations in storm patterns across the catchment. Additionally, two rainfall stations are located within the adjacent rural Gungahlin catchment no more than 600 metres north of the border of the Giralang catchment (refer Figure 9) The location of all gauges are shown on Figure 3-4 in Technical Paper No. 29.

Urbanisation of the Giralang catchment commenced in 1974 and was completed by late 1976. Tree planting was progressively implemented from early 1976 and was virtually complete by late 1977.

Rainfall and runoff data is available from 1976 on both the Gungahlin rural and Giralang urban catchments upon request for use in model calibration.

Figure 10 also indicates the nodes locations adopted for the RAT-HGL model data set. Table 10 provides the RAT-HGL data set which indicates appropriate sub-catchment parameters based around a Rational Formula network model.

Table 10 RAT-HGL Giralang Data Set

```
    0
'Giralang catchment 21'
! ctype , units
  1977      0
! ARR 1977 runoff curves coefficients
5.81      4.46      3.22      3.140      2.910      2.310      2.090      1.940
 .47      0.96      0.70      0.594      0.483      0.376      0.313      0.266
! ARR 1987 runoff curve definition ie intensity 10 yr 1hr duration
!  32.2
!  IFD option
    1987
! ARR1987 IFD parameters
22.0  4.3   1.14   43.0   8.0   2.25   0.24   4.28    15.55
! run mode   design freq.    no. of plots
    1          0               0
!  required return periods to be analysed
   1    2    5   10   20   50  100
!no. of sub areas
     51
!imp. area  tot area   c curve  sub area tc  tc to node   node repeat
     0.81      3.02        4       19.0       0           'N1' 1
     0.20      0.30        4        8.0       0           'N2' 1
     0.45      0.67        4       14.0       0           'N3' 1
     0.00     16.90        4       34.0       0           'N4' 1
     0.67      3.02        4       17.0       0           'N5' 1
     0.30      1.03        4       13.3       0           'N6' 1
     0.43      2.27        4       15.0       0           'N7' 1
     0.37      2.76        4       17.0       0           'N8' 1
     0.20      0.20        4        6.0       0           'N9' 1
     0.42      1.47        4       13.0       0          'N10' 1
     0.67      2.25        4       14.0       0          'N11' 1
     0.30      1.22        4       17.0       0          'N12' 1
     0.22      1.33        4       19.0       0          'N13' 1
     0.55      0.96        4       12.0     0.2          'N14' 1
     0.77      1.44        4       18.0     0.2          'N15' 1
     0.30      1.20        4       17.0     0.3          'N16' 1
     0.48      1.14        4       18.0       0          'N17' 1
     0.25      0.46        4       13.0     0.2          'N18' 1
     0.50      1.10        4       14.0     0.3          'N19' 1
     0.20      0.20        4       12.0       0          'N20' 1
     0.91      2.63        4       22.0       0          'N21' 1
     0.11      0.37        4       12.0       0          'N22' 1
     0.18      8.01        4       21.0       0          'N23' 1
     0.96      2.99        4       18.0       0          'N24' 1
     1.69      4.74        4       22.0     0.2          'N25' 1
     0.67      3.75        4       23.0       0          'N26' 1
     1.08      3.13        4       16.0       0          'N27' 1
     1.17      3.00        4       17.0       0          'N28' 1
     0.90      2.03        4       14.0       0          'N29' 1
     0.38      1.24        4       15.0       0          'N30' 1
     0.14      0.63        4       15.0       0          'N31' 1
     0.22      2.24        4       14.0       0          'N32' 1
     0.13      0.42        4       11.0       0          'N33' 1
     0.19      1.02        4       13.5       0          'N34' 1
     0.63      2.33        4       14.0       0          'N35' 1
     0.33      0.69        4       13.0       0          'N36' 1
     0.34      0.78        4       15.0     0.1          'N37' 1
     0.10      0.10        4        6.0       0          'N38' 1
     0.31      0.74        4       17.0     0.1          'N39' 1
     0.48      1.25        4       18.0     0.6          'N40' 1
     0.20      0.20        4        6.0       0          'N41' 1
     0.11      6.95        4       24.0       0          'N42' 1
     0.42      1.22        4       17.0       0          'N43' 1
     0.10      0.10        4        6.0       0          'N44' 1
     0.43      1.84        4       17.5     0.3          'N45' 1
     0.57      1.81        4       15.0       0          'N46' 1
     0.21      0.86        4       16.0       0          'N47' 1
```

```
      0.88      3.72        4     17.3      0          'N48' 1
      0.33      1.25        4     16.3      0          'N49' 1
      0.10      0.10        4      6.0      0          'N50' 1
      0.29      0.76        4     15.0     0.2         'N51' 1
 !no. nodes  visc     ftol    htol
   56         0         5     0.05
 !                                  Sequential node nos. reference only
 !ulable   dlable   pcap    ofp(1)   olab(1)
 'N1'      'N2'     0.49    /                          !    1 ----> 2
 ! rtt
   0.20     999 /
 !
 !ulable   dlable   pcap    ofp(1)   olab(1)
 'N2'      'N3'     0.63     1       'N20'     2 /     !    2 ---->  3
 ! rtt
   0.36     999 /
 !
 !ulable   dlable   pcap    ofp(1)   olab(1)
 'N3'      'N20'    0.70    /                            !    3 ----> 20
 ! rtt
   0.50     999 /
 !
 !ulable   dlable   pcap    ofp(1)   olab(1)
 'N4'      'N6'     2.80    /                            !    4 ----> 6
 ! rtt
   0.03     999 /
 !
 !ulable   dlable   pcap    ofp(1)   olab(1)
 'N5'      'N6'     0.47    /                            !    5 ----> 6
 ! rtt
   0.21     999 /
 !
 !ulable   dlable   pcap    ofp(1)   olab(1)
 'N6'      'N14'    3.51    /                            !    6 ----> 14
 ! rtt
   0.37     999 /
 !
 !ulable   dlable   pcap    ofp(1)   olab(1)
 'N7'      'N8'     0.43    0.15     'N15'    2  /       !     7 ----> 8
 ! rtt
   0.23     999 /
 !
 !ulable   dlable   pcap    ofp(1)   olab(1)
 'N8'      'N9'     0.80    0.15     'N17'    2    /     !     8 ----> 9
 ! rtt
   0.25     999 /
 !
 !ulable   dlable   pcap    ofp(1)   olab(1)
 'N9'      'N13'    0.86    /                            !    9 ----> 13
 ! rtt
   0.13     999 /
 !
 !ulable   dlable   pcap    ofp(1)   olab(1)
 'N10'     'N11'    0.43    /                            !    10 ----> 11
 ! rtt
   0.34     999 /
 !
 !ulable   dlable   pcap    ofp(1)   olab(1)
 'N11'     'N12'    0.68    /                            !    11 ----> 12
 ! rtt
   0.27     999 /
 !
 !ulable   dlable   pcap    ofp(1)   olab(1)
 'N12'     'N13'    0.97    /                            !    12 ----> 13
 ! rtt
   0.17     999 /
 !
 !ulable   dlable   pcap    ofp(1)   olab(1)
 'N13'     'N24'    2.08    /                            !    13 ----> 24
 ! rtt
   0.88     999 /
```

```
!
!ulable  dlable  pcap   ofp(1)  olab(1)
'N14'    'N15'   3.74   /                                 !   14 ----> 15
! rtt
  0.32   999 /
!
!ulable  dlable  pcap   ofp(1)  olab(1)
'N15'    'N17'   3.74   /                                 !   15 ----> 17
! rtt
  0.29   999 /
!
!ulable  dlable  pcap   ofp(1)  olab(1)
'N16'    'N17'   0.37   /                                 !   16 ----> 17
! rtt
  0.37   999 /
!
!ulable  dlable  pcap   ofp(1)  olab(1)
'N17'    'N18'   4.30   /                                 !   17 ----> 18
! rtt
  0.30   999 /
!
!ulable  dlable  pcap   ofp(1)  olab(1)
'N18'    'N19'   4.30   /                                 !   18 ----> 19
! rtt
  0.19   999 /
!
!ulable  dlable  pcap   ofp(1)  olab(1)
'N19'    'N20'   4.68   /                                 !   19 ----> 20
! rtt
  0.21   999 /
!
!ulable  dlable  pcap   ofp(1)  olab(1)
'N20'    'N24'   5.53    1.0     'N23'     2 /            !   20 ----> 24
! rtt
  0.26   999 /
!
!ulable  dlable  pcap   ofp(1)  olab(1)
'N21'    'N22'   0.46   /                                 !   21 ----> 22
! rtt
  0.17   999 /
!
!ulable  dlable  pcap   ofp(1)  olab(1)
'N22'    'N24'   0.52   /                                 !   22 ----> 24
! rtt
  0.50   999 /
!
!ulable  dlable  pcap   ofp(1)  olab(1)
'N23'    'N24'   25.0   /  !1.0     'N42'    2   /        !   23 ----> 24
! rtt
  0.01   999 /
!
!ulable  dlable  pcap   ofp(1)  olab(1)
'N24'    'N25'   8.56   /                                 !   24 ----> 25
! rtt
  0.30   999 /
!
!ulable  dlable  pcap   ofp(1)  olab(1)
'N25'    'N26'   9.27   /                                 !   25 ----> 26
! rtt
  0.24   999 /
!
!ulable  dlable  pcap   ofp(1)  olab(1)
'N26'    'N27'   9.78   /                                 !   26 ----> 27
! rtt
  0.33   999 /
!
!ulable  dlable  pcap   ofp(1)  olab(1)
'N27'    'N28'   9.58   /                                 !   27 ----> 28
! rtt
  0.37   999 /
!
```

```
!ulable  dlable  pcap   ofp(1)  olab(1)
'N28'    'N41'   9.58   /                                          !   28 ----> 41
! rtt
  0.31   999 /
!
!ulable  dlable  pcap   ofp(1)  olab(1)
'N29'    'N30'   0.43   /                                          !   29 ----> 30
! rtt
  0.50   999 /
!
!ulable  dlable  pcap   ofp(1)  olab(1)
'N30'    'N31'   0.72   /                                          !   30 ----> 31
! rtt
  0.32   999 /
!
!ulable  dlable  pcap   ofp(1)  olab(1)
'N31'    'N39'   0.71   /                                          !   31 ----> 39
! rtt
  0.59   999 /
!
!ulable  dlable  pcap   ofp(1)  olab(1)
'N32'    'N35'   0.38   /                                          !   32 ----> 35
! rtt
  0.38   999 /
!
!ulable  dlable  pcap   ofp(1)  olab(1)
'N33'    'N34'   0.25   /                                          !   33 ----> 34
! rtt
  0.54   999 /
!
!ulable  dlable  pcap   ofp(1)  olab(1)
'N34'    'N35'   0.34   /                                          !   34 ----> 35
! rtt
  0.22   999 /
!
!ulable  dlable  pcap   ofp(1)  olab(1)
'N35'    'N36'  1.41   /                                           !   35 ----> 36
! rtt
  0.23   999 /
!
!ulable  dlable  pcap   ofp(1)  olab(1)
'N36'    'N37'   1.60   /                                          !   36 ----> 37
! rtt
  0.15   999 /
!
!ulable  dlable  pcap   ofp(1)  olab(1)
'N37'    'N38'   1.65   /                                          !   37 ----> 38
! rtt
  0.10   999 /
!
!ulable  dlable  pcap   ofp(1)  olab(1)
'N38'    'N39'   1.67   /                                          !   38 ----> 39
! rtt
  0.24   999 /
!
!ulable  dlable  pcap   ofp(1)  olab(1)
'N39'    'N40'   2.48   /                                          !   39 ----> 40
! rtt
  0.10   999 /
!
!ulable  dlable  pcap   ofp(1)  olab(1)
'N40'    'N41'   2.69   /                                          !   40 ----> 41
! rtt
  0.16   999 /
!
!ulable  dlable  pcap   ofp(1)  olab(1)
'N41'    'N44'  12.69    1.0     'N42'     2  /                    !   41 ----> 44
! rtt
  0.23   999 /
!
!ulable  dlable  pcap   ofp(1)  olab(1)
```

```
'N42'     'N56'    /                                              !    42 ----> 56
! rtt
/
!
!ulable  dlable  pcap   ofp(1)  olab(1)
'N43'    'N44'   0.45    1.0     'N45'     2  /                   !    43 ----> 44
! rtt
  0.55    999 /
!
!ulable  dlable  pcap   ofp(1)  olab(1)
'N44'    'N45'   8.97    1.0     'N56'     2  /                   !    44 ----> 45
! rtt
  0.29    999 /
!
!ulable  dlable  pcap   ofp(1)  olab(1)
'N45'    'N46'   8.97    1.0     'N56'     2  /                   !    45 ----> 46
! rtt
  1.00    999 /
!
!ulable  dlable  pcap   ofp(1)  olab(1)
'N46'    'N56'   /                                                !    46 ----> 56
! rtt
/
!
!ulable  dlable  pcap   ofp(1)  olab(1)
'N47'    'N48'   0.34    1.0     'N49'     2  /                   !    47 ----> 48
! rtt
  0.14    999 /
!
!ulable  dlable  pcap   ofp(1)  olab(1)
'N48'    'N49'   9.38   /                                         !    48 ----> 49
! rtt
  0.18    999 /
!
!ulable  dlable  pcap   ofp(1)  olab(1)
'N49'    'N50'   0.87   /                                         !    49 ----> 50
! rtt
  0.44    999 /
!
!ulable  dlable  pcap   ofp(1)  olab(1)
'N50'    'N56' /                                                  !    50 ----> 56
! rtt
 /
!
!ulable  dlable  pcap   ofp(1)  olab(1)
'N51'    'N52'   0.78   /                                         !    51 ----> 52
! rtt
  0.20    999 /
!ulable  dlable  pcap   ofp(1)  olab(1)
'N52'    'N53'   0.70    1.0     'N55'     2  /                   !    52 ----> 53
! rtt
  0.40    999 /
!ulable  dlable  pcap   ofp(1)  olab(1)
'N53'    'N54'   0.71   /                                         !    53 ----> 54
! rtt
  0.28    999 /
!ulable  dlable  pcap   ofp(1)  olab(1)
'N54'    'N55'   0.91   /                                         !    54 ----> 55
! rtt
  0.11    999 /

!ulable  dlable  pcap   ofp(1)  olab(1)
'N55'    'N56'   /                                                !    55 ----> 56
! rtt
/
```

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APPENDIX F

WILLING AND PARTNERS (1993) DRAINAGE DESIGN PRACTICE PART II

DEPARTMENT OF URBAN SERVICES
ACT CITY SERVICES GROUP
ROADS AND TRANSPORT BRANCH
STORMWATER SECTION

Drainage Design Practice

Part II

FINAL REPORT

DECEMBER 1993

WILLING & PARTNERS
CONSULTING ENGINEERS

DEPARTMENT OF URBAN SERVICES
ACT CITY SERVICES GROUP
ROADS AND TRANSPORT BRANCH
STORMWATER SECTION

Drainage Design Practice

Part II

PROJECT NO. 3018
DECEMBER 1993

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1. INTRODUCTION

This study has sought to provide practice guidelines when utilising hydrograph based estimation procedures in urban drainage projects.

The work follows on from an earlier study entitled "Drainage Design Practice for Land Development in the ACT - Part 1: Rational Formula Procedures".

The earlier study recommended appropriate time of concentration and runoff coefficient estimation procedures based on gauged data collected from the Giralang and Mawson urban catchments.

The aim of this study has been to test several currently available rainfall/runoff computer programs including RAFTS, RORB and ILSAX on Canberra's gauged urban catchments.

In particular, the objectives of the study have been directed at:

(i) Determining appropriate design rainfall loss rate estimation parameters applicable to individual programs,

(ii) Determining appropriate surface runoff routing parameters for pervious and impervious areas specific to each program tested, and

(iii) Determination of appropriate design storm event modelling procedures specific to each program tested.

2.0 URBAN CATCHMENT DETAILS

2.1 Giralang Urban Catchment

The Giralang catchment is located in the northwest corner of Canberra and has an area of approximately 90 hectares. Approximately eighty five per cent of the catchment has been urbanised, mostly constructed during the 1970's. The catchment is typical for the Canberra area. The roads are fully sealed with kerb and gutter. Roof drainage is connected directly to the pipe network. The pipe drainage in the catchment is generally designed for the 5 Year ARI flow with overland flow paths provided over public land.

The Giralang catchment is relatively well serviced by rainfall gauges as is shown on a map of the Giralang catchment in Figure 1. A total of four pluviograph gauging stations are located inside or adjacent to the catchment. Records for these stations stretch from 1973 to date with few interruptions. The streamflow gauging station for the Giralang catchment is located as shown on Figure 1 and has operated from 1978 onwards.

2.2 Mawson and Curtin Urban Catchments

The Mawson catchment is considerably larger than the Giralang catchment being 410 hectares in area and forms part of a larger catchment known as the Curtin catchment which has an area of approximately 2,700 hectares. The catchments have been urbanised to a similar level to that of the Giralang catchment.

The Mawson catchment is not well served by rainfall gauging stations possessing only a single pluviograph located adjacent to the catchment outlet. The streamflow gauging station for the Mawson catchment has been in operation since September of 1971. The Mawson catchment is shown in Figure 2.

The Curtin catchment is served by a total of two pluviographs including the pluviograph located in the Mawson catchment. The streamflow gauging station for the Curtin catchment has been operating since January of 1970. The Curtin catchment is shown in Figure 3.

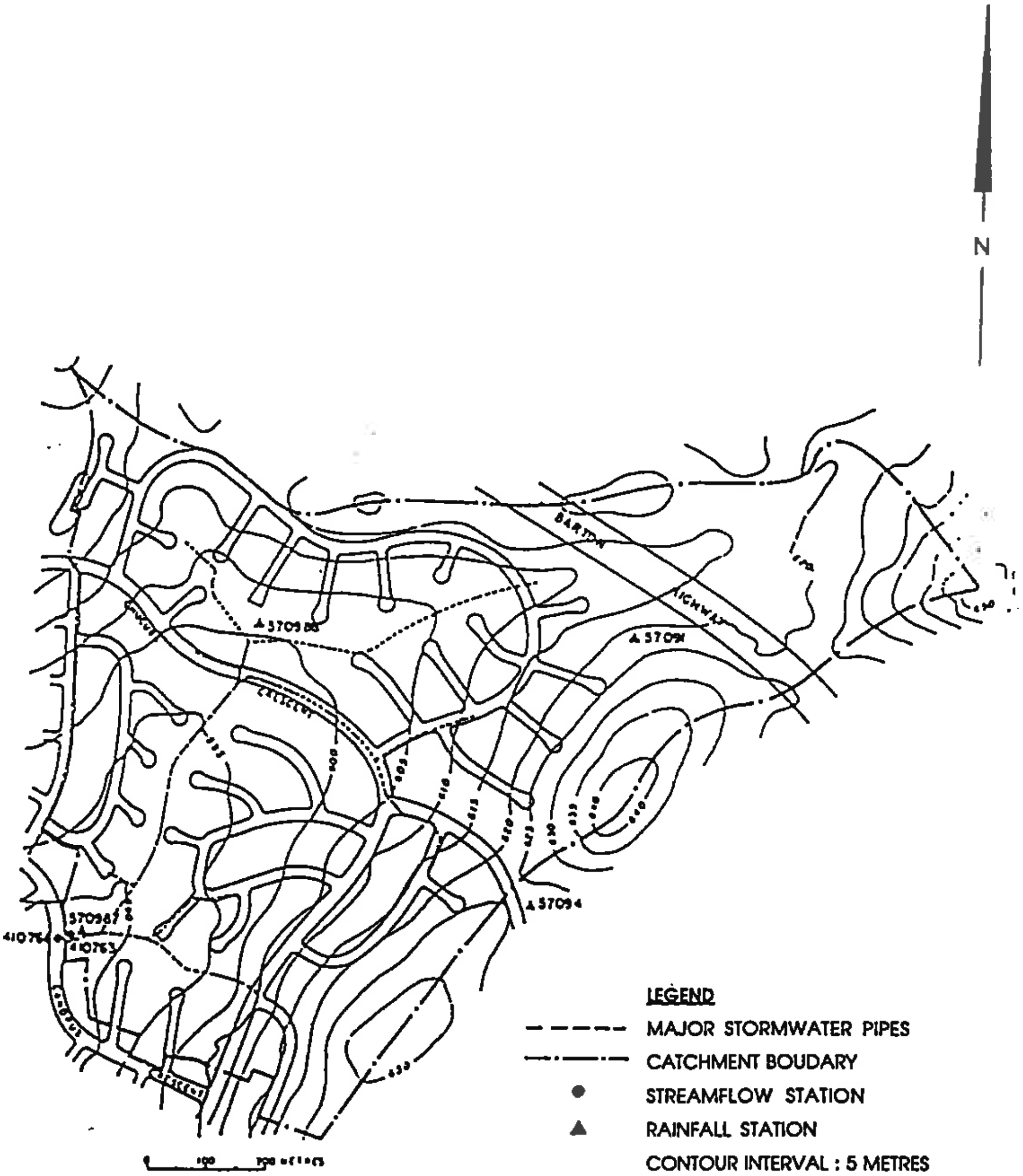


Figure 1

Giralang Catchment Plan

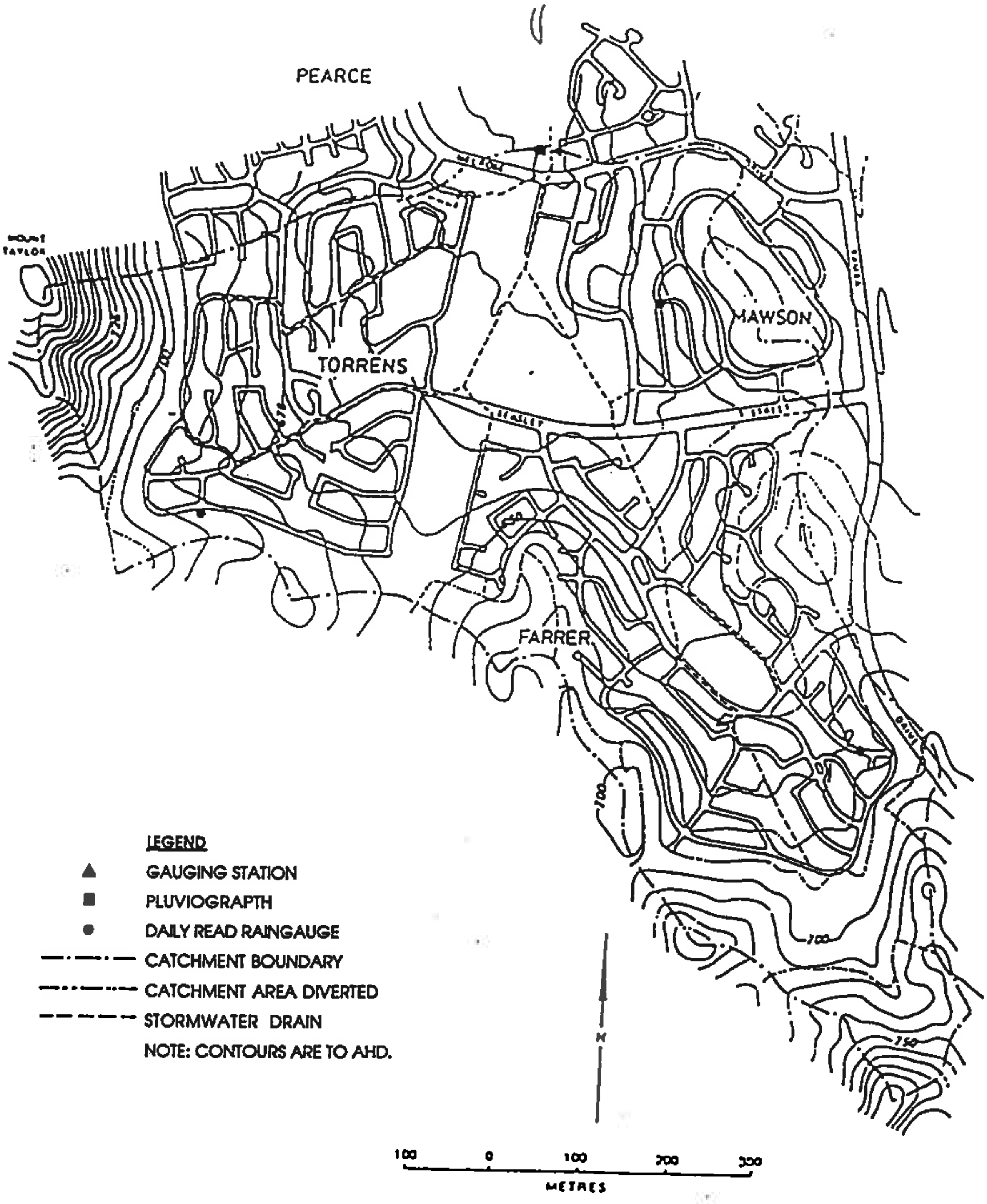
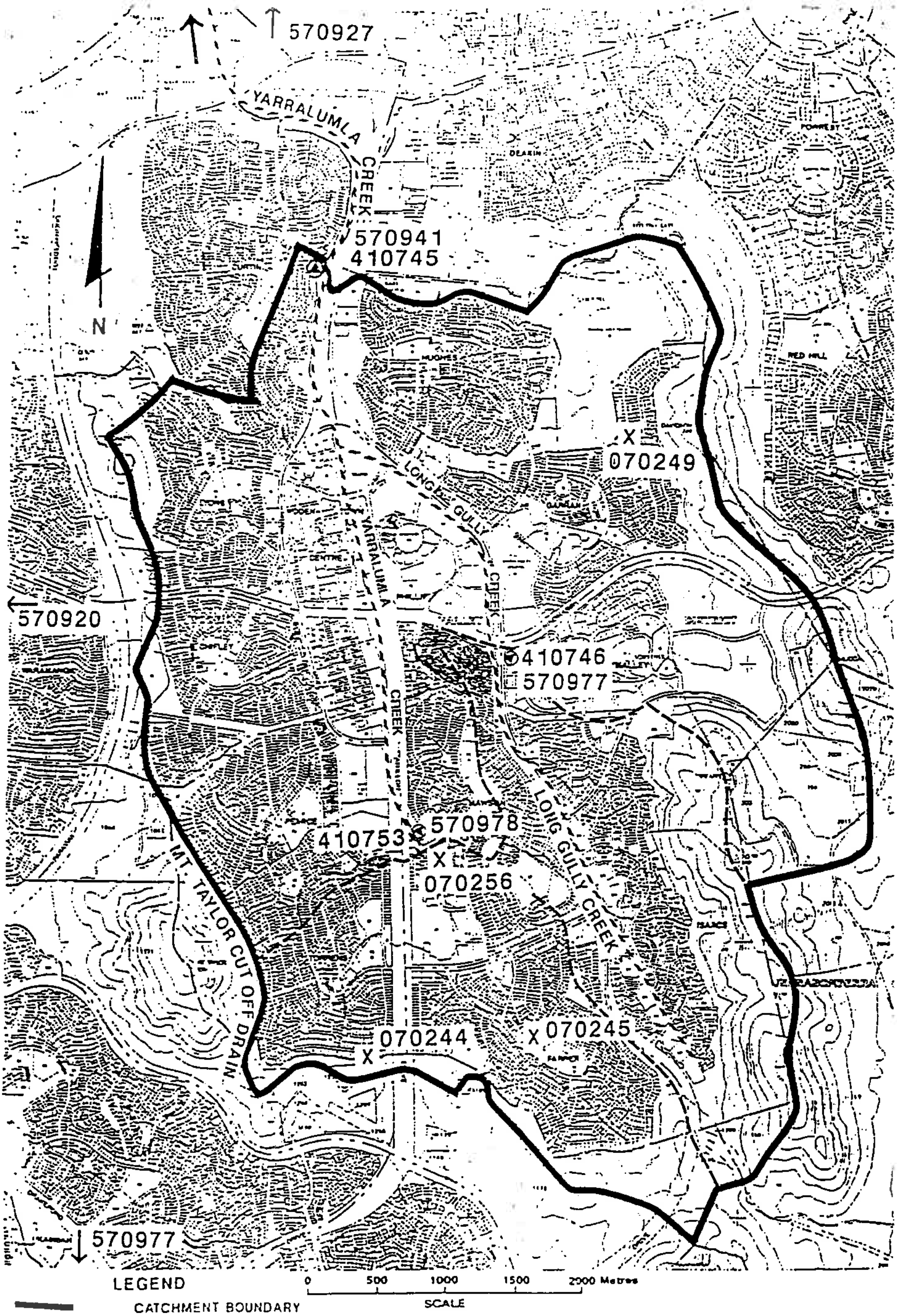


Figure 2

Mawson Catchment Plan



LEGEND

CATCHMENT BOUNDARY
SUB CATCHMENT BOUNDARY
CREEK
570978 RAINFALL STATION
X DAILY READ RAINFALL STATION
410745 STREAM GAUGING STATION

Figure 3

Curtin Catchment Plan

= Woden

3.0 GAUGED DATA

Since the Rational Formula Study in 1989 the ACT Electricity & Water have re-issued revised flood frequency curves for their gauged urban catchments taking into account subsequent data as well as any variations to rating curves found during the intermediate period.

3.1 Giralang Flood Frequency Data

Revised flood frequency curves for the Giralang catchment have not differed greatly from the earlier versions and as such do not influence earlier work carried out on that catchment. The earlier and revised flood frequency curve flows for the Giralang catchment are shown below in Table 1. The small difference in flows is due to the additional data collected since the 1989 study.

ARI (Years)	Peak Flow (m^3/s) 1989	1991
2	5.2	5.4*
5	7.2	7.0*
10	8.6	8.7**
20	10.0	10.5**
50	12.1	11.9**
100	13.8	13.2**

Table 1 - Giralang flood Frequency Flows

* based on Partial Series Analysis
** based on Annual Series Analysis

3.2 Mawson and Curt~~a~~in Flood Frequency Data

The flood frequency curves for the Mawson and Giralang catchments have recently been extensively reviewed by the Hydrology and Water Resources Branch of the ACT Electricity & Water. The results of the flood frequency curve review are discussed fully in the report "ACT Urban Catchment Flood Study, A RORB Model of the Yarralumla Creek Catchment Area", presented in Appendix C.

Much of the flood frequency review centred around a very extreme storm event occurring just before the installation of the Mawson gauge and the difficulties in rating and positioning the same event within the Curtin frequency curve. Additionally the rating table for the Mawson gauge has recently been altered significantly based on a theoretical review. This has had the effect of significantly lowering the recorded flows.

The outcome of the extensive review of the Mawson and Curtin catchment flood frequency curves was the revised flood frequency curves presented in Table 2. A full discussion on the derivation of the revised flood frequency curves is contained in the Appendix C report.

ARI Years	Mawson Catchment Peak Flow (m^3/s)		Curtin Catchment Peak Flow (m^3/s)
	1989	1991	1991
2	20	22	64
5	35	29	83
10	48	34	97
20	60	43	119
50	82	63	150
100	100	80	180

Table 2 - Mawson and Curtin Catchment Flood Frequency Flows

The length of data available in developing the flood frequency curves for the catchments is approximately 20 years. It is probable that insufficient time has elapsed for sufficient streamflow data points to be recorded to enable a highly accurate plotting of the flood frequency curves particularly for storms of an equivalent ARI of 20 years and higher. It is therefore possible that some discrepancy could occur between the rainfall/runoff model generated high level ARI flows and the current flood frequency high level ARI flows.

4.0 RAFTS

4.1 Procedure

Version 2.75 of the RAFTS-XP program was utilised in this study.

Parameter estimation, model calibration and verification was carried out at both the neighbourhood and trunk catchment levels. The neighbourhood level utilised the Giralang and Mawson urban catchments while the trunk drainage simulation concentrated on the total Woden Valley catchment to the Curtin gauge.

The general procedures to build appropriate catchment models followed the recommendations in the user and reference manual.

4.2 Sub-Catchment Discretisation

Sub-catchment discretisation was carried out at a number of levels to test the sensitivity on discharge estimates.

This included a 41 and 180 sub-catchment breakup on the Giralang and Mawson catchments respectively. The RAFTS catchment models for Giralang and Mawson are shown on Figures 4 and 5 respectively.

Additionally 20 and 10 node versions of the Giralang and 60 and 30 node versions of the Mawson catchments were tested.

The Curtin catchment model utilised the 57 node model previously developed by Ross Knee (Knee, 1990). All models utilised split pervious/impervious sub-catchments with proportions estimated from ortho-photo maps.

4.3 Rainfall Loss Assessment

RAFTS allows two approaches to rainfall loss estimation. Firstly a simple initial/continuing loss approach can be employed, and secondly a more formal infiltration/water balance procedure utilising the Australian Representative Basins Program can be applied.

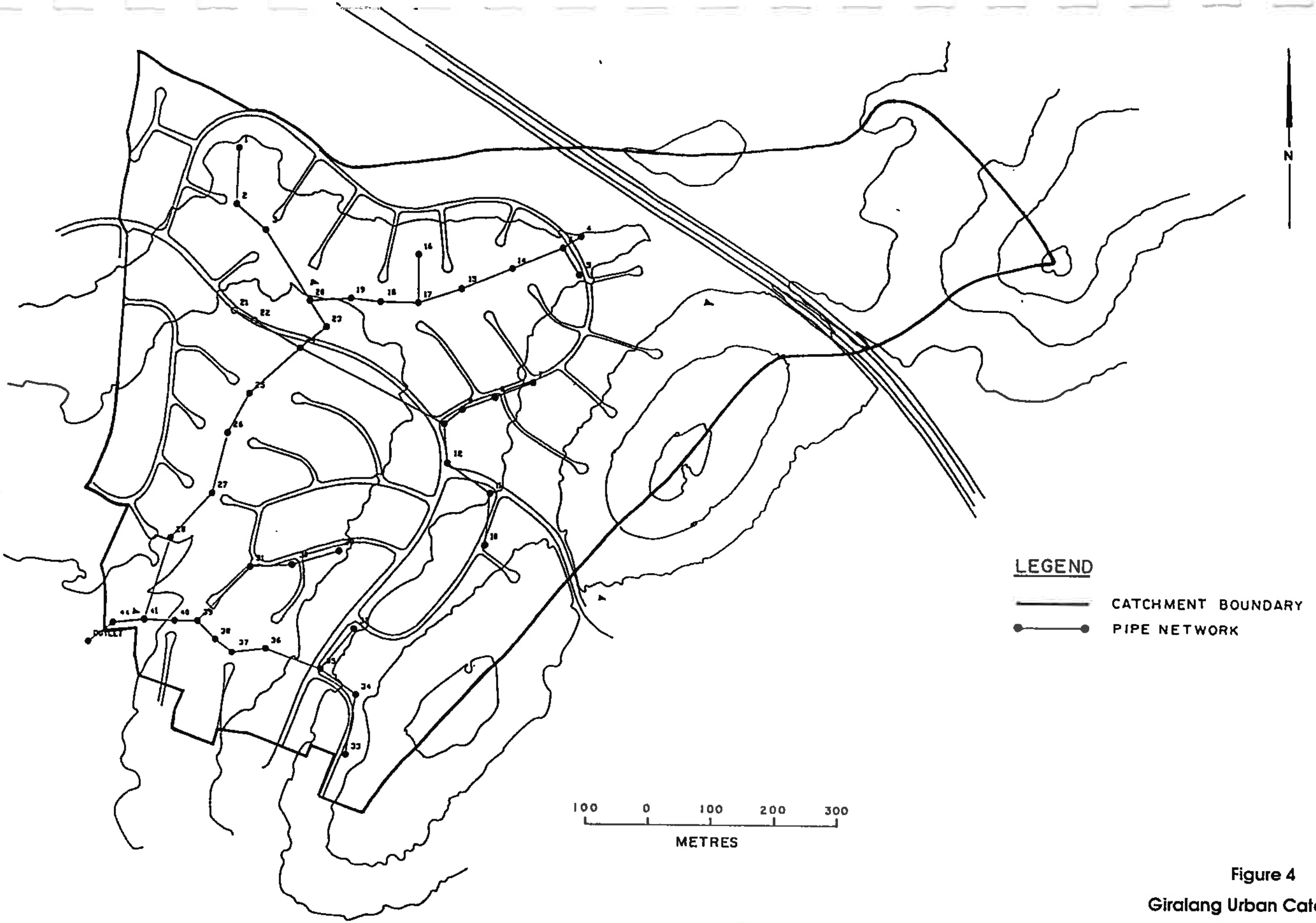


Figure 4
Giralang Urban Catchment
RAFTS Network

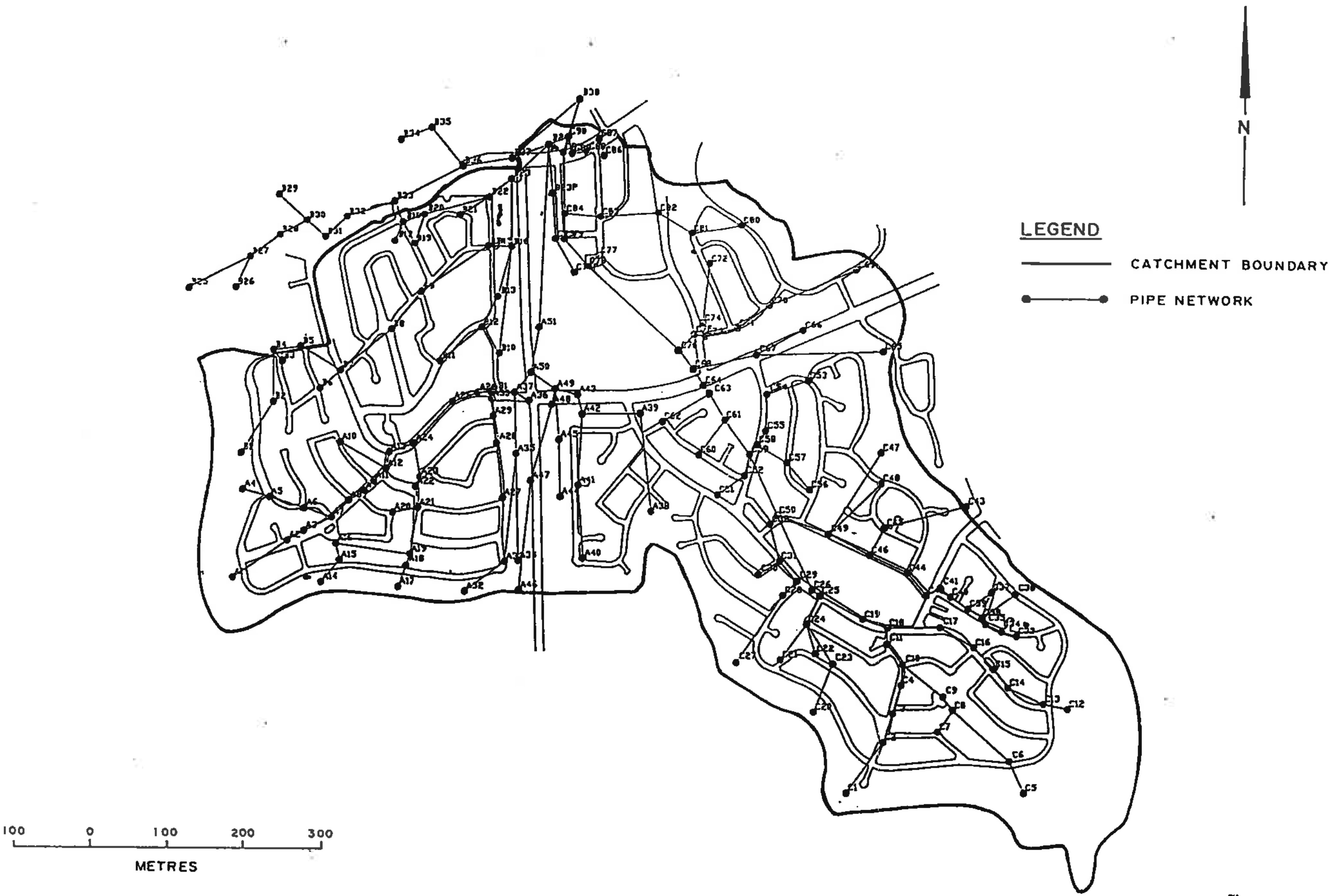


Figure 5
Mawson Urban Catchment
RAFTS Network

4.3.1 Initial/Continuing Loss Rates Approach

To date the simpler initial/continuing loss approach has been utilised most widely.

Considerable effort was put into deducing suitable design initial losses as well as continuing losses that when combined with surface routing parameters provided a reasonable approximation to the gauged flood frequency curve.

Losses of 0 - 15 mm and 2.5 - 4.0 mm/hr for pervious areas and 1.5 mm and 0 mm/hr for impervious areas had been commonly applied to many previous studies within the ACT and elsewhere.

Tables 2 and 3 indicate the results of an initial loss/continuing loss sensitivity study utilising the default surface runoff routing parameters generated by RAFTS-XP on the split pervious/impervious catchments of Giralang and Mawson. The results also indicate the variations in results in respect to catchment discretisation.

The initial/continuing loss model approach was unable to reproduce the full range of flood frequency flows as is clearly evident from Tables 3 and 4. Further graphical output displaying the variation in modelled flows as a function of the level of catchment discretisation is presented in Appendix E. It is not proposed to discuss here in depth the limitations of the initial/continuing loss model other than to say that the initial/continuing loss model is a relatively crude approximation of the loss process and models represented by Horton's equation and Phillip's equation are considered far superior. The limited ability of the initial/continuing loss model to reproduce the targetted flood frequency flows with a single set of initial and continuing loss values is therefore not surprising.

	Initial Loss (mm)	Continuing Loss (mm)	ARI (Years)	Flood Frequency Flow ($m^3/_s$)	40 nodes Modelled Flow ($m^3/_s$)	20 nodes Modelled Flow ($m^3/_s$)	10 nodes Modelled Flow ($m^3/_s$)
			100	13.2	16.6	15.1	14.2
			20	10.5	9.4	9.1	8.5
Pervious	15.0	3.0	5	7.0	5.1	4.6	4.3
Impervious	1.5	0.0	2	5.4	3.4	3.4	3.2
Bx=1.0							
			100	13.2	19.8	18.5	16.7
			20	10.5	13.0	11.5	10.8
Pervious	10.0	2.0	5	7.0	7.5	7.1	6.6
Impervious	1.5	0.0	2	5.4	4.4	4.1	3.6
Bx=1.0							
			100	13.2	21.9	21.2	19.5
			20	10.5	15.7	14.6	13.5
Pervious	5.0	2.0	5	7.0	10.6	9.6	8.8
Impervious	1.5	0.0	2	5.4	7.1	6.2	5.7
Bx=1.0							

Table 3 Giralang Loss Rate Sensitivity Results for Split Catchments

	Initial Loss (mm)	Continuing Loss (mm)	ARI (Years)	Flood Frequency Flow (m^3/s)	180 nodes Modelled Flow (m^3/s)	60 nodes Modelled Flow (m^3/s)	30 nodes Modelled Flow (m^3/s)
			100	80	79	71	64
			20	43	45	41	37
Pervious	15.0	3.0	5	29	23	21	18
Impervious	1.5	0.0	2	22	12	12	12
Bx=1.0							
			100	80	94	84	74
			20	43	60	53	47
Pervious	10.0	2.0	5	29	35	32	29
Impervious	1.5	0.0	2	22	20	18	16
Bx=1.0							
			100	80	104	94	85
			20	43	73	64	57
Pervious	5.0	2.0	5	29	47	42	37
Impervious	1.5	0.0	2	22	31	28	25
Bx=1.0							

Table 4 - Mawson Loss Rate Sensitivity Results for Split Catchments

4.3.2 ARBM Infiltration Approach to Rainfall Losses

The Australian Representative Basins Model (ARBM) provides full water balance algorithms to represent surface infiltration via Philips equation and multiple soil water stores to represent evapotranspiration, drainage, wetting and groundwater interaction. A detailed description of the ARBM loss model is given in the RAFTS-XP user manual.

In urban drainage systems all facets of soil structure and texture can influence the initial conditions prior to a significant storm event, however only a few of the parameters representing the surface infiltration and upper soil characteristics usually affect runoff generation during the storm.

Based on the above points it is necessary to only calibrate three or four parameters to characterise the pervious surfaces in question, these being Sorptivity, Hydraulic Conductivity and the Upper and Lower Soil Store Capacities.

4.4 Surface Runoff Routing Parameters

RAFTS-XP utilises two forms of surface routing processes. Firstly it utilises a storage routing model based on Laurensons (1964) non-linear cascading storage scheme to estimate subcatchment outflow hydrographs based on an input excess rainfall hyetograph over the sub-catchment.

Separate storage routing parameters ("B") are computed by the program for the pervious and impervious portions of the sub-catchment.

Pervious and impervious sub-catchment hydrographs are separately developed and then added together to give a combined hydrograph at the sub-catchment outlet.

The second form of surface routing is applied to the stream portion of the catchment runoff process.

Sub-catchment hydrographs are further routed explicitly either using the Muskingum-Cunge channel routing procedure or via a simple hydrograph lagging process based on estimated channel velocities or handed over to detailed hydraulic analysis using the comparison program EXTRAN-XP.

As the sub-catchment storage routing process is non-linear in nature, two parameters are required, namely 'B' and 'n' where 'B' is the storage delay parameter and 'n' is the parameter representing the degree of non-linearity.

RAFTS-XP sets 'n' equal to -0.285 as its default value based on research carried out by Askew (1968). Although there are provisions in RAFTS-XP to alter 'n' it is recommended in the manual where better data is unavailable, to adopt the default value.

This leaves only one surface routing parameter to estimate being 'B'. The parameter is resolved internally by RAFTS-XP based on a regression equation relating 'B' to a function of sub-catchment area, weighted slope, imperviousness and surface roughness.

4.5 Historical Storm Event Modelling

The historical storm event modelling was carried out using the ARBM loss model. The initial calibration was carried out on a Giralang catchment storm which occured on 5 February, 1981.

The approach adopted to calibration was as follows:

1. The Upper Soil Store Capacity was set to a value of 25 mm. This was based on a typical catchment predetermined grass root depth of 50 mm and from Goyen's Master Thesis measurements (1981) of 50% voids.

2. The Hydraulic Conductivity was set to a value of 0.33 mm/min based on an average value measured for various topsoils, throughout the ACT.

3. The sorptivity was set to a value of 3 mm/min0.5 half which is representative of a clay based topsoil and was obtained from a range of values reported by Talsma (1969).

4. The calibration against the 1981 gauged storm then proceeded by varying the Lower Soil Store Capacity along with initial soil store values for the Upper and Lower Soil Stores until a volume calibration was achieved. All other values within the ARBM loss model were allocated model default values.

5. Following the volume calibration, the value of the pervious surface roughness was modified until a peak flow calibration was achieved.

The resulting set of calibrated ARBM parameter values for the 1981 Giralang storm event are shown in Table 5. The calibrated surface roughness values were 0.015 and 0.040 for the impervious and pervious areas respectively. The simple hydrograph lagging process was employed for the calibration based on estimated pipe and channel velocities.

Loss Model	Adopted Values	Initial Values
Storage Capacities		
Impervious (IMP)	0.50	0.0
Interception (ISC)	1.00	0.0
Depression (DSC)	1.00	0.0
Upper soil (USC)	25.00	20.00
Lower soil (LSC)	50.00	40.00
Infiltration		
Dry soil sorptivity (SO)	3.00	
Hydraulic conductivity (K_O)	0.33	
Lower soil drainage factor (LDF)	0.05	
Groundwater recession;		
constant rate (KG)	0.94	
variable rate (GN)	1.00	
Evapo-Transpiration		
Proportion of rainfall intercepted by vegetation (IAR)	0.70	
Max potential evapotranspiration;		
upper soil (UH)	10.00	
lower soil (LH)	10.00	
Proportion of evapotranspiration from upper soil zone (ER)	0.70	
Ratio of potential evaporation to A class pan (ECOR)	0.90	

Table 5 - Calibrated ARBM Parameters for Pervious Areas on Giralang Catchment

The routing interval (DT) adopted for the above calibration was equal to 0.5 minutes due to the short lag times present in the model. The values of initial upper and lower soil store capacities which resulted in the best calibration were 15 mm and 30 mm respectively.

A verification of the calibrated Giralang RAFTS model was carried out against another storm which occurred on 13 December, 1983. All parameter values were retained excluding the initial upper and lower soil store capacities. The results of the calibration and verification analysis are shown in Table 6.

Event Date	Gauged Data			Calibrated	RAFTS	Model
	Total Rainfall (mm)	Peak Flow ($m^3/_s$)	Volume Runoff (m^3)	Initial Catchment Wetness (%)	Peak flow ($m^3/_s$)	Volume Runoff (m^3)
05.02.81	59.2	10.1	32,100	60*	9.7	32,700
13.12.83	25.0	6.8	14,900	90**	7.2	14,700

Table 6 - Giralang RAFTS Storm Event Modelling

* corresponds to 15.0 mm and 30.0 mm initial upper and lower soil store values

** corresponds to 22.5 mm and 45.0 mm initial upper and lower soil store values

A high level of calibration was achieved on the two storm events modelled. The calculated and gauged hydrographs for the above two storms are shown on Figures 6 and 7. The large number of rainfall recording stations relative to the area of the catchment would have be one of the principal reasons that a successful calibration and verification was achieved.

The RAFTS modelling of the Mawson catchment was carried out using the same approach as that for the Giralang catchment. A summary of the results is shown in Table 7.

Event Date	Gauged Data			Calibrated	RAFTS	Model
	Total Rainfall (mm)	Peak Flow ($m^3/_s$)	Volume Runoff (m^3)	Initial Catchment Wetness (%)	Peak flow ($m^3/_s$)	Volume Runoff (m^3)
14.01.77	29.7	44	71,400	70*	53	76,800
06.01.81	22.8	43	67,700	95**	37	69,600

Table 7 - Mawson RAFTS Storm Event Modelling

* corresponds to 17.5 mm and 35.0 mm initial upper and lower soil store values

** corresponds to 23.75 mm and 47.5 mm initial upper and lower soil store values

Values of 0.015 and 0.040 were again adopted as the sub-catchment roughness factors for the impervious and pervious areas respectively. A relatively poor level of calibration was achieved on the Mawson catchment model. This could be attributed to the fact that Mawson is a relatively large catchment (410ha) compared to Giralang and currently relies on a single pluviograpth rainfall recording station. It is therefore highly susceptable to rainfall area variability.

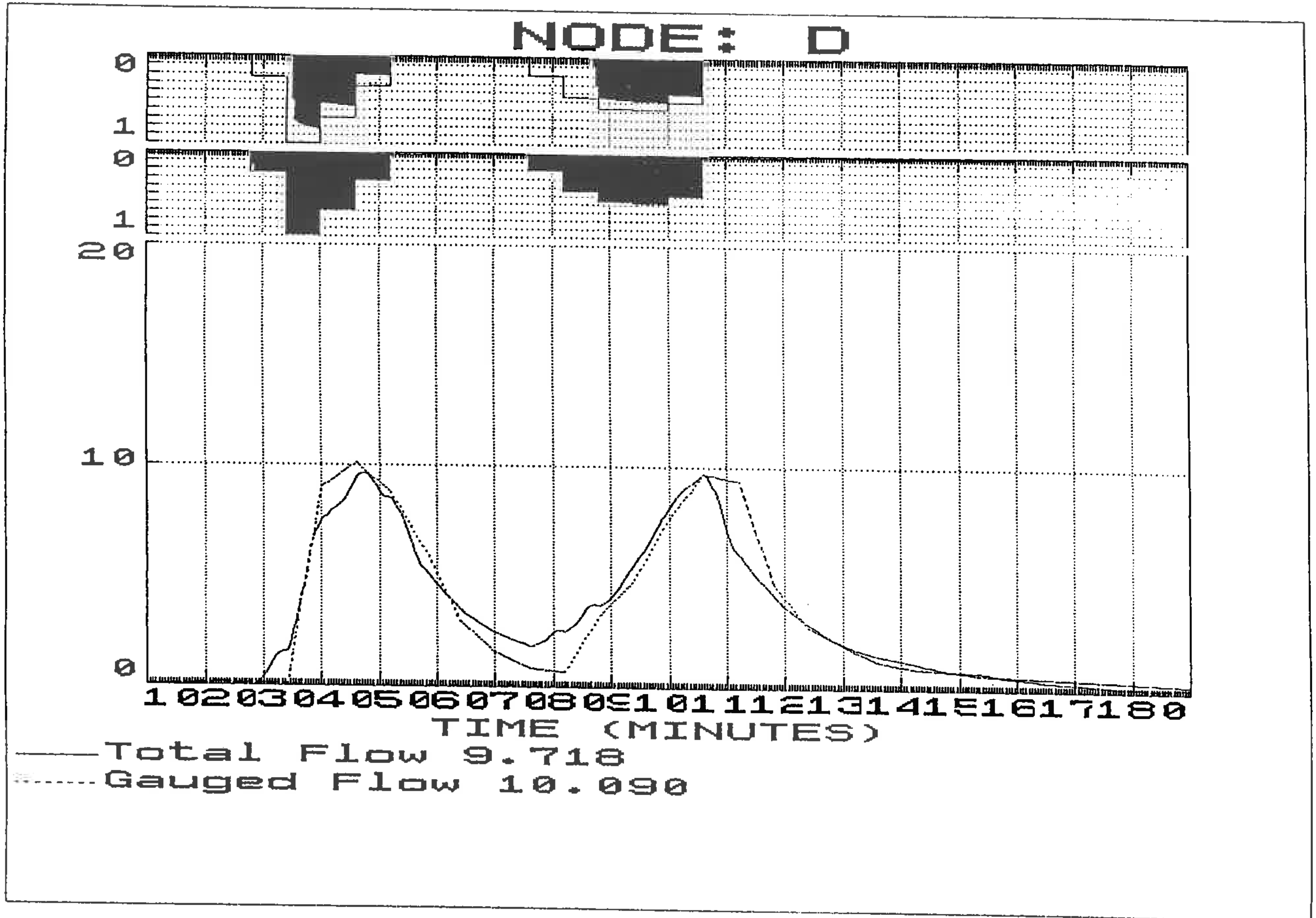


Figure 6

Giralang Gauged Storm 05.02.81

RAFTS Calibration

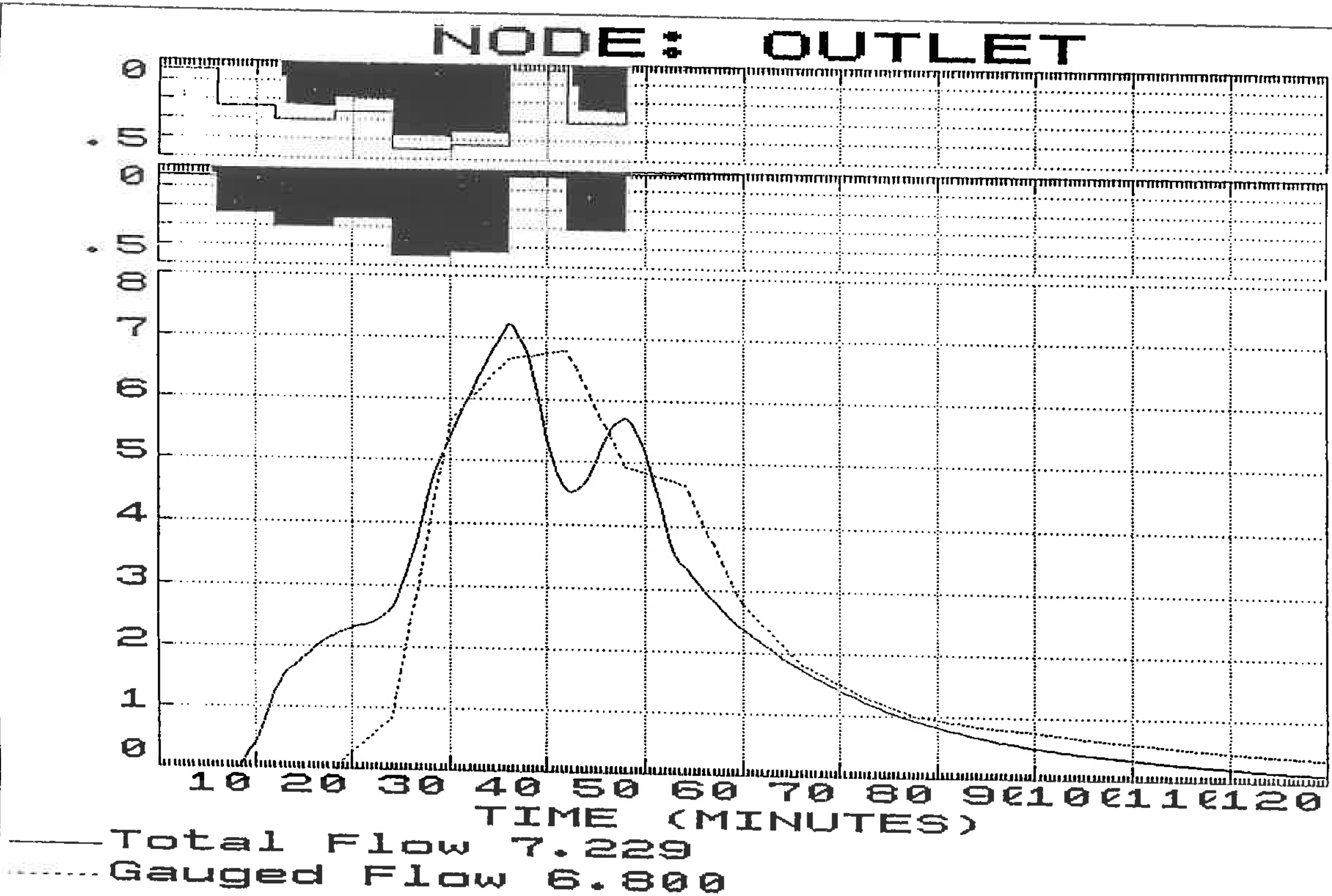


Figure 7

Giralang Gauged Storm 13.12.83

RAFTS Calibration

4.6 Design Storm Event Modelling

4.6.1 Initial Catchment Wetness Condition

The analysis of historical storm events enabled initial catchment wetness conditions to be varied until the model best simulated the gauged flow. By comparing the historical storm events with the current flood frequency curves for both the Mawson and Giralang catchments the approximate magnitude of the historical storm events modelled in terms of average recurrence interval was able to be calculated. A plot of initial wetness condition against the storms equivalent average recurrence interval was then made and is shown in Figure 8. The results for the 4 points shown range from 60 to 95% with no clearly identifiable trend being evident.

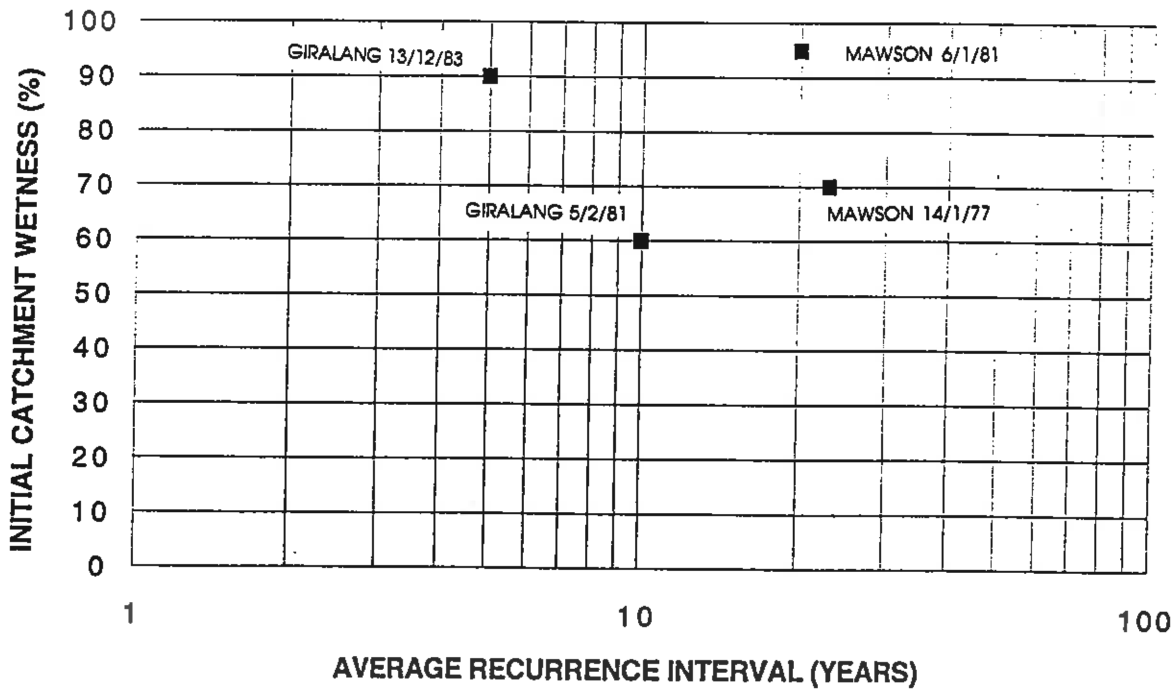


Figure 8 - Initial Catchment Wetness Condition Versus ARI

The non-evidence of any trends in the above results could be attributable to the following causes:

1. Insufficient historical storms have been analysed to indicate a trend.

2. The unreliability of the Mawson data points for reasons given earlier.

3. Failure to model rainfall occurring prior to historical design storm bursts.

4.6.2 Design Storm Analysis

After consideration was given to the above results, the approach adopted to design storm modelling was to adopt the ARBM parameters listed in Table 5 and calibrate the initial catchment wetness condition against the current 5 Year ARI flood frequency design flow. Peak flow for design storms of all other recurrence intervals were then calculated using the 5 Year ARI calibrated model parameters.

This approach was adopted as it is considered the current flood frequency curves for the Giralang and Mawson catchments are unlikely to be accurate for storm events of magnitude greater than 20 Years ARI due to the extremely small number of data points in this range. The results of the design storm modelling are shown in Table 8. All modelling was based on a design storm duration of one hour which coincides with the critical length event.

Design Storm ARI (Years)	Giralang Catchment Flood Frequency Flow (m^3/s)	Giralang Catchment Modelled Flow (m^3/s)	Mawson Catchment Flood Frequency Flow (m^3/s)	Mawson Catchment Modelled Flow (m^3/s)	Curtin Catchment Flood Frequency Flow (m^3/s)	Curtin Catchment Modelled Flow (m^3/s)
2	5.4	3.6	22	17	64	56
5	7.0	7.3	29	31	83	80
10	8.7	9.0	34	38	97	103
20	10.1	11.5	43	48	119	134
50	11.9	13.8	63	60	150	172
100	13.2	16.4	80	75	180	208

Table 8 - RAFTS Design Storm Modelling

The Giralang and Mawson catchment calibrations against the 5 Year flood frequency flows resulted in an initial wetness level of 80% being adopted. The Curtin catchment calibration required a lower store wetness level of 72.5% to reproduce the 5 Year flood frequency flow.

4.7 Sensitivity Analysis

4.7.1 Initial Catchment Wetness Condition

The sensitivity of peak flow to the adopted initial catchment wetness condition was checked on the Giralang catchment. The results of the analysis are shown in Table 9. The results indicate that the peak flow is reasonably sensitive to the adopted initial catchment wetness during storms of sufficient magnitude to induce runoff from the pervious areas within the catchment.

A possible approach to overcoming uncertainty as to what is an appropriate average initial catchment wetness condition prior to a storm burst would be to undertake a study involving continuous modelling of a catchment over a long period of time. The continuous modelling would eliminate the need for adopting an arbitrary catchment wetness condition prior to storm bursts and hopefully result in the exposure of a catchment wetness condition which could then be used for design storm modelling purposes.

Initial Catchment Wetness (%)	Giralang Catchment 5 Year ARI Peak Flow (m^3/s)	5 Year ARI Volume Runoff (m^3)	100 Year ARI Peak Flow (m^3/s)	100 Year ARI Volume Runoff (m^3)
0	4.2	5,300	7.6	14,800
10	4.2	5,300	8.2	16,400
20	4.2	5,300	8.5	17,400
30	4.2	5,300	11.4	21,400
40	4.2	6,400	14.1	24,500
50	4.6	7,800	15.2	26,100
60	4.8	8,600	15.8	27,300
65	6.5	11,300	16.1	27,800
70	7.3	12,300	16.5	28,400
75	7.6	12,900	17.7	30,100
80	8.0	13,500	18.2	30,900
90	9.0	15,000	19.0	32,400
100	10.5	18,600	20.8	36,400

Table 9 - Sensitivity of Peak Flow to Initial Catchment Wetness

4.7.2 Surface Runoff Routing Parameters

The sensitivity of peak flows to both the surface roughness value and the degree of catchment non-linearity were evaluated. Values of surface roughness and catchment non-linearity were varied on the RAFTS Giralang catchment model to determine the sensitivity of the model to these parameters. The results of the sensitivity analysis are shown in Table 10.

ARI (Years)	Surface Roughness (Pervious) *	Flow (m^3/s)	Non-Linearity Parameter (n) **	Flow (m^3/s)
5	0.025	9.1	-0.001 (linear)	9.2
	0.030	8.4	-0.1	8.6
	0.040	7.3	-0.2	8.0
	0.050	6.6	-0.25	7.6
	0.060	6.1	-0.285 (default)	7.0
	0.080	5.3	-0.35	6.6
	0.100	5.0	-0.5	5.2
100	0.025	18.8	-0.001	17.7
	0.030	17.6	-0.1	17.3
	0.040	16.3	-0.2	16.9
	0.050	14.0	-0.25	16.6
	0.060	12.8	-0.285	16.3
	0.080	11.1	-0.35	15.7
	0.100	10.0	-0.5	13.2

Table 10 - Sensitivity of Peak Flow to Surface Runoff Parameters

* Constant catchment non-linearity of -0.285 retained

** Constant pervious surface roughness of 0.04 retained

The resulting trends arising from the sensitivity analysis indicate that the peak flow will decrease with increasing pervious surface roughness and increasing catchment non-linearity response. The increase in surface roughness will produce a physical retardation effect on surface runoff and is therefore, expected to have an attenuation effect on the resulting peak flow. Increasing the catchment non-linearity response has the effect of reducing the peak flow due to the form of the storage-discharge relationship as follows:

$$S = BQ^{(n-1)}$$

Considerable discussion occurs in recent literature that proposes that catchment response becomes more linear during infrequent events and flow levels are proportionally lower than non-linear estimates. The reported discussions however are based around catchment sectors utilizing one storage routing scheme and and are usually developed from larger river basins.

River/floodplain systems may well become more linear in response as flows exceed channels. Urban surface runoff however, differs significantly from major river flood flows and due to shallow flow depths may in fact become more non-linear with flow due to decreases in effective roughnesses.

Additionally and possibly more important is the fact that when catchment storage routing is carried out at a sub-catchment level as in RAFTS-XP, changes in non-linearity can have either of two effects. When sub-catchment pervious flows are less than one m^3/s changes to a linear system will in fact increase flow levels as indicated in Table 10. When flows are greater than one the opposite is true based on the form of the storage/discharge equation alone.

5.0 ILSAX

5.1 Procedure

The ILSAX program was also used to analyse both historical gauged storms and design storms on the Giralang and Mawson urban catchments. Versions 2.08 of the ILSAX program was utilised in this study. The general procedures to build appropriate catchment models followed the recommendation in the user manual.

5.2 Catchment Discretisation

ILSAX sub-catchment models consisting of 41 and 180 sub-catchments for the Giralang and Mawson catchments respectively were used for this study. Model sub-catchments pervious and impervious portions were identical to that used in the RAFTS analysis. Copies of the ILSAX Giralang and Mawson data sets are presented in Appendix B.

5.3 Rainfall Loss Assessment

The ILSAX program uses Horton's infiltration equation to determine rainfall losses occuring on pervious surfaces. A pervious depression storage value is specified and acts as a kind of initial loss. A value of 5 mm for the pervious depression storage was adopted in this study as typically recommended in the ILSAX user manual.

Losses from impervious surfaces are calculated by subtracting an impervious depression storage value from the rainfall hyetograpth. A value of 1 mm was adopted for the impervious depression storage in this study.

ILSAX also requires that a catchment soil type be specified. For both the Giralang and Mawson catchments a soil type which assumes slow infiltration rates was adopted. It is described as Soil Type 3 within the ILSAX user manual.

5.4 Surface Runoff Routing Parameters

Times of concentration required by the ILSAX catchment model must be calculated by the user and input directly as model data. An earlier report entitled "Drainage Design Practice for Land Development in ACT - Part 1: Rational Formula Procedures", recommended that Friend's equation be adopted for the determination of the time of overland flow. Accordingly times of concentration for the pervious surfaces within the catchments were calculated using Friend's equation.

The ILSAX user manual recommends that a time of 5 minutes be allowed for roof to gutter travel times. The subsequent approach adopted in this study was to allocate a global impervious area time of concentration of 6 minutes to all impervious surfaces.

5.5 Historical Storm Event Modelling

5.5.1 Approach Adopted

As mentioned previously, the catchment soil type and depression storage loss parameters were assigned suitable values. The calibration approach adopted was therefore to leave these values constant and vary the antecedant moisture condition (AMC) parameter which represents the catchment wetness condition at the commencement of modelling.

The ILSAX analysis assumed that ponding did not occur within the catchment at pit inlets. This assumption was considered reasonable given the surface level grades present in both the Giralang and Mawson catchments.

5.5.2 Giralang Catchment

The results of the Giralang ILSAX historical storm event modelling analysis are summarised in Table 11. The results indicate that the AMC approaches saturation point on two of the storm events (AMC=4.0 corresponds to soil saturation).

Event	Gauged Data			Calibrated ILSAX Model		
Date	Total Rainfall (mm)	Peak Flow (m^3/s)	Volume Runoff (m^3)	AMC Adopted	Peak Flow (m^3/s)	Volume Runoff (m^3)
05.02.81	69.4	10.1	34,000	2.7	12.3	34,200
13.12.83	25.0	6.8	14,900	3.8	9.1	14,900

Table 11 - Giralang ILSAX Historical Storm Event Modelling

The calculated and gauged hydrographs for the above storms are shown in Figures 9 and 10.

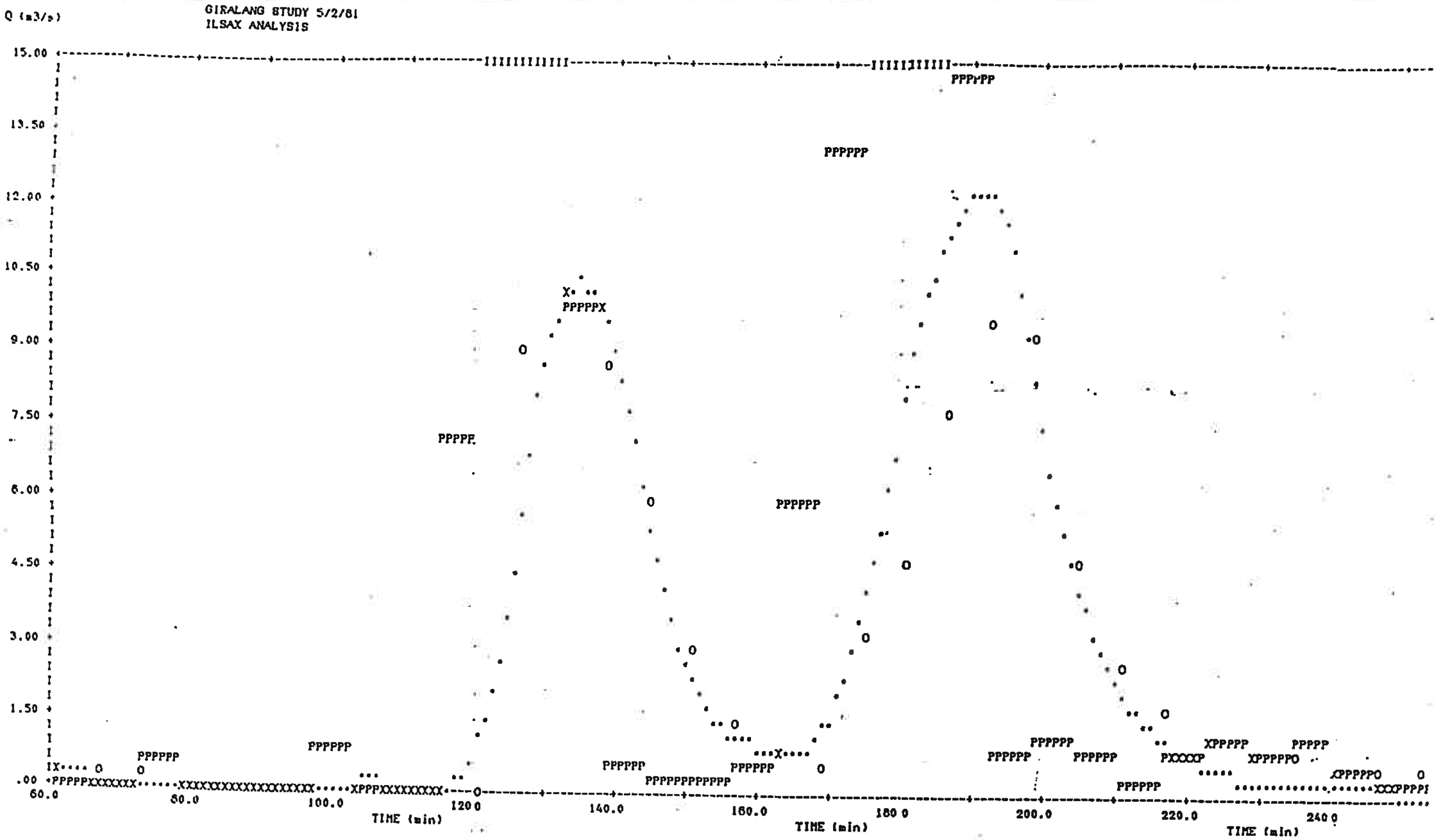


Figure 9

Giralang Gauged Storm 05.02.81

ILSAX Calibration

GIRALANG STUDY 13/12/83
ILSAX ANALYSIS

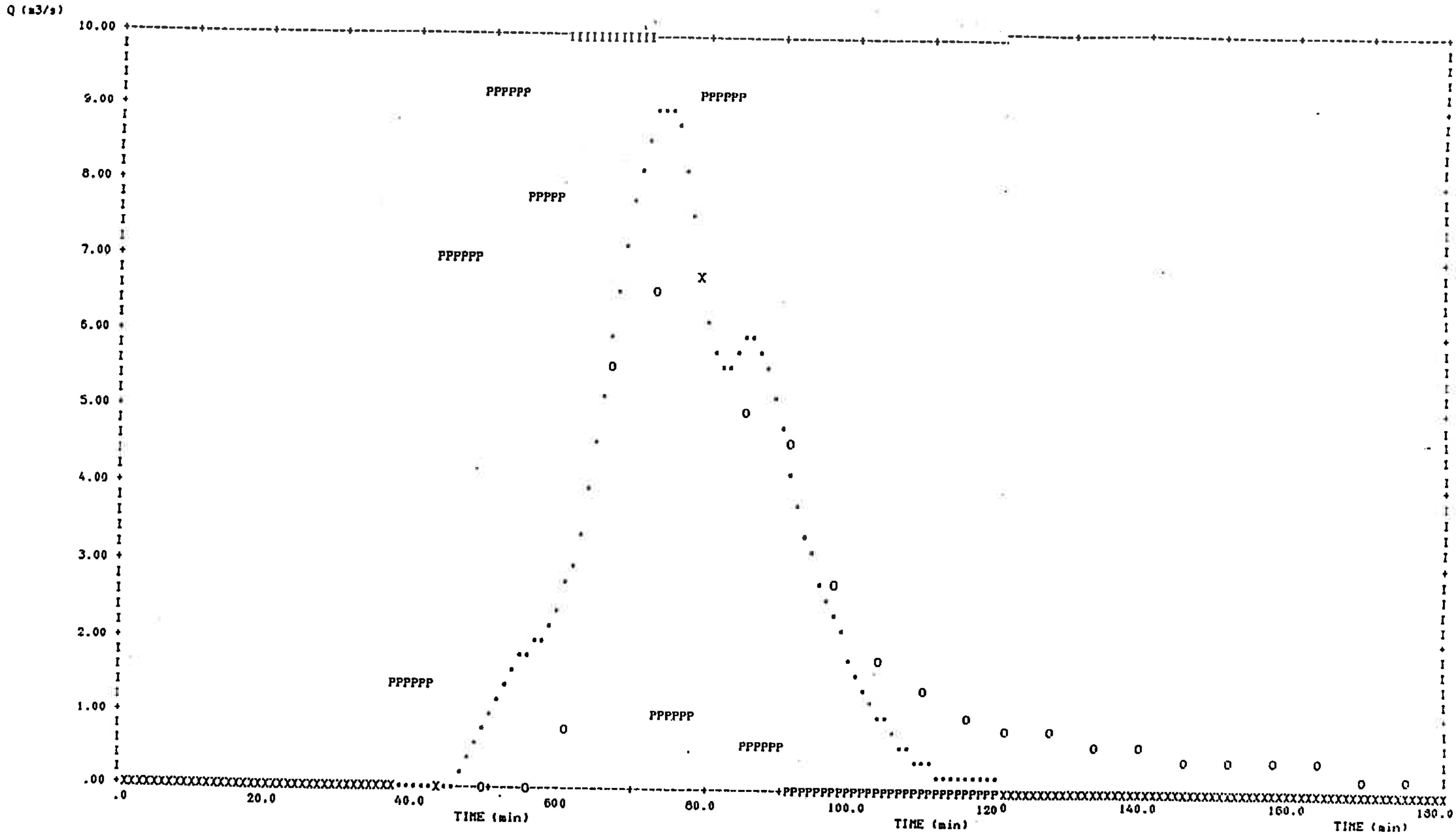


Figure 10

Giralang Gauged Storm 13.12.83

ILSAX Calibration

5.5.3 Mawson Catchment

The results of the Mawson ILSAX historical storm event modelling analysis are summarised in Table 12. The results indicate a relatively poor level of calibration was achieved. This could be attributed to the same factors discussed earlier in the RAFTS section of this report. They were failure to model rainfall area variability effects.

Event Date	Gauged Data			Calibrated ILSAX Model		
	Total Rainfall (mm)	Peak Flow (m^3/s)	Volume Runoff (m^3)	AMC Adopted	Peak Flow (m^3/s)	Volume Runoff (m^3)
14.01.77	29.7	44.3	71,400	3.0	51.9	69,800
06.01.81	27.3	42.6	75,100	4.0	42.8	70,700

Table 12 - Mawson ILSAX Historical Storm Event Modelling

5.6 Design Storm Event Modelling

5.6.1 Antecedant Moisture Condition (AMC)

The analysis of historical storm events enabled the AMC to be varied until the ILSAX model most closely simulated the gauged flow. The equivalent ARI of the historical storm events was determined by comparison with their respective flood frequency curves. This allowed a plot of AMC versus ARI to be made which is shown on Figure 12. No clear evidence of a relationship between AMC and ARI is evident from the four points plotted. It is considered that further historical storm event analysis incorporating continuous catchment modelling may reveal an average catchment AMC.

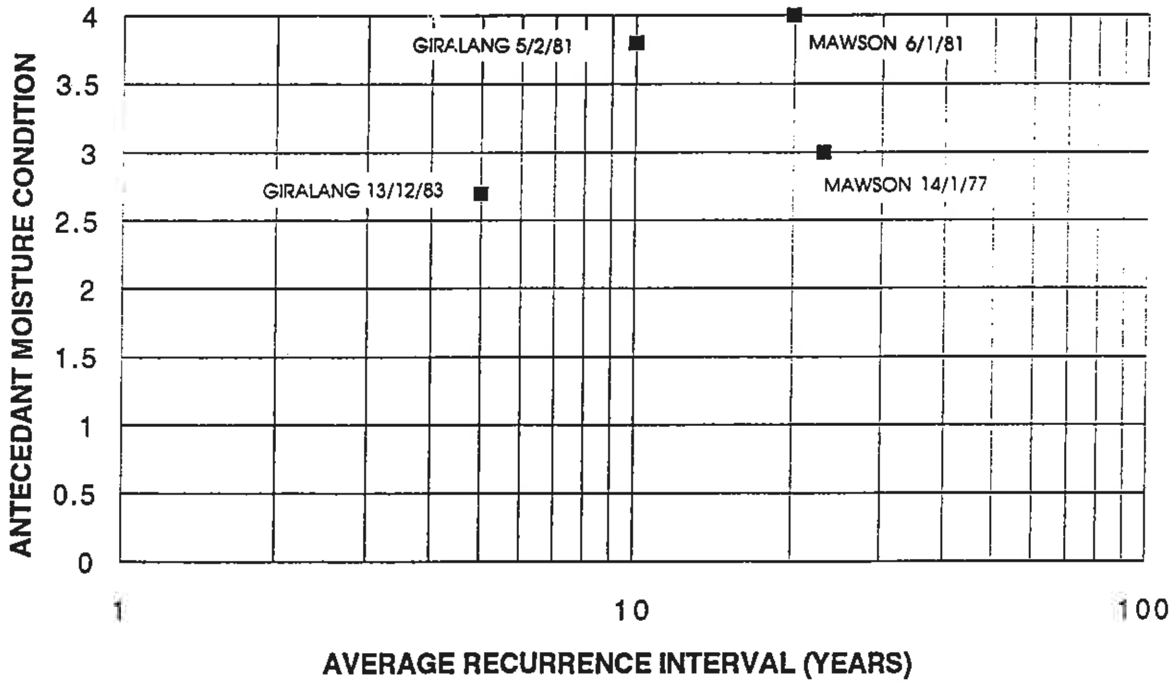


Figure 11 - Antecedant Moisture Condition Versus ARI

* Giralang Data Points
** Mawson Data Points

5.6.2 Design Storm Analysis

As was the case with the RAFTS design storm modelling the approach adopted was to firstly calibrate the ILSAX model against the current 5 Year ARI flood frequency flow. All other design ARI flows were obtained by retaining the same AMC. The results of the ILSAX design storm modelling for both catchments is summarised in Table 13. All modelling was based on a design storm duration of one hour.

Design Storm ARI (Years)	Giralang Catchment Flood Frequency Flow ($m^3/_s$)	Giralang Catchment Modelled Flow ($m^3/_s$)	Mawson Catchment Flood Frequency Flow ($m^3/_s$)	Mawson Catchment Modelled Flow ($m^3/_s$)
2	5.4	3.5	22	15
5	7.0	7.0	29	30
10	8.7	9.0	34	37
20	10.1	11.9	43	49
50	11.9	14.8	63	63
100	13.2	17.6	80	75

Table 13 - ILSAX Design Storm Modelling

The analysis resulted in an AMC of 3.15 and 3.20 being calibrated against the 5 Year ARI flood frequency flows for the Giralang and Mawson catchments respectively. The AMC parameter was fixed at these values to produce the full range of modelled flood frequency flows. The modelled flows for the Giralang catchment exceed the flood frequency flows for all events greater than the 5 Year ARI storm. This could be attributed to the fact that only a small number of storms of magnitude greater than 10 years ARI in magnitude have been recorded. The proportion of pervious contributed runoff is small for the bulk of the recorded storms on the current flood frequency curve for Giralang. However, storms of magnitude greater than say 10 Years ARI would have a much higher proportion of pervious runoff and could therefore be underestimated by the current flood frequency curves.

The Mawson modelled flows do not appear to show any identifiable trend. The ILSAX model was able to reproduce the Mawson flood frequency flows to an acceptable comparison level.

5.6.3 Sensitivity Analysis

The sensitivity of design storm peak flows to the AMC was examined for 5 Year and 100 Year ARI, 1 hour duration design storm events on the Giralang catchment. The result of the sensitivity analysis are shown in Table 14.

AMC	Giralang Catchment			
	5 Year ARI		100 Year ARI	
	Peak Flow ($m^3/_s$)	Volume Runoff (m^3)	Peak Flow ($m^3/_s$)	Volume Runoff (m^3)
0.00	3.7	5,510	6.2	10,130
0.50	3.7	5,510	6.2	10,130
1.00	3.7	5,510	6.2	10,130
1.50	3.7	5,510	7.4	12,720
2.00	3.7	5,510	10.6	17,890
2.25	3.7	5,510	12.2	20,650
2.50	3.7	5,650	13.7	23,630
2.75	4.7	7,780	15.3	26,750
3.00	6.2	10,200	16.8	29,770
3.25	7.6	12,520	18.2	32,630
3.50	9.0	14,820	19.3	35,200
3.75	10.1	16,550	20.1	37,040
4.00	10.5	17,320	20.5	37,810

Table 14 - Sensitivity of Peak Flow to antecedant Moisture Condition (AMC)

The AMC is responsible for controlling levels of runoff from pervious areas within the catchment. The peak design flows are therefore, sensitive to storms sufficient in magnitude to produce pervious runoff. The AMC, therefore, will be a more sensitive parameter for high level ARI storms.

6.0 RORB

The RORB analysis for this study was carried out by the Hydrology and Water Resources Branch of the ACT Electricity and Water department. A separate report prepared on the RORB analysis portion of the study is presented in Appendix C. The following sections summarize the methology used and the results obtained from the RORB modelling.

6.1 Procedure

Version 4 of the RORB rainfall/runoff program was utilized for analysis in this study.

The RORB program was used to model design storms on both the Curtin and Mawson urban catchments. The general procedures to build appropriate catchment models followed the recommendations in the program user manual.

6.2 Catchment Discretisation

The RORB catchment models of the Mawson and Curtin catchments consisted of 7 and 56 sub-catchments respectively. The RORB catchment models for Mawson and Curtin are shown in Appendix C.

6.3 Rainfall Loss Assessment

The loss model approach adopted followed the recommended procedures in the program user manual. The RORB model assumes 90% of the rainfall occuring on impervious areas acts as runoff. Rainfall losses on pervious areas were modelled as an initial loss followed by a runoff coefficient or constant (continuing) proportional loss rate. An initial loss of 10 mm was adopted. The runoff coefficient was used as a calibration parameter.

6.4 Surface Runoff Routing Parameters

The routing method used within the RORB model and detailed in the user manual is based on the reach storages behaving as per the following storage-discharge relationship:

$$S = 3{,}600\,KQ^{m}$$

where S = storage (m^3)

Q = outflow discharge ($m^3/_s$)

m = a dimensionless exponent

K = a dimensional emperical coefficient

The two principle storage routing parameters, K and m used in the RORB model play an important role in catchment model calibration. The parameter m, a dimensionless coefficient, is a measure of the catchments non-linearity and a value of 1.0 implies a linear catchment. The RORB user manual recommends that for catchment areas up to several thousand kilometres in area a value in the range of 0.6 to 1.0 be adopted. The approach to model calibration in this study was to adopt the default RORB value for m of 0.8.

The other RORB routing parameter K, a dimensional emperical coefficient, is the product of two factors, Kr and Kc, where Kr is a dimensionless ratio called the relative delay time applicable to an individual reach storage and Kc is an emperical coefficient applicable to the entire catchment and stream network.

The relative delay time Kr, is calculated within the RORB program based on the user inputted reach data. The emperical coefficient Kc is the principle runoff routing parameter and is crucial to the calibration process. The approach recommended in the RORB user manual and adopted in this study was to calculate Kc via the default RORB model emperical equation as follows:

$$Kc = 2.2A^{0.5}({}^{QP}/_{2})^{0.8-m}$$

Which reduces to the following equation when a value of 0.8 is adopted for m:

$$Kc = 2.2A^{0.5} \qquad \text{where } A = \text{catchment area } (km^2)$$

As the value of Kc depends on the catchment area, it is not possible to use the same calibrated catchment Kc value on other catchments of differing size.

6.5 DESIGN STORM EVENT MODELLING

Both the Curtin and Mawson RORB models were calibrated against the flood frequency curve flows. The runoff coefficient parameter was varied to achieve a RORB model calibration against the targetted flood frequency peak flows. The following sections provide details on the results of the RORB modelling.

6.5.1 Catchment Calibration Results

The results of the RORB modelling on the Mawson and Curtin catchment are summarised in Table 15. The critical duration storm event for the Mawson catchment was found to be one hour. The runoff coeficient parameter used for calibration on the Mawson catchment was found to vary from 50 to 70% with a medium value of 55%. The critical storm duration on the Curtin catchment was found to vary from two hours for the 2 Year ARI design storm to 45 minutes for the 100 Year ARI design storm. The calibrated runoff coefficients for the Curtin catchment ranged from 35% to 50% with a medium value of 40%.

Design Storm ARI (Years)	Mawson Catchment Flood Frequency Flow ($m^3/_s$)	Mawson Catchment Calibrated Runoff Coefficient (%)	Curtin Catchment Flood Frequency Flow ($m^3/_s$)	Curtin Catchment Calibrated Runoff Coefficient (%)
2	22	65	64	50
5	29	55	83	45
10	34	50	97	40
20	43	50	119	35
50	63	70	150	35
100	80	55	180	40

Table 15 - RORB Calibration Results

The results indicate that the calibrated runoff coefficients do not vary appreciably with design storm recurrence interval. However, a significant variation in the medium runoff coefficients between the two catchment was recorded. This result is therefore inconclusive in determining a parameter value for use on all ACT urban catchments. Further discussion on the above results is contained in Appendix C.

6.5.2 Application of Constant Runoff Coefficient

The adoption of a constant runoff coefficient for application to both the Mawson and Curtin catchments was carried out to assess the models ability to reproduce the adopted flood frequency curve peak flows.

In view of the previous results a constant runoff coefficient of 45% was adopted. The subsequent RORB predicted flows for each of the catchments are shown in Table 16.

Design Storm ARI (Years)	Mawson Catchment Flood Frequency Flow (m^3/s)	Mawson Catchment Modelled Flow (m^3/s)	Curtin Catchment Flood Frequency Flow (m^3/s)	Curtin Catchment Modelled Flow (m^3/s)
2	22	17	64	61
5	29	26	83	90
10	34	31	97	108
20	43	40	119	140
50	63	47	150	174
100	80	55	180	190

Table 16 - RORB Predicted Flows Using Runoff Coefficient 45%

The above RORB predicted flows range from within -31% to +18% of the flood frequency flows. The poorest results occurred on the Mawson catchment. Further discussion of the above results is contained in Appendix C.

6.5.3 RORB Calibration of LP3 Flood Frequency Curves

Further RORB analysis was carried out against the LP3 generated flood frequency curves. The same calibration procedure was adopted to that previously used on the composite flood frequency curves. Using a constant calibrated median runoff coefficient of 45% the RORB model was able to reproduce the full range of the Curtin LP3 flood frequency curve flows to within -5% to +18%.

The same procedure was repeated on the Mawson catchment using a calibrated constant runoff coefficient of 55%. The RORB model was able to predict the flood frequency curve flows to within -16% to +6%.

It should be noted that the above calibrated constant runoff coefficients were determined by calibration against the 2, 5 and 10 Year ARI LP3 flood frequency flows. As the medium value calibrated runoff coefficient varied from catchment to catchment, the results are again seen to be inconclusive. Further discussion and tabulation of results regarding the LP3 RORB modelling is contained in Appendix C.

7.0 RECOMMENDED MODEL PROCEDURES AND PARAMETER VALUES

7.1 RAFTS

The RAFTS analysis in this study involved the use of two approaches to rainfall loss estimation. They were the initial/continuing loss approach and the infiltration/water balance procedure approach which utilizes the Australian Representative Basin Program (ARBM).

The Giralang catchment analysis results were based on a 41 node RAFTS-XP network which is equivalent to an average sub-catchment size of approximately 2.2 hectares. The Mawson catchment analysis results were based on a 180 node network which is equivalent to an average sub-catchment size of approximately 2.3 hectares.

The initial/continuing loss model analysis failed to produce a single set of loss rates which were able to model the full range of flood frequency curve flows on the catchments modelled. The results indicated the peak flows are sensitive to the losses adopted.

Analysis was carried out on the Giralang and Mawson catchments to determine the effect of the level of sub-catchment discretisation adopted. The conclusion to be drawn from this analysis was that the modelled peak flow increases with increasing catchment discretisation. Alternatively the modelled peak flows decrease with decreasing catchment discretisation.

The RAFTS ARBM loss model approach to calibration was to vary the initial catchment wetness conditions until a volume calibration was achieved against the targetted flood hydrograph. Following this a further calibration against the targetted peak flow was carried out by varying the catchment surface roughness parameters.

The results of the RAFTS ARBM modelling produced a high level of calibration achievement particular on the Giralang catchment which is well gauged. The design storm event modelling against the catchment flood frequency curves also revealed that single set of model parameter values was able to reasonably predict a full range of the ARI flood frequency flows. The results of the design storm analysis should be viewed with some caution due to the uncertainty which exists regarding the catchment flood frequency curves, particularly at the higher magnitude ARI events.

However, it is concluded that based on the RAFTS historical storm and design storm analysis carried out in this study, the following recommendations can be made in terms of procedure and parameter value adoption for use on ACT urban catchments:

(i) The use of the ARBM loss model is favoured over the initial/continuing loss model due to the ARBM's ability to model a range of ARI events with a single set of model parameters;

(ii) The use of the ARBM model on gauged urban catchments should involve the adoption of the parameter values listed in Table 5 of this report;

(iii) The initial catchment wetness conditions represented by the initial upper and lower soil store capacities should be set at 80% of the available capacity for design storm modelling. This corresponds to values of 20.0 mm and 40.0 mm for the initial upper and lower soil store values when applied to the Table 5 soil storage capacities.

(iv) The surface runoff routing parameters recommended for adoption are 0.015 and 0.040 for the impervious and pervious surface roughness parameters respectively. In addition, it is recommended that the model default non-linearity parameter value of 0.285 be adopted.

7.2 ILSAX

The ILSAX modelling carried out in this study involved the calibration of the catchment models against historical storm events. The calibration approach adopted was to allocate constant values to all parameters excluding the antecedant moisture condition (AMC) parameter. The AMC parameter value was then varied until the targetted peak flow or hydrograph volume was obtained. This approach was considered to be similar in principle to the RAFTS calibration procedure which was to vary the initial catchment wetness level.

Alternative approaches to ILSAX calibration could have been to adopt a constant AMC and to vary the initial catchment depression storage values. The approach was not considered to be favourable due to the high level of uncertainty associated with initial loss rates on urban catchments.

A successful level of calibration was achieved on both the historical and design storm ILSAX modelling. A single set of ILSAX parameters was able to predict the flood frequency curve flows for each of the ARI events up to 100 Years ARI to an acceptable level. Consequently, it is possible to make the following recommendations in terms of procedure and parameter value adoption for ILSAX modelling on urban catchments in the ACT:

(i) ILSAX calibrations on historical storms should proceed by adopting constant values for all parameters excluding the AMC which should be varied during the calibration;

(ii) Design storm event modelling should adopt the following rainfall loss rate parameters:

Impervious (Paved) Depression Storage:	1mm
Pervious (Grassed) Depression Storage:	5mm
Soil Type:	3.0
AMC:	3.2

(iii) The procedures employed to calculate the travel times or times of concentration for sub-catchment pervious runoff should be based on Friends Equation as was recommended in the earlier report "Drainage Design Practice for Land Development in the ACT - Part 1: Rational Formula Procedures".

The travel time or time of concentration for all impervious areas should be adopted globally and set at six minutes.

7.3 RORB

The RORB analysis carried out in this study followed the recommended procedure guidelines contained in the user manual. The calibration procedure adopted in the study and recommended for any RORB modelling of ACT ungauged urban catchments is as follows:

(i) Design storm losses be represented by an initial loss followed by a runoff coefficient. The loss parameter should be set at the following values:

Initial loss:	10mm
Runoff Coefficient:	45%

(ii) The runoff routing parameters m and Kc be adopted as follows:

	m	(adopt RORB default) =	0.8
	Kc	(adopt RORB default emperical equation)=	$2.2A^{0.5}$
where	A	catchment area (km^2)	

The above parameters are recommended for ungauged catchments. All gauged catchments should be calibrated against recorded storm events using the approach adopted in this study which was to use the runoff coefficient as the calibration parameter. It should be stressed that the RORB analysis carried out as part of this study did not achieve a conclusive outcome in relation to the above recommended runoff coefficient for application to all ACT urban catchments. The recommended runoff coefficient value of 45% was the value which was best able to reproduce the targetted flows in this study. The RORB model should therefore be used with some caution when modelling ungauged catchments.

7.4 TRUNK VERSUS NEIGHBOURHOOD DRAINAGE PROCEDURES

It is considered that no conclusions can be drawn to regard to different modelling procedures being adopted for trunk and neighbourhood drainage systems. The analysis carried out in this study did not reveal any clear trends in differing catchment response behaviour between the two levels. It is therefore considered that the recommendations for the three rainfall/runoff models be applied without differences at both the trunk and neighbourhood drainage level.

8.0 REFERENCES

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APPENDIX A

RAFTS DATA FILE

GIRALANG RAFTS DATA FILE

```
! Rafts 2.72 data file generated by RaftsXP 2.72
 1    1    2          272
GIRALANG URBAN CATCHMENT 5 year storm
   .5      1     2     0       1.              240911122 0 0000
! --- STACKED STORM DATA - Storm no.5
! --- STORM DATA
      120.                      -1        20 0
0.53        2.79        3.66        3.11        5.42        5.13        0.41        3.67
0.21        0.03        0.          0.          0.          0.          0.          0.
0.          0.          0.          0.
! --- LINK DEFINITION DATA
1 1.00      130         00      10       4                     4
1 2.00      130         00      10       5                     5
0 1.00 2.000
1 1.01      130         00      10       6                     6
1 1.02      130         00      10       14                    14
1 1.03      130         00      10       15                    15
1 3.00      130         00      10       16                    16
0 1.03 3.000
1 1.04      130         00      10       17                    17
1 1.05      130         00      10       18                    18
1 1.06      130         00      10       19                    19
1 4.00      130         00      10       1                     1
1 4.01      130         00      10       2                     2
1 4.02      130         00      10       3                     3
0 1.06 4.020
1 1.07      130         00      10       20                    20
1 1.08      130         00      10       23                    23
1 5.00      130         00      10       21                    21
1 5.01      130         00      10       22                    22
1 6.00      130         00      10       7                     7
1 6.01      130         00      10       8                     8
1 6.02      130         00      10       9                     9
1 7.00      130         00      10       10                    10
1 7.01      130         00      10       11                    11
1 7.02      130         00      10       12                    12
0 6.02 7.020
1 6.03      130         00      10       13                    13
0 1.08 5.010
0 1.08 6.030
1 1.09      130         00      10       24                    24
1 1.10      130         00      10       25                    25
1 1.11      130         00      10       26                    26
1 1.12      130         00      10       27                    27
1 1.13      130         00      10       28                    28
1 8.00      130         00      10       29                    29
1 8.01      130         00      10       30                    30
1 8.02      130         00      10       31                    31
1 9.00      130         00      10       33                    33
1 9.01      130         00      10       34                    34
110.00      130         00      10       32                    32
0 9.0110.000
1 9.02      130         00      10       35                    35
1 9.03      130         00      10       36                    36
1 9.04      130         00      10       37                    37
1 9.05      130         00      10       38                    38
0 8.02 9.050
1 8.03      130         00      10       39                    39
1 8.04      130         00      10       40                    40
0 1.13 8.040
1 1.14      130         01      10       41                    41
1 1.15      130         00      10       44                    44
1 1.16      110         01      10       d                     d
```

```
0
! --- LINK    1.00
! ------ FIRST SUBCATCHMENT DATA
      16.9       0.0       5.4                                      0 0.04
     99999

! ------ ARBM DATA
        0.5       1.0       1.0       25.       50.       10.0      10.0      0.7
        0.        0.        0.        22.5      45.       0.        1.0
        3.0       .33       6.8       .5        .05       0.94      0.9       0.7
! ------ LAG DATA
.0299
! --- LINK    2.00
! ------ FIRST SUBCATCHMENT DATA
      2.35       0.0       8.3                                      1 0.04
     99999

! ------ ARBM DATA
        0.5       1.0       1.0       25.       50.       10.0      10.0      0.7
        0.        0.        0.        22.5      45.       0.        1.0
        3.0       .33       6.8       .5        .05       0.94      0.9       0.7
! ------ SECOND SUBCATCHMENT DATA
       .67      100.       8.3                                      00.015
     99999

! ------ ARBM DATA
        0.5       1.0       1.0       25.       50.       10.0      10.0      0.7
        0.        0.        0.        22.5      45.       0.        1.0
        3.0       .33       6.8       .5        .05       0.94      0.9       0.7
! ------ LAG DATA
.2100
! --- LINK    1.01
! ------ FIRST SUBCATCHMENT DATA
       .73       0.0       8.3                                      1 0.04
     99999

! ------ ARBM DATA
        0.5       1.0       1.0       25.       50.       10.0      10.0      0.7
        0.        0.        0.        22.5      45.       0.        1.0
        3.0       .33       6.8       .5        .05       0.94      0.9       0.7
! ------ SECOND SUBCATCHMENT DATA
        .3      100.       8.3                                      00.015
     99999

! ------ ARBM DATA
        0.5       1.0       1.0       25.       50.       10.0      10.0      0.7
        0.        0.        0.        22.5      45.       0.        1.0
        3.0       .33       6.8       .5        .05       0.94      0.9       0.7
! ------ LAG DATA
.3699
! --- LINK    1.02
! ------ FIRST SUBCATCHMENT DATA
       .41       0.0       7.5                                      1 0.04
     99999

! ------ ARBM DATA
        0.5       1.0       1.0       25.       50.       10.0      10.0      0.7
        0.        0.        0.        22.5      45.       0.        1.0
        3.0       .33       6.8       .5        .05       0.94      0.9       0.7
! ------ SECOND SUBCATCHMENT DATA
       .55      100.       7.5                                      00.015
     99999
```

```
! ------ ARBM DATA
      0.5       1.0       1.0       25.       50.      10.0      10.0      0.7
       0.        0.        0.      22.5       45.        0.       1.0
      3.0       .33       6.8        .5       .05      0.94       0.9      0.7
! ------ LAG DATA
.3202
! --- LINK    1.03
! ------ FIRST SUBCATCHMENT DATA
.67             0.05.5                                         1 0.04
     99999

! ------ ARBM DATA
      0.5       1.0       1.0       25.       50.      10.0      10.0      0.7
       0.        0.        0.      22.5       45.        0.       1.0
      3.0       .33       6.8        .5       .05      0.94       0.9      0.7
! ------ SECOND SUBCATCHMENT DATA
.77       100       5.5                                        00.015
     99999

! ------ ARBM DATA
      0.5       1.0       1.0       25.       50.      10.0      10.0      0.7
       0.        0.        0.      22.5       45.        0.       1.0
      3.0       .33       6.8        .5       .05      0.94       0.9      0.7
! ------ LAG DATA
.2902
! --- LINK    3.00
! ------ FIRST SUBCATCHMENT DATA
.9              0.07                                           1 0.04
     99999

! ------ ARBM DATA
      0.5       1.0       1.0       25.       50.      10.0      10.0      0.7
       0.        0.        0.      22.5       45.        0.       1.0
      3.0       .33       6.8        .5       .05      0.94       0.9      0.7
! ------ SECOND SUBCATCHMENT DATA
.3        100       7                                          00.015
     99999

! ------ ARBM DATA
      0.5       1.0       1.0       25.       50.      10.0      10.0      0.7
       0.        0.        0.      22.5       45.        0.       1.0
      3.0       .33       6.8        .5       .05      0.94       0.9      0.7
! ------ LAG DATA
.3699
! --- LINK    1.04
! ------ FIRST SUBCATCHMENT DATA
.66             0.05                                           1 0.04
     99999

! ------ ARBM DATA
      0.5       1.0       1.0       25.       50.      10.0      10.0      0.7
       0.        0.        0.      22.5       45.        0.       1.0
      3.0       .33       6.8        .5       .05      0.94       0.9      0.7
! ------ SECOND SUBCATCHMENT DATA
.48       100       5                                          00.015
     99999

! ------ ARBM DATA
      0.5       1.0       1.0       25.       50.      10.0      10.0      0.7
       0.        0.        0.      22.5       45.        0.       1.0
      3.0       .33       6.8        .5       .05      0.94       0.9      0.7
! ------ LAG DATA
.3001
```

```
! --- LINK    1.05
! ------ FIRST SUBCATCHMENT DATA
.21             0.08.3                                                    1 0.04
    99999

! ------ ARBM DATA
       0.5        1.0        1.0        25.        50.      10.0      10.0       0.7
        0.         0.         0.       22.5        45.        0.       1.0
       3.0        .33        6.8         .5        .05      0.94       0.9       0.7
! ------ SECOND SUBCATCHMENT DATA
.25      100        8.3                                                   00.015
    99999

! ------ ARBM DATA
       0.5        1.0        1.0        25.        50.      10.0      10.0       0.7
        0.         0.         0.       22.5        45.        0.       1.0
       3.0        .33        6.8         .5        .05      0.94       0.9       0.7
! ------ LAG DATA
.1902
! --- LINK    1.06
! ------ FIRST SUBCATCHMENT DATA
.6              0.08.3                                                    1 0.04
    99999

! ------ ARBM DATA
       0.5        1.0        1.0        25.        50.      10.0      10.0       0.7
        0.         0.         0.       22.5        45.        0.       1.0
       3.0        .33        6.8         .5        .05      0.94       0.9       0.7
! ------ SECOND SUBCATCHMENT DATA
.5       100        8.3                                                   00.015
    99999

! ------ ARBM DATA
       0.5        1.0        1.0        25.        50.      10.0      10.0       0.7
        0.         0.         0.       22.5        45.        0.       1.0
       3.0        .33        6.8         .5        .05      0.94       0.9       0.7
! ------ LAG DATA
.2098
! --- LINK    4.00
! ------ FIRST SUBCATCHMENT DATA
      2.21        0.0        4.2                                          1 0.04
    99999

! ------ ARBM DATA
       0.5        1.0        1.0        25.        50.      10.0      10.0       0.7
        0.         0.         0.       22.5        45.        0.       1.0
       3.0        .33        6.8         .5        .05      0.94       0.9       0.7
! ------ SECOND SUBCATCHMENT DATA
       .81       100.        4.2                                          00.015
    99999

! ------ ARBM DATA
       0.5        1.0        1.0        25.        50.      10.0      10.0       0.7
        0.         0.         0.       22.5        45.        0.       1.0
       3.0        .33        6.8         .5        .05      0.94       0.9       0.7
! ------ LAG DATA
.2
! --- LINK    4.01
! ------ FIRST SUBCATCHMENT DATA
.1              0.012                                                     1 0.04
    99999

! ------ ARBM DATA
```

```
      0.5       1.0       1.0       25.       50.       10.0      10.0      0.7
       0.        0.        0.       22.5      45.        0.        1.0
      3.0       .33       6.8        .5       .05       0.94      0.9       0.7
! ------ SECOND SUBCATCHMENT DATA
.2       100       12                                          00.015
    99999

! ------ ARBM DATA
      0.5       1.0       1.0       25.       50.       10.0      10.0      0.7
       0.        0.        0.       22.5      45.        0.        1.0
      3.0       .33       6.8        .5       .05       0.94      0.9       0.7
! ------ LAG DATA
.3597
! --- LINK    4.02
! ------ FIRST SUBCATCHMENT DATA
.22             0.011                                          1 0.04
    99999

! ------ ARBM DATA
      0.5       1.0       1.0       25.       50.       10.0      10.0      0.7
       0.        0.        0.       22.5      45.        0.        1.0
      3.0       .33       6.8        .5       .05       0.94      0.9       0.7
! ------ SECOND SUBCATCHMENT DATA
.45      100       11                                          00.015
    99999

! ------ ARBM DATA
      0.5       1.0       1.0       25.       50.       10.0      10.0      0.7
       0.        0.        0.       22.5      45.        0.        1.0
      3.0       .33       6.8        .5       .05       0.94      0.9       0.7
! ------ LAG DATA
.4996
! --- LINK    1.07
! ------ FIRST SUBCATCHMENT DATA
.001            0.05                                           1 0.04
    99999

! ------ ARBM DATA
      0.5       1.0       1.0       25.       50.       10.0      10.0      0.7
       0.        0.        0.       22.5      45.        0.        1.0
      3.0       .33       6.8        .5       .05       0.94      0.9       0.7
! ------ SECOND SUBCATCHMENT DATA
.2       100       5                                           00.015
    99999

! ------ ARBM DATA
      0.5       1.0       1.0       25.       50.       10.0      10.0      0.7
       0.        0.        0.       22.5      45.        0.        1.0
      3.0       .33       6.8        .5       .05       0.94      0.9       0.7
! ------ LAG DATA
.1
! --- LINK    1.08
! ------ FIRST SUBCATCHMENT DATA
7.83            0.05                                           1 0.04
    99999

! ------ ARBM DATA
      0.5       1.0       1.0       25.       50.       10.0      10.0      0.7
       0.        0.        0.       22.5      45.        0.        1.0
      3.0       .33       6.8        .5       .05       0.94      0.9       0.7
! ------ SECOND SUBCATCHMENT DATA
.18      100       5                                           00.015
    99999
```

```
! ------ ARBM DATA
       0.5        1.0        1.0        25.        50.       10.0       10.0        0.7
        0.         0.         0.       22.5        45.        0.        1.0
       3.0        .33        6.8         .5        .05       0.94        0.9        0.7
! ------ LAG DATA
.2
! --- LINK    5.00
! ------ FIRST SUBCATCHMENT DATA
1.72            0.04.6                                           1 0.04
     99999

! ------ ARBM DATA
       0.5        1.0        1.0        25.        50.       10.0       10.0        0.7
        0.         0.         0.       22.5        45.        0.        1.0
       3.0        .33        6.8         .5        .05       0.94        0.9        0.7
! ------ SECOND SUBCATCHMENT DATA
.91       100        4.6                                          00.015
     99999

! ------ ARBM DATA
       0.5        1.0        1.0        25.        50.       10.0       10.0        0.7
        0.         0.         0.       22.5        45.        0.        1.0
       3.0        .33        6.8         .5        .05       0.94        0.9        0.7
! ------ LAG DATA
.1700
! --- LINK    5.01
! ------ FIRST SUBCATCHMENT DATA
.26             0.03.2                                           1 0.04
     99999

! ------ ARBM DATA
       0.5        1.0        1.0        25.        50.       10.0       10.0        0.7
        0.         0.         0.       22.5        45.        0.        1.0
       3.0        .33        6.8         .5        .05       0.94        0.9        0.7
! ------ SECOND SUBCATCHMENT DATA
.11       100        3.2                                          00.015
     99999

! ------ ARBM DATA
       0.5        1.0        1.0        25.        50.       10.0       10.0        0.7
        0.         0.         0.       22.5        45.        0.        1.0
       3.0        .33        6.8         .5        .05       0.94        0.9        0.7
! ------ LAG DATA
.5004
! --- LINK    6.00
! ------ FIRST SUBCATCHMENT DATA
1.84            0.09                                             1 0.04
     99999

! ------ ARBM DATA
       0.5        1.0        1.0        25.        50.       10.0       10.0        0.7
        0.         0.         0.       22.5        45.        0.        1.0
       3.0        .33        6.8         .5        .05       0.94        0.9        0.7
! ------ SECOND SUBCATCHMENT DATA
.43       100         9                                           00.015
     99999

! ------ ARBM DATA
       0.5        1.0        1.0        25.        50.       10.0       10.0        0.7
        0.         0.         0.       22.5        45.        0.        1.0
       3.0        .33        6.8         .5        .05       0.94        0.9        0.7
! ------ LAG DATA
```

```
.2301
! --- LINK    6.01
! ------ FIRST SUBCATCHMENT DATA
2.39             0.013                                                1 0.04
     99999

! ------ ARBM DATA
       0.5       1.0       1.0       25.       50.      10.0      10.0       0.7
        0.        0.        0.      22.5       45.        0.       1.0
       3.0       .33       6.8        .5       .05      0.94       0.9       0.7
! ------ SECOND SUBCATCHMENT DATA
.37        100        13                                         00.015
     99999

! ------ ARBM DATA
       0.5       1.0       1.0       25.       50.      10.0      10.0       0.7
        0.        0.        0.      22.5       45.        0.       1.0
       3.0       .33       6.8        .5       .05      0.94       0.9       0.7
! ------ LAG DATA
.2497
! --- LINK    6.02
! ------ FIRST SUBCATCHMENT DATA
.001             0.0.2                                                1 0.04
     99999

! ------ ARBM DATA
       0.5       1.0       1.0       25.       50.      10.0      10.0       0.7
        0.        0.        0.      22.5       45.        0.       1.0
       3.0       .33       6.8        .5       .05      0.94       0.9       0.7
! ------ SECOND SUBCATCHMENT DATA
.2         100        .2                                          00.015
     99999

! ------ ARBM DATA
       0.5       1.0       1.0       25.       50.      10.0      10.0       0.7
        0.        0.        0.      22.5       45.        0.       1.0
       3.0       .33       6.8        .5       .05      0.94       0.9       0.7
! ------ LAG DATA
.1299
! --- LINK    7.00
! ------ FIRST SUBCATCHMENT DATA
      1.05       0.0       6.5                                        1 0.04
     99999

! ------ ARBM DATA
       0.5       1.0       1.0       25.       50.      10.0      10.0       0.7
        0.        0.        0.      22.5       45.        0.       1.0
       3.0       .33       6.8        .5       .05      0.94       0.9       0.7
! ------ SECOND SUBCATCHMENT DATA
       .42      100.       6.5                                    00.015
     99999

! ------ ARBM DATA
       0.5       1.0       1.0       25.       50.      10.0      10.0       0.7
        0.        0.        0.      22.5       45.        0.       1.0
       3.0       .33       6.8        .5       .05      0.94       0.9       0.7
! ------ LAG DATA
.3397
! --- LINK    7.01
! ------ FIRST SUBCATCHMENT DATA
      1.58       0.0        9.                                        1 0.04
     99999
```

```
! ------ ARBM DATA
       0.5        1.0        1.0        25.        50.       10.0       10.0        0.7
        0.         0.         0.       22.5        45.         0.        1.0
       3.0        .33        6.8         .5        .05       0.94        0.9        0.7
! ------ SECOND SUBCATCHMENT DATA
.67        100        9                                             00.015
     99999

! ------ ARBM DATA
       0.5        1.0        1.0        25.        50.       10.0       10.0        0.7
        0.         0.         0.       22.5        45.         0.        1.0
       3.0        .33        6.8         .5        .05       0.94        0.9        0.7
! ------ LAG DATA
.2698
! --- LINK    7.02
! ------ FIRST SUBCATCHMENT DATA
       .92        0.0        9.4                                    1 0.04
     99999

! ------ ARBM DATA
       0.5        1.0        1.0        25.        50.       10.0       10.0        0.7
        0.         0.         0.       22.5        45.         0.        1.0
       3.0        .33        6.8         .5        .05       0.94        0.9        0.7
! ------ SECOND SUBCATCHMENT DATA
        .3       100.        9.4                                    00.015
     99999

! ------ ARBM DATA
       0.5        1.0        1.0        25.        50.       10.0       10.0        0.7
        0.         0.         0.       22.5        45.         0.        1.0
       3.0        .33        6.8         .5        .05       0.94        0.9        0.7
! ------ LAG DATA
.1700
! --- LINK    6.03
! ------ FIRST SUBCATCHMENT DATA
1.11             0.07.5                                             1 0.04
     99999

! ------ ARBM DATA
       0.5        1.0        1.0        25.        50.       10.0       10.0        0.7
        0.         0.         0.       22.5        45.         0.        1.0
       3.0        .33        6.8         .5        .05       0.94        0.9        0.7
! ------ SECOND SUBCATCHMENT DATA
.22        100        7.5                                           00.015
     99999

! ------ ARBM DATA
       0.5        1.0        1.0        25.        50.       10.0       10.0        0.7
        0.         0.         0.       22.5        45.         0.        1.0
       3.0        .33        6.8         .5        .05       0.94        0.9        0.7
! ------ LAG DATA
.8803
! --- LINK    1.09
! ------ FIRST SUBCATCHMENT DATA
2.03             0.04                                               1 0.04
     99999

! ------ ARBM DATA
       0.5        1.0        1.0        25.        50.       10.0       10.0        0.7
        0.         0.         0.       22.5        45.         0.        1.0
       3.0        .33        6.8         .5        .05       0.94        0.9        0.7
! ------ SECOND SUBCATCHMENT DATA
.96        100        4                                             00.015
```

```
     99999

! ------ ARBM DATA
       0.5       1.0       1.0       25.       50.       10.0      10.0      0.7
        0.        0.        0.      22.5       45.        0.       1.0
       3.0       .33       6.8        .5       .05      0.94       0.9      0.7
! ------ LAG DATA
.3002
! --- LINK    1.10
! ------ FIRST SUBCATCHMENT DATA
3.05         0.03.4                                     1 0.04
     99999

! ------ ARBM DATA
       0.5       1.0       1.0       25.       50.       10.0      10.0      0.7
        0.        0.        0.      22.5       45.        0.       1.0
       3.0       .33       6.8        .5       .05      0.94       0.9      0.7
! ------ SECOND SUBCATCHMENT DATA
1.69      100       3.4                                 00.015
     99999

! ------ ARBM DATA
       0.5       1.0       1.0       25.       50.       10.0      10.0      0.7
        0.        0.        0.      22.5       45.        0.       1.0
       3.0       .33       6.8        .5       .05      0.94       0.9      0.7
! ------ LAG DATA
.2399
! --- LINK    1.11
! ------ FIRST SUBCATCHMENT DATA
     3.08       0.0       2.5                           1 0.04
     99999

! ------ ARBM DATA
       0.5       1.0       1.0       25.       50.       10.0      10.0      0.7
        0.        0.        0.      22.5       45.        0.       1.0
       3.0       .33       6.8        .5       .05      0.94       0.9      0.7
! ------ SECOND SUBCATCHMENT DATA
      .67      100.       2.5                           00.015
     99999

! ------ ARBM DATA
       0.5       1.0       1.0       25.       50.       10.0      10.0      0.7
        0.        0.        0.      22.5       45.        0.       1.0
       3.0       .33       6.8        .5       .05      0.94       0.9      0.7
! ------ LAG DATA
.3298
! --- LINK    1.12
! ------ FIRST SUBCATCHMENT DATA
     2.05       0.0       6.4                           1 0.04
     99999

! ------ ARBM DATA
       0.5       1.0       1.0       25.       50.       10.0      10.0      0.7
        0.        0.        0.      22.5       45.        0.       1.0
       3.0       .33       6.8        .5       .05      0.94       0.9      0.7
! ------ SECOND SUBCATCHMENT DATA
     1.08      100.       6.4                           00.015
     99999

! ------ ARBM DATA
       0.5       1.0       1.0       25.       50.       10.0      10.0      0.7
        0.        0.        0.      22.5       45.        0.       1.0
       3.0       .33       6.8        .5       .05      0.94       0.9      0.7
```

```
! ------ LAG DATA
.3703
! --- LINK    1.13
! ------ FIRST SUBCATCHMENT DATA
1.83              0.06                                               1 0.04
     99999

! ------ ARBM DATA
       0.5       1.0       1.0       25.       50.      10.0      10.0        0.7
        0.        0.        0.      22.5       45.        0.       1.0
       3.0       .33       6.8        .5       .05      0.94       0.9        0.7
! ------ SECOND SUBCATCHMENT DATA
1.17       100       6                                               00.015
     99999

! ------ ARBM DATA
       0.5       1.0       1.0       25.       50.      10.0      10.0        0.7
        0.        0.        0.      22.5       45.        0.       1.0
       3.0       .33       6.8        .5       .05      0.94       0.9        0.7
! ------ LAG DATA
.3096
! --- LINK    8.00
! ------ FIRST SUBCATCHMENT DATA
      1.13       0.0        7.                                       1 0.04
     99999

! ------ ARBM DATA
       0.5       1.0       1.0       25.       50.      10.0      10.0        0.7
        0.        0.        0.      22.5       45.        0.       1.0
       3.0       .33       6.8        .5       .05      0.94       0.9        0.7
! ------ SECOND SUBCATCHMENT DATA
.9         100       7                                               00.015
     99999

! ------ ARBM DATA
       0.5       1.0       1.0       25.       50.      10.0      10.0        0.7
        0.        0.        0.      22.5       45.        0.       1.0
       3.0       .33       6.8        .5       .05      0.94       0.9        0.7
! ------ LAG DATA
.4994
! --- LINK    8.01
! ------ FIRST SUBCATCHMENT DATA
       .86       0.0        8.                                       1 0.04
     99999

! ------ ARBM DATA
       0.5       1.0       1.0       25.       50.      10.0      10.0        0.7
        0.        0.        0.      22.5       45.        0.       1.0
       3.0       .33       6.8        .5       .05      0.94       0.9        0.7
! ------ SECOND SUBCATCHMENT DATA
       .38      100.        8.                                       00.015
     99999

! ------ ARBM DATA
       0.5       1.0       1.0       25.       50.      10.0      10.0        0.7
        0.        0.        0.      22.5       45.        0.       1.0
       3.0       .33       6.8        .5       .05      0.94       0.9        0.7
! ------ LAG DATA
.3194
! --- LINK    8.02
! ------ FIRST SUBCATCHMENT DATA
.49              0.04.2                                              1 0.04
     99999
```

```
! ------ ARBM DATA
       0.5        1.0        1.0        25.        50.        10.0       10.0        0.7
        0.         0.         0.       22.5        45.         0.        1.0
       3.0        .33        6.8         .5        .05       0.94        0.9        0.7
! ------ SECOND SUBCATCHMENT DATA
.14        100        4.2                                     00.015
     99999

! ------ ARBM DATA
       0.5        1.0        1.0        25.        50.        10.0       10.0        0.7
        0.         0.         0.       22.5        45.         0.        1.0
       3.0        .33        6.8         .5        .05       0.94        0.9        0.7
! ------ LAG DATA
.5888
! --- LINK    9.00
! ------ FIRST SUBCATCHMENT DATA
       .29        0.0        10.                              1 0.04
     99999

! ------ ARBM DATA
       0.5        1.0        1.0        25.        50.        10.0       10.0        0.7
        0.         0.         0.       22.5        45.         0.        1.0
       3.0        .33        6.8         .5        .05       0.94        0.9        0.7
! ------ SECOND SUBCATCHMENT DATA
.13        100        10                                      00.015
     99999

! ------ ARBM DATA
       0.5        1.0        1.0        25.        50.        10.0       10.0        0.7
        0.         0.         0.       22.5        45.         0.        1.0
       3.0        .33        6.8         .5        .05       0.94        0.9        0.7
! ------ LAG DATA
.5408
! --- LINK    9.01
! ------ FIRST SUBCATCHMENT DATA
.83              0.013.6                                      1 0.04
     99999

! ------ ARBM DATA
       0.5        1.0        1.0        25.        50.        10.0       10.0        0.7
        0.         0.         0.       22.5        45.         0.        1.0
       3.0        .33        6.8         .5        .05       0.94        0.9        0.7
! ------ SECOND SUBCATCHMENT DATA
.19        100        13.6                                    00.015
     99999

! ------ ARBM DATA
       0.5        1.0        1.0        25.        50.        10.0       10.0        0.7
        0.         0.         0.       22.5        45.         0.        1.0
       3.0        .33        6.8         .5        .05       0.94        0.9        0.7
! ------ LAG DATA
.2202
! --- LINK   10.00
! ------ FIRST SUBCATCHMENT DATA
2.02             0.011                                        1 0.04
     99999

! ------ ARBM DATA
       0.5        1.0        1.0        25.        50.        10.0       10.0        0.7
        0.         0.         0.       22.5        45.         0.        1.0
       3.0        .33        6.8         .5        .05       0.94        0.9        0.7
! ------ SECOND SUBCATCHMENT DATA
```

```
.22       100       11                                                  00.015
     99999

! ------ ARBM DATA
       0.5       1.0       1.0       25.       50.       10.0      10.0      0.7
        0.        0.        0.       22.5      45.        0.        1.0
       3.0       .33       6.8        .5       .05       0.94       0.9      0.7
! ------ LAG DATA
.3794
! --- LINK    9.02
! ------ FIRST SUBCATCHMENT DATA
1.7              0.09.4                                                1 0.04
     99999

! ------ ARBM DATA
       0.5       1.0       1.0       25.       50.       10.0      10.0      0.7
        0.        0.        0.       22.5      45.        0.        1.0
       3.0       .33       6.8        .5       .05       0.94       0.9      0.7
! ------ SECOND SUBCATCHMENT DATA
       .63      100.       9.4                                        00.015
     99999

! ------ ARBM DATA
       0.5       1.0       1.0       25.       50.       10.0      10.0      0.7
        0.        0.        0.       22.5      45.        0.        1.0
       3.0       .33       6.8        .5       .05       0.94       0.9      0.7
! ------ LAG DATA
.2300
! --- LINK    9.03
! ------ FIRST SUBCATCHMENT DATA
.36              0.08.8                                                1 0.04
     99999

! ------ ARBM DATA
       0.5       1.0       1.0       25.       50.       10.0      10.0      0.7
        0.        0.        0.       22.5      45.        0.        1.0
       3.0       .33       6.8        .5       .05       0.94       0.9      0.7
! ------ SECOND SUBCATCHMENT DATA
.33       100        8.8                                              00.015
     99999

! ------ ARBM DATA
       0.5       1.0       1.0       25.       50.       10.0      10.0      0.7
        0.        0.        0.       22.5      45.        0.        1.0
       3.0       .33       6.8        .5       .05       0.94       0.9      0.7
! ------ LAG DATA
.1501
! --- LINK    9.04
! ------ FIRST SUBCATCHMENT DATA
       .44       0.0      10.7                                         1 0.04
     99999

! ------ ARBM DATA
       0.5       1.0       1.0       25.       50.       10.0      10.0      0.7
        0.        0.        0.       22.5      45.        0.        1.0
       3.0       .33       6.8        .5       .05       0.94       0.9      0.7
! ------ SECOND SUBCATCHMENT DATA
       .34      100.      10.7                                        00.015
     99999

! ------ ARBM DATA
       0.5       1.0       1.0       25.       50.       10.0      10.0      0.7
        0.        0.        0.       22.5      45.        0.        1.0
```

```
       3.0        .33       6.8         .5        .05      0.94       0.9       0.7
! ------ LAG DATA
.0999
! --- LINK    9.05
! ------ FIRST SUBCATCHMENT DATA
.001            0.0.2                                      1 0.04
     99999

! ------ ARBM DATA
       0.5        1.0       1.0        25.        50.      10.0      10.0       0.7
        0.         0.        0.       22.5        45.        0.       1.0
       3.0        .33       6.8         .5        .05      0.94       0.9       0.7
! ------ SECOND SUBCATCHMENT DATA
.1        100        .2                                    00.015
     99999

! ------ ARBM DATA
       0.5        1.0       1.0        25.        50.      10.0      10.0       0.7
        0.         0.        0.       22.5        45.        0.       1.0
       3.0        .33       6.8         .5        .05      0.94       0.9       0.7
! ------ LAG DATA
.2401
! --- LINK    8.03
! ------ FIRST SUBCATCHMENT DATA
.43             0.03                                       1 0.04
     99999

! ------ ARBM DATA
       0.5        1.0       1.0        25.        50.      10.0      10.0       0.7
        0.         0.        0.       22.5        45.        0.       1.0
       3.0        .33       6.8         .5        .05      0.94       0.9       0.7
! ------ SECOND SUBCATCHMENT DATA
.31       100        3                                     00.015
     99999

! ------ ARBM DATA
       0.5        1.0       1.0        25.        50.      10.0      10.0       0.7
        0.         0.        0.       22.5        45.        0.       1.0
       3.0        .33       6.8         .5        .05      0.94       0.9       0.7
! ------ LAG DATA
.0999
! --- LINK    8.04
! ------ FIRST SUBCATCHMENT DATA
.77             0.05                                       1 0.04
     99999

! ------ ARBM DATA
       0.5        1.0       1.0        25.        50.      10.0      10.0       0.7
        0.         0.        0.       22.5        45.        0.       1.0
       3.0        .33       6.8         .5        .05      0.94       0.9       0.7
! ------ SECOND SUBCATCHMENT DATA
.48       100        5                                     00.015
     99999

! ------ ARBM DATA
       0.5        1.0       1.0        25.        50.      10.0      10.0       0.7
        0.         0.        0.       22.5        45.        0.       1.0
       3.0        .33       6.8         .5        .05      0.94       0.9       0.7
! ------ LAG DATA
.1599
! --- LINK    1.14
! ------ FIRST SUBCATCHMENT DATA
.001            0.0.2                                      1 0.04
```

```
     99999

! ------ ARBM DATA
       0.5       1.0       1.0       25.       50.      10.0      10.0       0.7
        0.        0.        0.      22.5       45.        0.       1.0
       3.0       .33       6.8        .5       .05      0.94       0.9       0.7
! ------ SECOND SUBCATCHMENT DATA
.2         100        .2                                       00.015
     99999

! ------ ARBM DATA
       0.5       1.0       1.0       25.       50.      10.0      10.0       0.7
        0.        0.        0.      22.5       45.        0.       1.0
       3.0       .33       6.8        .5       .05      0.94       0.9       0.7
! ------ LAG DATA
.2298
! ------ GAUGED HYDROGRAPH
  55
0.        0.        0.        0.        0.        0.        .02       .15
.17       .16       .45       .41       .22       .09       .06       .06
.05       .05       .12       .10       .08       8.98      10.09     8.75
6.11      2.94      1.56      .83       .67       3.27      4.85      7.70
9.69      9.39      4.72      2.79      1.94      1.24      .95       .83
.66       .61       .55       .39       .31       .23       .19       .17
.16       .16       .15       .18       .18       .14       .14
0.        6.        12.       18.       24.       30.       36.       42.
48.       54.       60.       66.       72.       78.       84.       90.
96.       102.      108.      114.      120.      126.      132.      138.
144.      150.      156.      162.      168.      174.      180.      186.
192.      198.      204.      210.      216.      222.      228.      234.
240.      246.      252.      258.      264.      270.      276.      282.
288.      294.      300.      306.      312.      318.      500.
! --- LINK    1.15
! ------ FIRST SUBCATCHMENT DATA
      .001       0.0        .2                                     1 0.04
     99999

! ------ ARBM DATA
       0.5       1.0       1.0       25.       50.      10.0      10.0       0.7
        0.        0.        0.      22.5       45.        0.       1.0
       3.0       .33       6.8        .5       .05      0.94       0.9       0.7
! ------ SECOND SUBCATCHMENT DATA
        .1      100.        .2                                     00.015
     99999

! ------ ARBM DATA
       0.5       1.0       1.0       25.       50.      10.0      10.0       0.7
        0.        0.        0.      22.5       45.        0.       1.0
       3.0       .33       6.8        .5       .05      0.94       0.9       0.7
! ------ LAG DATA
.0083
! --- LINK    1.16
! ------ FIRST SUBCATCHMENT DATA
      0.01       0.0       0.2                                     1 0.04
     99999

! ------ ARBM DATA
       0.5       1.0       1.0       25.       50.      10.0      10.0       0.7
        0.        0.        0.      22.5       45.        0.       1.0
       3.0       .33       6.8        .5       .05      0.94       0.9       0.7
! ------ SECOND SUBCATCHMENT DATA
      0.01      100.        1.                                     00.015
     99999
```

```
! ------ ARBM DATA
       0.5        1.0        1.0        25.        50.       10.0       10.0        0.7
        0.         0.         0.       22.5        45.         0.        1.0
       3.0        .33        6.8         .5        .05       0.94        0.9        0.7
! ------ Outlet node dummy link
  0.0
! ------ GAUGED HYDROGRAPH
   27
0.         0.05       0.12       0.10       0.08       8.98       10.09      8.75
6.11       2.94       1.56       0.83       0.67       3.27       4.85       7.70
9.69       9.39       4.72       2.79       1.94       1.24       0.95       0.83
0.66       0.61       0.55
0.         6.         12.        18.        24.        30.        36.        42.
48.        54.        60.        66.        72.        78.        84.        90.
96.        102.       108.       114.       120.       126.       132.       138.
144.       150.       156.
```

APPENDIX B

ILSAX DATA FILES

GIRALANG ILSAX DATA FILE

```
 1    1    1
GIRALANG STUDY 5 YEAR ANALYSIS 60 MINUTE DURATION STORM
ILSAX ANALYSIS
1     3     -1     0     0.013     0     0     0.013
1     5     3.0   3.15     0      0     -1
1
4     60    5   0.5   0.0
1.10  1.98  4.75  3.40  6.56  2.85  2.51  1.61  1.36  0.88  0.74  0.53
0
START OF BRANCH 1
  1   1    -1     -1    0
  0    0     60.0      2.7      450     0
3.02  27.0  6.0  0  73.0  19.0  0

  1   2    -1     -1    0
  0    0     60.0      3.1      525     0
0.30  67.0  6.0  0  33.0  8.0  0

  1   3    -1     -1    0
  0    0     155.0      3.3      525     0
3.02  27.0  6.0  0  73.0  14.0  0

START OF BRANCH 2
  2   1    -1     -1    0
  0    0     30.0      3.4      900     0
16.90  0.0  6.0  0  100.0  34.0  0

START OF BRANCH 3
  3   1    -1     -1    0
  0    0     45.0      2.5      450     0
3.02  22.0  6.0  0  78.0  17.0  0

ADD   3 TO   2

  2   2    -1     -1    0
  0    0    105.0      1.8      1050     0
3.02  27.0  6.0  0  73.0  13.3  0

  2  3    -1     -1    0
  0    0     83.2      3.0      1050     0
8.01  2.0  6.0  0  98.0  12.0  0

  2  4    -1     -1    0
  0    0     81.5      1.7      1050     0
2.63  35.0  6.0  0  65.0  18.0  0

START OF BRANCH 4
  4  1    -1     -1    0
  0    0     95.0      3.3      450     0
0.37  30.0  6.0  0  70.0  17.0  0

ADD   4 TO   2

  2  5    -1     -1    0
  0    0     66.1      1.1      1200     0
```

```
2.27  19.0  6.0  0  81.0  18.0  0

  2  6    -1      -1     0
  0    0     54.8       1.2      1200      0
2.76  13.0  6.0  0  87.0  13.0  0

  2  7    -1      -1     0
  0    0     65.1       1.7      1200      0
0.2  100.0  6.0  0   0.0  14.0  0

ADD   2 TO   1

  1  4    -1      -1     0
  0    0     73.8       3.1      1200      0
1.47  29.0  6.0  0  71.0  12.0  0

  1  5    -1      -1     0
  0    0     45.0       0.9      1200      0
1.33  17.0  6.0  0  83.0  21.0  0

START OF BRANCH 5

  5  1    -1      -1     0
  0    0     70.0       5.4      450      0
0.96  57.0  6.0  0  43.0  15.0  0

  5  2    -1      -1     0
  0    0     55.0       5.4      525      0
1.44  53.0  6.0  0  47.0  17.0  0

  5  3    -1      -1     0
  0    0    130.0       4.1      600      0
1.2  25.0  6.0  0  75.0  6.0  0

START OF BRANCH 6

 6  1    -1      -1     0
  0    0     95.0       5.0      450      0
1.14 42.0  6.0  0  58.0  13.0  0

  6  2    -1      -1     0
  0    0    85.0       4.0      600      0
0.46  54.0  6.0  0  46.0  14.0  0

  6  3    -1      -1     0
  0    0     60.0       3.9      675      0
1.10  45.0  6.0  0  55.0  17.0  0

ADD   6 TO   5

  5  4    -1      -1     0
  0    0    318.0       1.7      900      0
0.2  100.0  6.0  0  0.0  19.0  0

ADD   5 TO   1
```

```
START OF BRANCH 7
  7  1    -1       -1      0
  0    0     35.0       2.0       450      0
2.25  30.0  6.0  0  70.0  22.0  0

  7  2    -1       -1      0
  0    0     115.0       2.0       525      0
1.22  25.0  6.0  0  75.0  12.0  0

ADD   7 TO   1

  1  6    -1       -1      0
  0    0     83.4        0.9       1800      0
2.99  32.0  6.0  0  68.0  18.0  0

  1  7    -1       -1      0
  0    0     27.5        0.9       1800      0
4.74  36.0  6.0  0  64.0  22.0  0

  1  8    -1       -1      0
  0    0     95.2        1.1       1650      0
3.75  18.0  6.0  0  82.0  23.0  0

  1  9    -1       -1      0
  0    0     108.0       1.0       1650      0
3.13  35.0  6.0  0  65.0  16.0  0

  1 10    -1       -1      0
  0    0     37.9        1.0       1650      0
3.00  39.0  6.0  0  61.0  17.0  0

START OF BRANCH 8

  8  1    -1       -1      0
  0    0     86.0        3.7       450      0
2.03  44.0  6.0  0  56.0  14.0  0

  8  2    -1       -1      0
  0    0    60.0        1.4       600      0
1.24  31.0  6.0  0  69.0  15.0  0

  8  3    -1       -1      0
  0    0    106.0        1.7       600      0
0.63 22.0  6.0  0  78.0  15.0  0

START OF BRANCH 9

  9  1    -1       -1      0
  0    0    28.0        3.2       450      0
0.42  31.0  6.0  0  69.0  14.0  0

START OF BRANCH 10

 10  1    -1       -1      0
```

```
  0    0    86.0       1.3      450      0
1.02  19.0  6.0  0  81.0  11.0  0

 10  2     -1       -1      0
  0    0   76.0       11.0       450      0
2.24  10.0  6.0  0  90.0  13.5  0

ADD  10 TO  9

  9  2     -1       -1      0
  0    0   90.0       6.0       675      0
2.33  27.0  6.0  0  73.0  14.0  0

  9  3     -1       -1      0
  0    0   50.0       3.3       675      0
0.69  48.0  6.0  0  52.0  13.0  0

  9  4     -1       -1      0
  0    0   31.0       3.5       675      0
0.78  44.0  6.0  0  56.0  15.0  0

  9  5     -1       -1      0
  0    0   50.0       2.1       750      0
0.1  100.0  6.0  0  0.0  6.0  0

ADD   9 TO  8

  8  4     -1       -1      0
  0    0   40.0       2.7       825      0
0.74  42.0  6.0  0 58.0  17.0  0

  8  5     -1       -1      0
  0    0   45.0       2.5       900      0
1.25  38.0  6.0  0 62.0  18.0  0

ADD   8 TO  1

  1 11     -1       -1      0
  0    0   100.0       1.5      1950      0
0.2  100.0  6.0  0  0.0  6.0  0

  1 12     -1       -1      0
  0    0   1.0       1.5      1950      0
0.1  100.0  6.0  0  0.0  6.0  0

END
```

APPENDIX C

REPORT ON RORB ANALYSIS AND FLOOD FREQUENCY CURVE REVIEW

HWR 92/665

WILLING and PARTNERS

ACT URBAN CATCHMENT FLOOD STUDY. A RORB MODEL OF THE YARRALUMLA CK. CATCHMENT AREA.

MAY 1992

Prepared by Hydrology and Water Resources Branch

ACT Electricity and Water

ACT URBAN CATCHMENT FLOOD STUDY.
RORB MODEL OF YARRALUMLA CK. CATCHMENT AREA.

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APPENDICES

Appendix A. ACT Department of Urban Services
document RE: Jan. 1971 Flood Event.

LIST OF FIGURES.

LIST OF TABLES

ABBREVIATIONS

ACTEW	ACT Electricity and Water
AEP	Annual Exceedance Probability
ARI	Average Recurrence Interval
Cumecs	cubic metres per second
DUS	ACT Department of Urban Services
FFC	Flood Frequency Curve
G.S.	Gauging Station
IEAust.	Institution of Engineers, Australia
LP3	Log Pearson Type 3 Probability Distribution

1. INTRODUCTION.

The following report details the development and performance of a RORB rainfall-runoff computer model of the Yarralumla Ck. catchment area. The report forms part of a much larger study on rainfall-runoff computer models suitable for ACT urban catchments.

The aim of this report has been to determine RORB model parameters and rainfall losses applicable to urbanised catchment areas within the ACT. To this end two catchment areas, the Yarralumla Ck. catchment area above /Curtin gauging station and the Yarralumla Ck. catchment area above the Mawson gauging station have been studied. Throughout this report the two catchments are referred to as the Curtin and Mawson catchments respectively.

This report has been prepared on behalf of Willing and Partners as part of the larger study on ACT urban catchments. Willing and Partners have undertaken their studies on behalf of the Department of Urban Services.

2. SUMMARY and RECOMMENDATIONS.

Attempts have been made to determine suitable values for the RORB model parameters and rainfall loss parameters applicable to the Curtin and Mawson catchment areas. To achieve this the RORB models were calibrated against FFCs independently determined for the gauging stations at the head of each catchment.

A major problem within this report has been the determination of the independent FFCs to which the RORB models could be calibrated and tested. To this end composite FFCs for both the Curtin and Mawson sites were determined. The composite FFCs are based partly on the results of LP3 analysis and partly on drawing the FFCs through a point representing a major historical flood event.

Based on general experience with RORB models of other catchment areas, on the results from the Curtin and Mawson catchments, and on the recommendations of the authors of the RORB model (Laurenson and Mein, 1983), the major findings of this report are :-

- that a un-calibrated RORB rainfall-runoff model will not necessarily result in accurate estimates of flood flows for ACT urban areas

- if adopted for the purposes of determining design flows for ACT Urban areas any RORB rainfall-runoff models should :-

    preferably be calibrated against major historically recorded storm flood events

    or calibrated against independently determined flood frequency curves. For example calibrated against the 2, 5, and 10 year flood flows as given by an LP3 partial series flood frequency curve

    RORB models calibrated against the 2, 5, and 10 year ARI LP3 flows may be expected to provide flow estimates with an approximate accuracy of plus or minus 15% or better up to the AEP 1 in 100 flood event. Such accuracies are of course heavily dependent on the accuracies of the LP3 flows as influenced by the length and variability of the recorded streamflows

- in the absence of recorded storm/flood and streamflow data, the adoption of the RORB model parameters of

  - m = 0.8

  - and Kc as determined by the default RORB empirical equation,

together with the adoption of a Proportional rainfall loss model with the parameters values of :-

  - Initial Loss = 10 mm.

and - Runoff Coefficient = 45%

can be expected to provide flow estimates for storm events having an AEP ranging from 1 in 2 to 1 in 100, with an approximate accuracy of plus or minus 20% or better.

Given the difficulty in determining accurate FFCs for both the Mawson and Curtin Gauging Stations, even with 20 years of streamflow record, a most important recommendation of this report is that streamflow records at these sites must continue to be kept for at least another 5 to 10 years.

3. YARRALUMLA CK. CATCHMENT AREA.

The Yarralumla Ck. catchment area to the Curtin gauging station encompasses the Woden Valley area of the ACT and totals 26.8 sq. km. The catchment is extensively urbanised with the impervious catchment surface area estimated to exceed 20% of the total catchment surface.

The two main waterways with-in the catchment are the Yarralumla Ck. and Long Gully Ck. which is a tributary to Yarralumla Ck.

Three Gauging Stations with-in the catchment, and their associated catchment size and period of streamflow record are :-

- G.S. 410745 Yarralumla Ck. at Curtin, 26.8 sq. km. Jan. 1970 to present
- G.S. 410753 Yarralumla Ck. at Mawson, 4.3 sq. km. Sept. 1971 to present

and

- G.S. 410746 Long Gully Ck. at Phillip, 4.8 sq. km. Dec. 1970 to present

A fourth Gauging Station used in the larger ACT Urban Catchment study but not used in this RORB study is G.S. 410763, Storm water Drain at Giralang, with a streamflow record from Aug. 1973 to present.

The Long Gully Ck. catchment area above Curtin and the location of the above three gauging stations is shown on Figure 1.

4. JANUARY 1971 STORM/FLOOD EVENT.

4.1 Summary

The Jan. 1971 storm/flood event was a severe flood event causing much damage and loss of life in the Woden area. The storm was highly localised with the eye of the storm centred above the Mawson catchment close to the Mawson gauging station.

The storm rainfalls, peak flood flows, and the estimated storm and flood AEPs for both the Curtin and Mawson catchments are summarised below.

JANUARY 1971 STORM/FLOOD EVENT.

G.S. 410745 Yarralumla Ck. at Curtin.
(catchment area = 26.8 sq. km.)

Average Depth of Rainfall = 60 mm.

Estimated Equivalent Point Depth of Rainfall = 65 mm

Estimated Storm AEP = 1 in 285.

Peak Flow at Curtin = 240 cumecs (175 to 350 cumecs)

Estimated Flood AEP = 1 in 350.

G.S. 410753 Yarralumla Ck. at Mawson.
(catchment area = 4.3 sq. km.)

Average Depth of Rainfall = 85 mm.

Estimated Equivalent Point Depth of Rainfall = 85 mm

Estimated Storm AEP = 1 in 900.

Peak Flow at Mawson = 195 cumecs
(based on debris slope)
OR = 127 cumecs
(based on channel bed slope)

Estimated Flood AEP = 1 in 1000.

4.2 Peak Flows.

4.2.1 Peak Flows at Curtin G.S.

The Curtin gauging station was commissioned in January 1970. The peak recorded flow for the January 1971 flood was 217 cumecs, however this figure excludes overland flow which bypassed the gauging station. Based on flood debris slope and estimates of Manning's roughness coefficients, the ACT Department of Urban Services (DUS), have estimated the January 1971 peak flow at Curtin as 240 cumecs with a range of 175 cumecs to 350 cumecs. Refer Appendix A for a copy of a letter from DUS documenting these figures.

4.2.2 Peak Flows at Mawson G.S.

The Mawson G.S. was not commissioned until September 1971 and hence was not in place during the January 1971 flood event. Nevertheless based on estimates of Manning's roughness coefficients, the DUS have estimated the January 1971 peak flow at Mawson as either 195 cumecs, based on flood debris slope, or 127 cumecs, based on channel bed slope, (refer Appendix A).

The surveyed January 1971 flood debris slope at Mawson may have been influenced by lateral inflow, or by the possible presence of an hydraulic jump a short distance downstream of the Mawson gauging station caused by a change from lined to unlined section with a jump drop in channel invert.

4.3 Return Period of Storm Event.

4.3.1 Storm over Curtin Catchment.

Using the storm isohyetal pattern derived by the Bureau of Meteorology (Bureau of Meteorology, 1972) for the rainfall over the Curtin and Mawson catchments for the Australia Day 1971 storm, 7.30 pm to 8.30 pm, the average depth of rainfall over the entire Curtin catchment has been calculated as 60 mm. Allowing a depth-area reduction factor of 0.95 for the 28 sq. km. Curtin catchment, (as per Figure 2.6 in IEAust. ,1987), results in an equivalent design point rainfall of 65 mm.

From the IEAust., 1987, design storm criteria, a one hour duration storm resulting in a point rainfall of 65 mm has an AEP of approx. 1 in 285. Refer Figure 8 for an Intensity - Frequency diagram for one hour duration storms in the ACT. This diagram has been determined from IEAust, (1987).

4.3.2. Storm over Mawson Catchment.

From the Jan 1971 storm isohyetal pattern (Bureau of Meteorology, 1972) the aeral weighted average depth of rainfall for the storm event above the Mawson catchment was 85 mm. As the Mawson catchment is less than 5 sq. km. the depth-area reduction factor is effectively = 1.0 and the equivalent design point rainfall is 85 mm.

From Figure 8 a design point rainfall of 85 mm at Canberra resulting from a one hour duration storm has an AEP of approx. 1 in 900.

4.4 Return Period of Flood Event.

4.4.1. G.S. 410745 Yarralumla Ck. at Curtin.

On the basis that the storm over Curtin has been assigned an AEP of 1 in 250 one could assign a similar AEP to the peak flow. However heavy storms had occurred the previous day (refer Appendix A), and the antecedent catchment conditions would have been anything but "median". Hence a storm of AEP = 1 in 250 occurring over a saturated catchment would have resulted in a rarer flood than that indicated by an AEP of 1 in 250.

For the purposes of this report the January 1971 flood event at Curtin has been assumed to have an AEP of 1 in 350. This figure of AEP = 1 in 350 is a subjective judgement of the influence of the wet antecedent catchment conditions on the peak flood flows, based on the knowledge that the storm AEP was 1 in 250.

4.4.2. G.S. 410753 Yarralumla Ck. at Mawson.

Given the wet antecedent catchment conditions, it has been assumed for the purposes of this report that the AEP 1 in 900 storm occurring over the Mawson catchment on Australia day 1971 resulted in a flood event of AEP = 1 in 1000. The figure of AEP = 1 in 1000 is somewhat subjective but the proposal that a storm having an AEP of 1 in 900 occurring over a wet catchment results in flood event rarer than that indicated by an AEP of 1 in 900 is valid.

5. FLOOD FREQUENCY CURVES.

5.1 General

Flood Frequency Curves (FFCs) for the Yarralumla Ck. at Curtin and Yarralumla Ck. at Mawson gauging stations are shown at Figures 2 and 3 respectively. The FFCs have been determined so as to allow calibration of the RORB model. The Long Gully Ck. at Phillip gauging station has not been used for calibration purposes and as such a FFC for this station has not been derived.

The FFCs are based on a composite curve method whereby the flows up to the 10 year Average Recurrence Interval (ARI) are determined using a partial series Log-Pearson Type 3 (LP3) probability distribution. For ARIs greater than 10 years, ie AEPs of less than 1 in 10, the flows have been estimated by drawing the FFC through a point representing the January 1971 flood event.

Given that the January 1971 storm/flood event was a rare event with subsequent high flows it was considered more appropriate to draw the FFC through a point representing the January 1971 flood event, than simply extrapolating the LP3 FFCs.

The adopted points used to represent the January 1971 flood events were :-

- G.S. 410745 Yarralumla Ck. at Curtin

  Flow = 240 cumecs
  Flood AEP = 1 in 350.
  (refer Section 4.)

- G.S. 410753 Yarralumla Ck. at Mawson

  Flow = 195 cumecs
  Flood AEP = 1 in 1000.
  (refer Section 4.)

As the FFCs for both Curtin and Mawson have been determined, at least for the AEP 1 in 20 and rarer flood events, by drawing the FFCs through a point representing the January 1971 flood event, it must be stressed that these FFCs apply to the catchment conditions as existed primarily in 1971.

5.2 Curtin G.S. Flood Frequency Curve.

5.2.1 Curtin LP3 FFC

The period of streamflow record at Curtin is 21 years. Hence the partial series LP3 FFC is based on a statistical analysis of the 21 highest independent flows recorded at Curtin.

Given that the January 1971 flood event at Curtin was a rare event with an estimated AEP of 1 in 350, it was decided to exclude the January 1971 flood event from the LP3 partial series analysis. It was considered that inclusion of an AEP = 1 in 350 flood event into just 21 years of streamflow record would result in over estimation of all the flows determined by the LP3 analysis.

Exclusion of the January 1971 flood event from the LP3 partial series analysis may result in slightly underestimated flows, but it is considered that more accurate flows can be gained by excluding the January 1971 flood event than by including the event.

The partial series LP3 FFC is shown in full at Figure 4. For interest the annual series LP3 FFC (with the January 1971 flood event excluded) is also shown.

5.2.2 Curtin FFC based on January 1971 Flood Event.

The composite FFC for Yarralumla Ck. at Curtin is shown at Figure 2.

The design flows as given at Figure 2 are summarised as follows :-

2 year ARI	64 cumecs	from LP3 partial series, (excludes Jan. 1971 flood flow).
5 year ARI	83 cumecs	
10 year ARI	97 cumecs	
1 in 20 AEP	119 cumecs	from FFC curve drawn through point representing Jan. 1971 flood event.
1 in 50 AEP	150 cumecs	
1 in 100 AEP	180 cumecs	

The estimated flow at Curtin for the January 1971 flood event was 240 cumecs with a range of 175 cumecs to 350 cumecs, (refer Section 4.2).

The FFC shown at Figure 5 has been derived for the purpose of determining extreme estimates of the 1 in 100 AEP flow at Curtin based on the extreme estimates of the January 1971 flood flow of 175 cumecs and 350 cumecs. The FFC shown at Figure 5 is not to be confused with the adopted composite FFC for Curtin shown at Figure 2.

From Figure 5 the extreme estimates of the 1 in 100 AEP flood flows at Curtin G.S. are 145 cumecs to 220 cumecs.

5.3 Mawson G.S. Flood Frequency Curve.

5.3.1 Mawson LP3 FFC

The partial series LP3 FFC at Mawson is shown at Figure 6. The estimated January 1971 flood flow at Mawson is between 127 to 195 cumecs, with an estimated flood AEP of 1 in 1000. (refer Section 4). Similarly to the Curtin LP3 partial series analysis the estimated January 1971 flood flow has been excluded form the LP3 analysis in order to ensure a sample of flood flows representative of the 21 years of streamflow record.

For interest sake the LP3 annual series FFC for Mawson is also shown at Figure 6. As with the partial series analysis the estimated January 1971 flood flow at Mawson was excluded from the annual series LP3 analysis.

5.3.2 Mawson FFC based on January 1971 Flood Event.

The composite FFC for Mawson is shown on Figure 3.

The design flows as given at Figure 3 are summarised as follows :-

- 2 year ARI	22 cumecs	} from LP3 partial series, (excludes Jan. 1971 flood flow).
- 5 year ARI	29 cumecs	}
- 10 year ARI	34 cumecs	}
- 1 in 20 AEP	43 cumecs	} from FFC curve drawn through point representing Jan. 1971 flood event.
- 1 in 50 AEP	63 cumecs	}
- 1 in 100 AEP	80 cumecs	}

The January 1971 flood event at Mawson was assigned an AEP of 1 in 1000. The estimated flows at Mawson for the flood event ranged from 127 cumecs, based on channel bed slope, to 195 cumecs, based on flood debris slope. Figure 7 shows the FFCs obtained by drawing a curve through two points representing these two flood estimates.

From Figure 7 it can be seen that the estimated AEP 1 in 100 flows obtained by drawing a curve through the estimated January 1971 flood flows at Mawson of 127 cumecs and 195 cumecs are 70 cumecs and 80 cumecs respectively. Hence even though there is a large variation in the estimated January 1971 flood flows the estimated AEP 1 in 100 flows obtained by using these flows vary from only 70 to 80 cumecs.

6. RORB Program

6.1 General

The rainfall runoff program used in this report to model the Yarralumla Ck. catchment area is the RORB Runoff Routing program, Version 3, as developed by Laurenson and Mein, 1983.

6.2 Modelling of Channel Storage

The RORB program assumes the storage discharge relationship of channel reaches is given by :-

$$S = 3600\ K\ Q^{m}$$

where S = storage in cubic metres
Q = outflow discharge in cumecs
m = a dimensionless exponent, viz. the Storage-Discharge exponent, and is a measure of the non-linear behaviour of the catchment

and K = a dimensional empirical coefficient

The empirical coefficient K is given by :-

$$K = kc\ kr$$

where kc = an empirical coefficient applicable to the entire catchment and stream network

and kr = a dimensionless ratio called the relative delay time, applicable to an individual channel reach storage

If kri is the relative delay time applicable to channel storage number i, then kri is calculated as :-

$$kri = F\ \frac{Li}{Dav}$$

where Li = the length of reach represented by storage number i

Dav = the average flow distance in the channel network of subarea inflows in kilometres

and F = a factor dependent on the type of channel reach such as lined, natural, excavated but unlined, or drowned by backwater.

6.3 Rainfall Loss Models

The RORB program assumes that 90% of rainfall falling onto impervious areas will run off such areas. For rainfall losses on pervious areas the RORB program can model these losses as

- a constant proportional rate of loss

in this case an initial loss (mm) and a dimensionless coefficient of runoff is assigned to the catchment. The coefficient of runoff is a volumetric runoff coefficient equal to the proportion of stormwater runoff to rainfall, for the catchment.

or - on an initial loss (mm) - constant continuing loss (mm/hour) basis

When using the constant proportional rate of rainfall loss model for subareas which are partly pervious and partly impervious the RORB program calculates an area weighted runoff coefficient applicable to the given subarea. For example if a sub-area is half pervious and half impervious and the pervious portion of the subarea is assigned a runoff coefficient of 40%, then the RORB model will calculate an area weighted volumetric runoff coefficient of 65% (0.5 times 90% plus 0.5 times 40%) applicable to the subarea as a whole.

Laurenson and Mein (1983), the authors of the RORB model, recommend that the constant proportional rate of rainfall loss model should be used for urbanised catchments together with an Initial loss of about 10 mm for design floods less than the Probable Maximum Flood (PMF).

7. RORB Model of Yarralumla Ck. Catchment.

7.1 Yarralumla Ck. Catchment above Curtin G.S.

A RORB model of the Yarralumla Ck. catchment above the Curtin G.S. is shown on Figure 9. The RORB model subdivision of the catchment is based on a RAFTS model of the catchment developed by Knee (Knee, 1990). The model consists of 56 sub-areas and 57 channel storages. Laurenson and Mein, 1983, recommend that RORB models need not consist of more than twenty subareas as accuracy is not improved. However in the larger ACT Urban Catchments study where comparison of the RORB model to other rainfall-runoff models is being undertaken it was considered that the various models should, as much as possible, be given similar input data such that any difference in model performance could be attributed to differences in the models themselves and not to variations in catchment sub-division and other input data.

Tables 7.1 and 7.2 attached summarise details of the catchment sub-areas and channel storages respectively.

Because the recommended Curtin FFC is based largely on the January 1971 flood event and in order to allow calibration of the Curtin RORB model against the recommended Curtin FFC, the Curtin RORB model has been developed to reflect the 1971 Curtin catchment conditions. For this purpose the existing Issacs area of the Curtin catchment has been modelled as a rural area with no associated impervious areas. The area of Issacs is a relatively new suburb which was not present in 1971.

7.2 Yarralumla Ck. Catchment above Mawson G.S.

A RORB model of the Mawson catchment area which is a sub-catchment of the Curtin catchment is shown on Figure 9. The Mawson model consists of 7 sub-areas and 7 channel storages and is simply a portion of the larger Curtin catchment. The Mawson catchment is considered as a separate entity so as to allow calibration of the RORB model for this catchment at the point of the Mawson gauging station. A calibrated RORB model of the Curtin catchment is suitable only for predicting flood flows at the Curtin gauging station itself and cannot be used for prediction of flows at Mawson.

TABLE 7.1 Summary of MAWSON and CURTIN Catchment Sub-Area Details.

	RORB MODEL	SUB-CATCHMENT DETAILS.		
SUB-~~CATCHMENT~~ AREA.	PERVIOUS CATCHMENT AREA (ha.)	IMPERVIOUS CATCHMENT AREA (ha.)	TOTAL CATCHMENT AREA (ha.)	FRACTION IMPERVIOUS
D2	16.5	7.1	23.6	0.3
C2	13.2	5.6	18.8	0.3
B2	17.9	7.7	25.6	0.3
A2	34.4	14.8	49.2	0.3
Z1	93.6	0	93.6	0
Y1	20	0	20	0
X1	30.9	5.5	36.4	0.15
T1	13.4	5.8	19.2	0.3
W1	5	0.6	5.6	0.1
V1	17.1	7.3	24.4	0.3
U1	33.6	14.4	48	0.3
M1	135	45	180	0.25
R1	73.6	31.6	105.2	0.3
S1	24.4	10.4	34.8	0.3
N1	65.7	2.7	68.4	0.04
Q1	3.1	1.3	4.4	0.3
O1	107.6	46.1	153.7	0.3
P1	11.6	11.6	23.2	0.5
T	15.8	15.8	31.6	0.5
K1	36.4	15.6	52	0.3
J1	51.7	9.1	60.8	0.15
L1	282.8	0	282.8	0
R	3.5	8.1	11.6	0.7
Q	37.5	16.1	53.6	0.3
O	4	4	8	0.5
N	20.2	8.6	28.8	0.3
C1	14.3	6.1	20.4	0.3
I1	19.6	8.4	28	0.3
G1	34.4	14.8	49.2	0.3
E1	23.8	10.2	34	0.3
D1	44	0	44	0
K	40.6	17.4	58	0.3
M	11.5	4.9	16.4	0.3
L	18.2	7.8	26	0.3
J	33	14.2	47.2	0.3
I	56.1	9.9	66	0.15
E	14.8	6.4	21.2	0.3
H	30	12.8	42.8	0.3
F	43.1	18.5	61.6	0.3
G	19.9	8.5	28.4	0.3
F1	32.3	5.7	38	0.15
B1	18.7	3.3	22	0.15
Z	33.6	0	33.6	0
C	71.7	30.7	102.4	0.3
B	48.4	20.8	69.2	0.3
Y	41.4	17.8	59.2	0.3
X	7.2	0.8	8	0.1
A	84	36	120	0.3
U	58.4	0	58.4	0

subareas A, B, C, D, E, F, and G form the MAWSON catchment.

continued on next page.

TABLE 7.1 Summary of MAWSON and CURTIN Catchment Sub-Area Details. (continued).

RORB MODEL SUB-CATCHMENT DETAILS.

SUB-CATCHMENT	PERVIOUS CATCHMENT AREA (ha.)	IMPERVIOUS CATCHMENT AREA (ha.)	TOTAL CATCHMENT AREA (ha.)	FRACTION IMPERVIOUS
P	12.2	3	15.2	0.2
A1	9.2	4	13.2	0.3
S	9	3.8	12.8	0.3
W	26.3	11.3	37.6	0.3
V	25.9	2.9	28.8	0.1
D	28.1	3.1	31.2	0.1
H1	14.8	6.4	21.2	0.3
TOTALS	2093	584.3	2677.3	

TABLE 7.2 Summary of MAWSON and CURTIN Catchment Channel Storage Details.

RAFTS LINK NBR.	RORB CHANNEL REACH NBR.	LENGTH (Km)	AVERAGE SLOPE (%)	AVERAGE SLOPE (%)	FROM CENTROID OF AREA	TO CENTROID OF AREA	R.L. TOP OF LENGTH (m AHD)	R.L. BOTTOM OF LENGTH (m AHD)
1.13	57	0.2	0.5	0.5	D2	Catchment Outlet	566	565
1.12	56	0.35	1	1	C2	D2	569.5	566
1.11	50	0.3	1	1	W1	C2	572.5	569.5
1.1	46	0.35	0.7	0.714285	S1	W1	575	572.5
1.09	44	0.4	0.6	0.625	Q1	S1	577.5	575
1.08	41	0.65	0.9	0.923076	N1	Q1	583.5	577.5
1.06	38	0.9	1.2	1.222222	K1	N1	594.5	583.5
1.05	36	0.7	2.2	2.214285	I1	K1	610	594.5
1.04	34	0.8	2.1	2.125	G1	I1	627	610
1.03	33	0.55	1.7	1.727272	F1	G1	636.5	627
1.02	29	0.5	1.9	1.9	B1	F1	646	636.5
1.01	26	0.6	3.3	3.333333	Y	B1	666	646
1.00	24	0.5	4.8	4.8	W	Y	690	666
1.001	22	0.5	12	12	U	W	750	690
24.02	55	0.6	1.3	1.25	B2	C2	582.5	575
24.01	54	0.75	4.3	4.333333	A2	B2	615	582.5
24	52	0.9	1.2	1.222222	Y1	A2	626	615
24.001	51	0.55	11.6	11.63636	X1	Y1	690	626
25.00	53	1.1	2.7	2.727272	Z1	A2	645	615
10.08	21	0.5	0.8	0.8	T	Q1	581.5	577.5
10.07	20	0.4	0.6	0.625	S	T	584	581.5
10.06	17	0.25	0.8	0.8	P	S	586	584
10.05	15	0.3	0.8	0.833333	N	P	588.5	586
10.04	14	0.4	1	1	M	N	592.5	588.5
10.03	10	0.5	1.4	1.4	I	M	599.5	592.5
10.02	9	0.45	1.2	1.222222	E	I	605	599.5
10.01	4	0.5	3.5	3.5	D	E	622.5	605
10.00	2	0.9	2.8	2.777777	B	D	647.5	622.5
10.001	1	0.9	1.4	1.388888	A	B	660	647.5
23.00	49	0.45	2.2	2.222222	V1	W1	582.5	572.5
23.01	48	1.1	2.5	2.5	U1	V1	610	582.5
22.00	47	0.45	4.4	4.444444	T1	W1	592.5	572.5
21.00	45	1.2	1.9	1.875	R1	S1	597.5	575
20.00	43	0.5	1.5	1.5	P1	Q1	585	577.5
20.001	42	1.1	1.8	1.818181	O1	P1	605	585
9.00	40	1.25	2.3	2.32	M1	N1	612.5	583.5
9.001	39	0.9	2.5	2.5	L1	M1	635	612.5
8.00	37	0.9	5.1	5.055555	J1	K1	640	594.5
7.00	31	0.45	16.7	16.66666	D1	E1	740	665
7.01	32	0.45	6.3	6.333333	E1	F1	665	636.5

continued on next page

TABLE 7.2 Summary of MAWSON and CURTIN Catchment Channel Storage Details. (continued from previous page).

RAFTS LINK NBR.	RORB CHANNEL REACH NBR.	LENGTH (Km)	AVERAGE SLOPE (%)	AVERAGE SLOPE (%)	FROM CENTROID OF AREA	TO CENTROID OF AREA	R.L. TOP OF LENGTH (m AHD)	R.L. BOTTOM OF LENGTH (m AHD)
6.00	30	0.5	4.7	4.7	C1	F1	660	636.5
5.00 5.01	27 28	0.4 0.45	12.5 12	12.5 12	Z A1	A1 B1	750 700	700 646
4.00	25	0.7	7.7	7.714285	X	Y	720	666
3.00	23	0.45	21.1	21.11111	V	W	785	690
2.00	3	0.8	4.1	4.0625	C	D	655	622.5
13.00	8	0.8	4.4	4.375	H	E	640	605
11.00	6	0.9	3.1	3.055555	F	E	632.5	605
12.00	7	0.45	6.1	6.111111	G	E	632.5	605
14.00	11	1.2	3.1	3.125	J	M	630	592.5
15.00	12	1.1	3.4	3.409090	K	M	630	592.5
16.00	13	0.4	4.4	4.375	L	M	610	592.5
17.00	16	0.5	1.8	1.8	O	P	595	586
18.00	18	0.9	2.9	2.888888	Q	S	610	584
19.00	19	0.3	2	2	R	S	590	584
27.00	35	0.6	3.3	3.333333	H1	I1	630	610
	5	0.01 km	1		E1	to upstream end of Channel Storage nbr. 9		

LINK NOS. 10.01 D to E
10.00 B to D
10.001 A to B
2.00 C to D
11.00 F to E
and 12.00 G to E,
form the LINKS within the MAWSON catchment.

8. Design Rainfall Data.

Tables 8.1 and 8.2 summarise the IEAust., 1987 design rainfall data for the Curtin and Mawson catchments respectively for storm durations ranging from 30 minutes to two hours and AEP storm events ranging from AEP = 1 in 2 to 1 in 100.

The design rainfalls as given in Tables 8.1 and 8.2 together with design temporal patterns as given by IEAust., (1987) Vol. 2 Table 3.2 were used as input data when calibrating both the Curtin and Mawson RORB models

The design rainfalls for the Curtin catchment are slightly lower than those used for the Mawson catchment because of areal reduction factors varying from 5% to 1% applied to the design point rainfalls on account of the larger Curtin catchment

TABLE 8.1

DESIGN RAINFALLS FOR YARALUMLA CK. CATCHMENT ABOVE CURTIN GAUGING STATION.

RAINFALL ARI (years)	STORM DURATION 30 minutes		45 minutes		1 hour		90 minutes		2 hours		3 hours	
	Rainfall Intensity (mm/hr)	Total Rainfall (mm)	Rainfall Intensity (mm/hr)	Total Rainfall (mm)	Rainfall Intensity (mm/hr)	Total Rainfall (mm)	Rainfall Intensity (mm/hr)	Total Rainfall (mm)	Rainfall Intensity (mm/hr)	Total Rainfall (mm)	Rainfall Intensity (mm/hr)	Total Rainfall (mm)
2	28.8	14.4	26.0	19.5	21.5	21.5	17.0	25.5	14	28	10.5	31.5
5	39.6	19.8	34.0	25.5	28	28	22.0	33	18	36	13.5	40.5
10	45.0	22.5	38.0	28.5	32	32	25.0	37.5	20	40	15.5	46.5
20	52.2	26.1	45.1	33.8	38	38	29.0	43.5	23	46	18	54
50	63.0	31.5	54.0	40.5	45	45	35.0	52.5	28	56	22	66
100	72	36	62.0	46.5	51	51	40	60	32	64	24	72
Areal Reduction Factor applied to AR&R Design Point Rainfalls	10%		7%		5%		4%		3%		Zero	

20

TABLE 8.2

DESIGN RAINFALLS FOR YARALUMLA CK. CATCHMENT ABOVE MAWSON GAUGING STATION.

Rainfall Details taken from AR&R Map 7c. Skewness for Canberra = +0.24

RAINFALL ARI (years)	STORM DURATION 30 minutes		45 minutes		1 hour		90 minutes		2 hours		3 hours	
	Rainfall Intensity (mm/hr)	Total Rainfall (mm)	Rainfall Intensity (mm/hr)	Total Rainfall (mm)	Rainfall Intensity (mm/hr)	Total Rainfall (mm)	Rainfall Intensity (mm/hr)	Total Rainfall (mm)	Rainfall Intensity (mm/hr)	Total Rainfall (mm)	Rainfall Intensity (mm/hr)	Total Rainfall (mm)
2	32	16		19.5	21.5	21.5	17.0	25.5	14	28	10.5	31.5
5	43	22		25.5	28	28	22.0	33	18	36	13.5	40.5
10	49	25		28.5	32	32	25.0	37.5	20	40	15.5	46.5
20	58	29		33.8	38	38	29.0	43.5	23	46	18	54
50	70	35		40.5	45	45	35.0	52.5	28	56	22	66
100	80	40		46.5	51	51	40	60	32	64	24	72

9. Calibration of RORB Model.

9.1 Calibration against FFCs

Both the Curtin and Mawson RORB models have been calibrated against the FFCs recommended for the respective gauging stations and shown on Figs. 2 and 3. Calibration of the RORB models against FFCs which were determined without reference to any rainfall-runoff models, was carried out in an attempt to determine RORB model parameters and values of constant proportional rainfall losses, suitable for ACT urbanised catchments.

9.2 Method of Calibration.

Calibration of the RORB model for both the Curtin and Mawson catchments was achieved by :-

- inputting the design rainfall data as detailed in Section 8

- adopting default RORB model parameters of m = 0.8

  and Kc determined by the default RORB model empirical equation

  $$Kc = 2.2\ A^{0.5}\ (\ Qp/2)^{(0.8\ -\ m)}$$

  effectively

  $$Kc = 2.2\ A^{0.5}$$

- adopting the proportional loss rainfall model as recommended by Laurenson and Mein, (1983) for urban catchments. As part of the proportional rainfall loss model an Initial rainfall loss of 10 mm was adopted. as per the recommendations of Laurenson and Mein, (1983). An initial loss of 10 mm amounts to a significant proportion of the design rainfall for the short duration, frequent, high probability, design storm rainfalls. However an Initial loss of 10 mm is considered a reasonable figure and should be independent of design storm frequency.

- for the various AEP flood events the volumetric runoff coefficient required to match the RORB predicted flows at Curtin and Mawson with the flows given by the recommended FFCs were then determined on a trial and error basis.

9.3 Results of Calibration.

9.3.1 General

The calibrated values of volumetric runoff coefficients for the various storm frequencies and durations analysed are summarised in Tables 9.1 and 9.2. The runoff coefficients have been calibrated in units of 5% as it was considered that finer calibration was not warranted.

9.3.2 Critical Storm Duration.

From Table 9.1 it can be seen that the critical storm duration for the Mawson catchment area is one hour. This is based on the observation that the lowest calibrated volumetric runoff coefficients occur for design storm durations of one hour. This means that if a given volumetric runoff coefficient were applied to all storm durations the highest flow would occur for the one hour duration storms.

From Table 9.2 which applies to the Curtin catchment, it can be seen that the critical storm duration ranges from two hours for the AEP 1 in 2 and 1 in 5 storm events to approximately 45 minutes for the AEP 1 in 100 storm event, with a critical storm duration of one hour for all the other storm event frequencies.

9.3.3. Volumetric Runoff Coefficients.

From Table 9.1 the calibrated volumetric runoff coefficients for the critical storm duration of one hour for the Mawson catchment ranges from 50% to 70%, with a median value of 55%. From Table 9.2 the calibrated runoff coefficients for the Curtin catchment range from 35% to 50% with a median value of 40%.

From these results it is evident that while runoff coefficients within a given catchment do not vary enormously with design storm exceedance probability, it is apparent that the runoff coefficients for the different Curtin and Mawson catchments, with median runoff coefficients of 40% and 55% respectively, do vary appreciably.

This result is disappointing in that it was hoped to determine universal values for the RORB model parameters and for the proportional rainfall loss model, applicable to all ACT urban areas and to design storm frequencies up to the AEP 1 in 100 storms.

The results highlight the importance of calibrating the RORB model against historical storm-flood events or pre-determined flood frequency curves in order to determine appropriate model parameters and runoff coefficients.

Table 9.1

Calibrated Volumetric Runoff Coefficients for the Yarralumla Ck. above MAWSON Gauging Station.

Catchment Area = 4.3 sq. km. approx.

Adopted Model parameters :- m = 0.8
Kc calculated as per RORB default equation
(for Mawson catchment Kc = 4.6)

Proportional rate of rainfall loss model adopted
As part of the proportional rainfall loss model an Initial Loss of 10 mm was adopted.

DESIGN STORM AEP	PEAK FLOW from FFC (cumecs)	STORM DURATION					CRITICAL STORM DURATION (hours)
		30 minutes	45 minutes	1 hour	90 minutes	2 hours	
1 in 2	22	95% (22)	80% (22)	65% (22)	85% (22)	65% (21) 70% (23)	1 hour
1 in 5	29	65% (29)	65% (28) 70% (30)	55% (29)	70% (28) 75% (30)	55% (28) 60% (30)	1 hour
1 in 10	34	60% (34)	65% (33) 70% (35)	50% (34)	70% (34)	55% (33) 60% (30)	1 hour
1 in 20	43	60% (43)	65% (42)	50% (42)	70% (42) 75% (44)	60% (43)	1 hour
1 in 50	63	75% (63)	85% (64)	70% (63)	95% (64)	80% (63)	1 hour
1 in 100	80	80% (79) 70% (82)	90% (80)	55% (80)	98% (79)	90% (81)	1 hour

NOTE The figures in brackets represent the peak flow predicted by the RORB model when using the runoff coeficient given.

Table 9.2

Calibrated Volumetric Runoff Coefficient for the Yarralumla Ck. Catchment above CURTIN Gauging Station.

Catchment Area = 28 sq. km. approx.

Adopted Model Parameters :-

m = 0.8
Kc calculated as per RORB default equation
(for Curtin catchment Kc = 11)

Proportional rainfall loss model adopted
As part of the proportional rainfall loss model an Initial loss = 10 mm adopted

DESIGN STORM AEP	PEAK FLOW from FFC (cumecs)	STORM DURATION 30 minutes	45 minutes	1 hour	90 minutes	2 hours	CRITICAL STORM DURATION (hours)	Runoff Coeff. for critical storm
1 in 2	64	100% (58)	656% (62)	55% (64)	50% (63)	50% (66)	2 hours	50%
1 in 5	83	70% (85)	50% (83)	45% (87)	45% (86)	40% (83)	1 hour	45%
1 in 10	97	60% (95)	50% (99)	40% (99)	40% (97)	40% (96)	1 hour	40%
1 in 20	119	60% (122)	45% (121)	35% (117)	40% (121)	40% (118)	1 hour	35%
1 in 50	150	55% (154)	40% (145)	35% (146)	40% (149)	40% (148)	1 hour	35%
1 in 100	180	50% (176)	40% (175)	40% (189)	40% (178)	40% (176)	1 hour	40%

NOTE The figures in brackets represent the peak flow predicted by the RORB model
when using the runoff coefficient given.

The difference in runoff coefficients between the Mawson and Curtin catchments of 50%-70% and 35%-50% respectively is difficult to explain.

One possibility is that the storages within the Mawson catchment have been overestimated or that the storages within the Curtin catchment have been underestimated. For the Mawson catchment the RORB program has modelled the storages in the underground piped stormwater system. The same is true for the RORB model of the Curtin catchment but in addition the RORB program also models the storages in the lined stormwater drainage channels in the Curtin catchment (such drainage channels are not present in the Mawson catchment).

Whether modelling stormwater pipes or lined stormwater channels the RORB program calculates the storage on the basis of the channel, or pipe, slope and length.

In view of the median runoff coefficients for the Curtin and Mawson catchments of 40% and 55% respectively and in an attempt to determine runoff coefficients applicable to both the Curtin and Mawson catchments, runoff coefficients of 45% were assigned to both catchments and the subsequent RORB predicted flood flows, for the critical durations determined earlier, have been compared to the recommended FFCs to determine their accuracy. The results of this analyses are shown in Tables 9.3 and 9.4.

Table 9.3

Mawson Catchment RORB predicted Peak Flows Using Runoff Coefficient = 45%

Storm AEP	Peak Flow as per FFC (cumecs)	Mawson Catchment	Comparison to FFC
1 in 2	22	17	- 21%
1 in 5	29	26	- 11%
1 in 10	34	31	- 8%
1 in 20	43	40	- 8%
1 in 50	63 (60)	47	- 25% (- 22%)
1 in 100	80 (75)	55	- 31% (-26%)

Table 9.4

Curtin Catchment RORB predicted Peak Flows Using Runoff Coefficient = 45%

Storm AEP	Peak Flow as per FFC (cumecs)	RORB predicted Peak Flow (cumecs)	Comparison to FFC
1 in 2	64	61	- 5%
1 in 5	83	90	+ 8%
1 in 10	97	108	+ 11%
1 in 20	119	140	+ 18%
1 in 50	150	174	+ 16%
1 in 100	180	190	+ 6%

The results from Tables 9.3 and 9.4 indicate that adopting Runoff Coefficients of 45% for both the Mawson and Curtin catchments results in predicted flows having an accuracy of between - 31% to + 18%. Putting aside the results for the AEP 1 in 50 and 1 in 100 storm events for the Mawson catchment the accuracy is increased to - 21% to + 18%, say plus or minus 20%.

The poorest results of -25% and - 31% for the AEP 1 in 50 and 1 in 100 storm events for the Mawson catchment may reflect the unsuitability of the RORB model for modeling small urban catchments, or may be due to an over estimation of the peak flows given in the Mawson FFC.

Section 4 explains that the AEP 1 in 50 and 1 in 100 flows at Mawson are based on the January 1971 flood event. The January 1971 flood event was judged to have an AEP of 1 in 1000 and the peak flow at Mawson was estimated at between 127 cumecs and 195 cumecs. In determining the FFC for Mawson the 195 cumec flow was adopted but if the average of the January 1971 extreme flood estimates of 160 cumecs was adopted then, following the procedure used in Section 4 to determine a FFC for Mawson, the AEP 1 in 50 and 1 in 100 flood estimates would become 60 cumecs and 75 cumecs respectively. These revised estimates do not vary greatly from the original estimates of 63 cumecs and 80 cumecs respectively. However if the revised estimates are adopted then the accuracy of the RORB model, using a runoff coefficient of 45%, in predicting the AEP 1 in 50 and 1 in 100 flows at Mawson, becomes -22% and -26% respectively making the overall efficiency of the RORB model for the design storms analysed -26% to +18%.

Such accuracy is not flattering but the conclusion can be made that in the absence of any data allowing calibration of a RORB model that the default RORB model parameters, together with the adoption of the proportional rainfall loss model with an initial loss of 10 mm and a runoff coefficient for pervious areas of 45%, should result in predicted flows being accurate to approximately plus or minus 25% or better.

The FFCs obtained by using runoff coefficients of 45% for both Mawson and Curtin RORB models are shown on Figure 10 together with a Comparison with the recommended composite FFCs.

9.4 Accuracy of RORB models calibrated against LP3 FFCs.

In flood studies where data from historically recorded storm/flood events is not available, RORB models are typically calibrated against minor flood flows as given by LP3 partial series curves up to the 10 year ARI, (assuming 5 to 10 years of streamflow data for LP3 curves is available). For the Curtin FFC the AEP 1 in 2, 1 in 5, and 1 in 10 flows are all based on the LP3 partial series for the Curtin gauging station. Table 9.2 shows the calibrated runoff coefficients determined for the Curtin AEP 1 in 2, 1 in 5, and 1 in 10 flood events are 50%, 40%, and 40% respectively. These results indicate a value of 40% to 45% would be appropriate for the runoff coefficient. However if a runoff coefficient of 45% were adopted for all the Curtin design storm flood events the resulting predicted flood flows would be accurate to within -5% to + 18% of the FFC flood flows, refer Table 9.4.

Following a similar exercise for the Mawson catchment a Runoff coefficient of 55%, based on the calibrated runoff coefficients for the AEP 1 in 2, 1 in 5, and 1 in 10 storm events of 65%, 55%, and 50% respectively, would result in the flood estimates as given in Table 9.5.

Table 9.5

Mawson Catchment RORB predicted Peak Flows Using Runoff Coefficient Calibrated against the 2, 5, and 10 year ARI LP3 partial flows. (Runoff Coefficient = 55%)

Storm AEP	Peak Flow as per FFC (cumecs)	RORB predicted Flow (cumecs)	Comparison to FFC
1 in 2	22	20	- 9%
1 in 5	29	30	+ 3%
1 in 10	34	36	+ 6%
1 in 20	43	45	+ 5%
1 in 50	63 (60)	54	- 14% (- 10%)
1 in 100	80 (75)	63	- 21% (- 16%)

These results indicate that adopting a runoff coefficient based on calibration against the 2, 5, and 10 year LP3 partial series flow estimates, results in RORB predicted flows with an estimated accuracy of -21% to +6%. However if the revised estimates of the Mawson AEP 1 in 50 and 1 in 100 flood flows of 60 cumecs and 75 cumecs respectively are adopted (refer Section 9.3.3), then the accuracy of the Mawson RORB flood estimates based on the runoff coefficients calibrated against the 2, 5, and 10 year ARI LP3 flows becomes -16% to + 6%.

The combined results from the Mawson and Curtin analyses indicate that adopting runoff coefficients based on calibrated runoff coefficients for the 2, 5, and 10 year ARI LP3 flood flows results in predicted flows for flood events ranging from AEP = 1 in 2 to 1 in 100, having an estimated accuracy of approximately plus or minus 15%.

Caution must be exercised in interpreting the accuracy of RORB flow estimates based on calibration of RORB models against the 2, 5, and 10 year ARI LP3 flows as such accuracies will certainly be influenced by the accuracy and suitability of the given LP3 flows as governed by the length and variability of the recorded streamflows.

The FFCs obtained by using the Runoff coefficients calibrated against the LP3 FFCs are shown on Figure 11 together with a Comparison with the recommended composite FFCs for Curtin and Mawson.

10. References.

Bureau of Meteorology (1972). " Final Report Woden Valley Storm 26 January 1971. "

The Institution of Engineers, Australia, IEAust. (1987), " Australian Rainfall and Runoff ".

Laurenson and Mein (1983). Monash University Department of Civil Engineering, " RORB Version 3 Runoff Routing Program User Manual "

Knee R.M. (1990). " Flood Frequency Analysis for Urban Catchments in the ACT ".

APPENDIX A. ACT DEPARTMENT OF URBAN SERVICES DOCUMENT RE: JAN. 1971 FLOOD EVENT.

FLOOD FLOW OF 26/01/91 AT YARRALUMLA CK. AT CURTIN (stn no 410745)

Data available: - cross sections, debris levels and flood slopes. The measured flood slope varies at the recorder and changes substantially just downstream of it.

These data plus estimates of "n" (based on text book values and values from Knee (1990)) were used to get estimates of flow at Yarralumla Ck at Curtin (including flow along Yarra Glen). [NB. Previous estimates from 1971 included Yarra Glen flows].

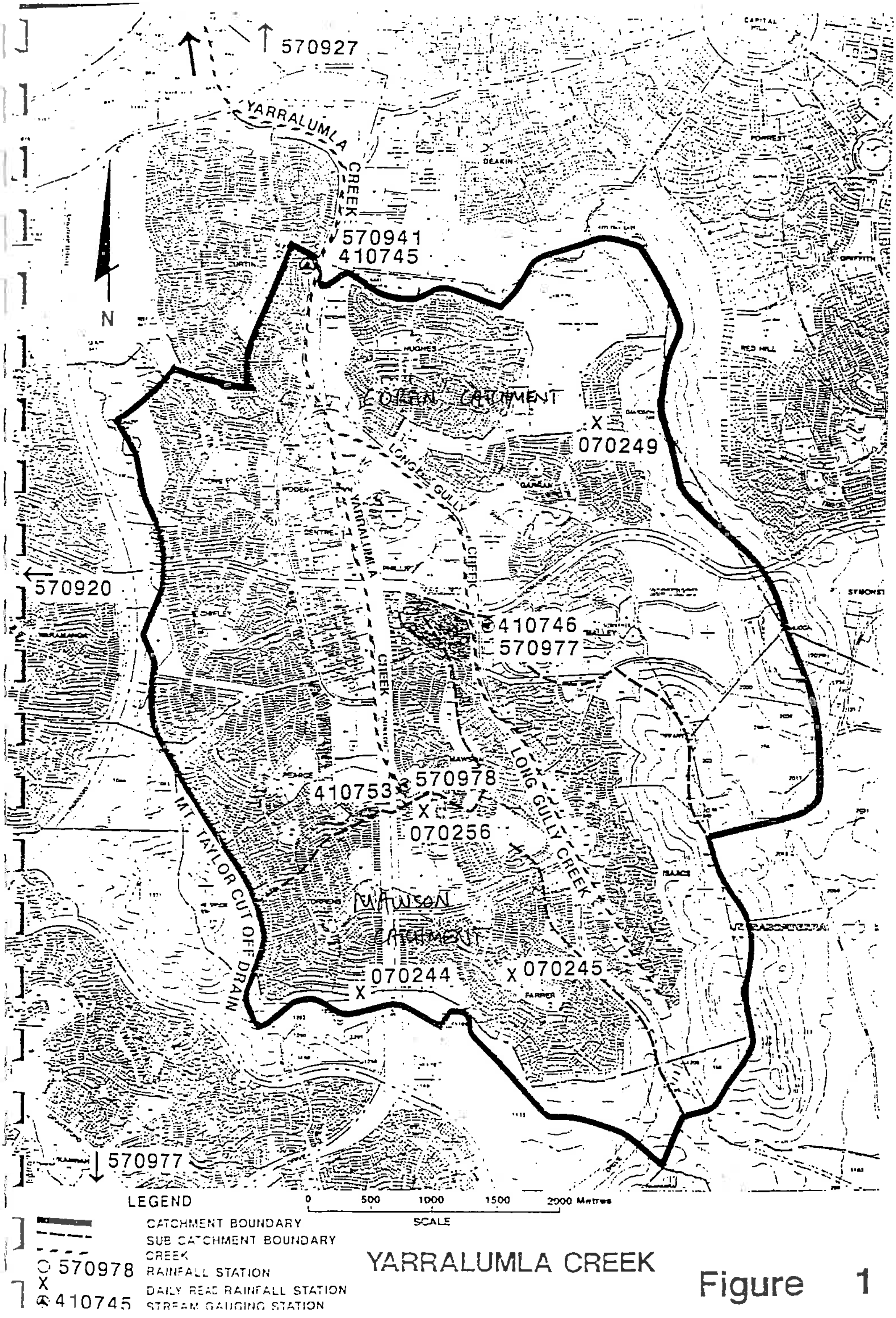
Using the above data, a reasonable estimate of the flow is 240 cumecs. However, considering the range of flood slopes and values of "n", the flow could be anything from 175 cumecs to 350 cumecs.

Using this range of flows in an LPIII annual series flood frequency analysis, the 100year ARI flood ranges from 200 to 420 cumecs (not including an increase in range given by the confidence limits). If the 1971 flood is removed from the annual series, the 100 year ARI flood is 120 cumecs.

From the graph, a 240 cumec flow can be considered to have an ARI of between 35 years and 200 Years (not including the annual series without 1971, and not including the increase in range given by confidence limits)

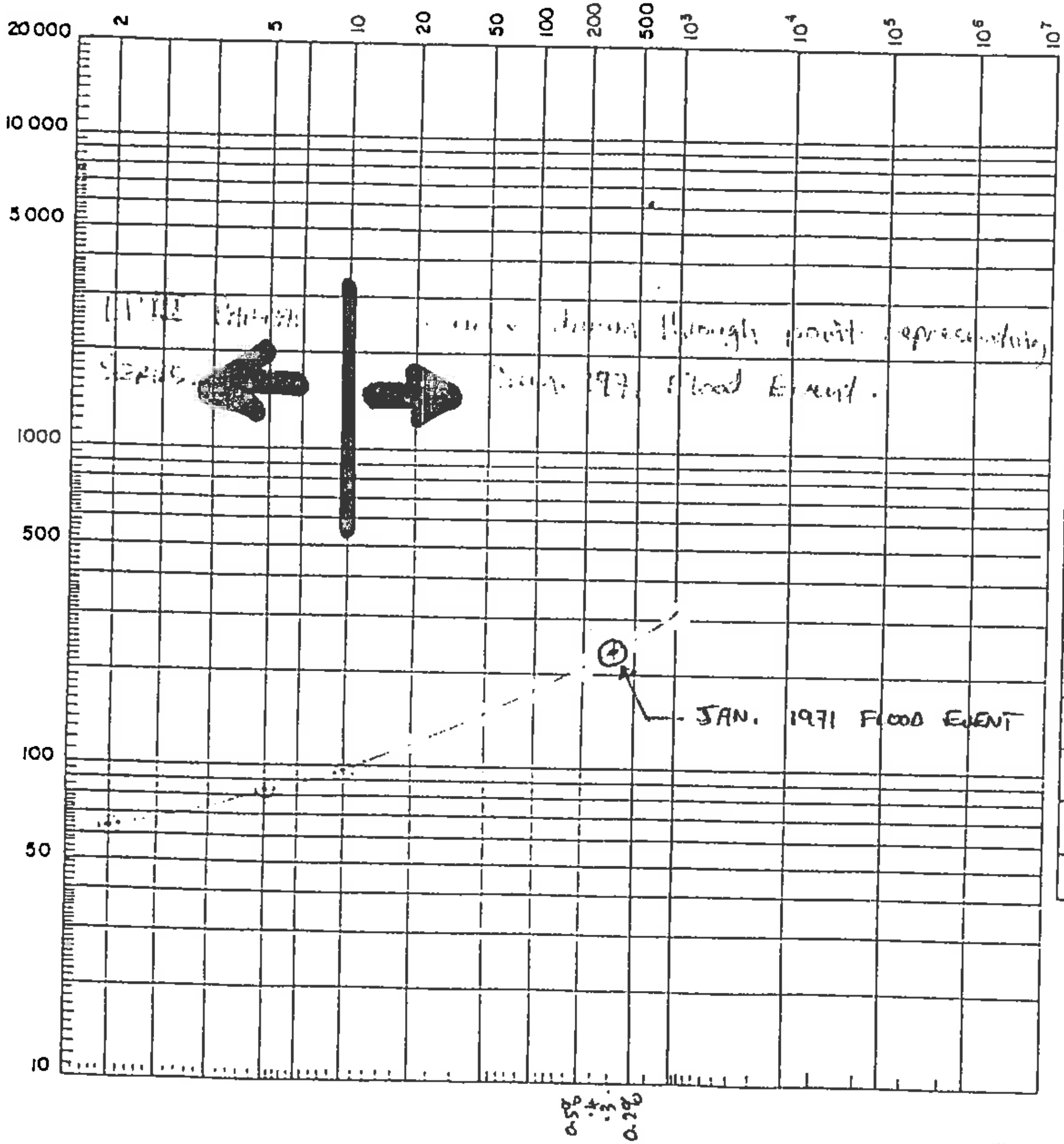
The average rainfall over the catchment of Yarralumla Ck at Curtin is approximately 60mm. Depending on whether this rainfall occurred over a period of 48 or 60 mins (both mentioned), this gives a rainfall ARI of just over 100 years(60mm/hr)to a lot more than 100 years (75mm/hr). Since the catchment was a wet one (eg. there were two overtoppings of the causeway the day before the Australia Day flood), one would expect a similar sort of recurrence interval for the streamflow.

NOTE: Flood Flows on 05/02/71.
The concrete lining in the bed under the Carruthers Street bridge was torn up approximately half an hour before the peak of the flow. The blocks of concrete were standing up at an angle, causing an increase in gauge height while the surface velocity was observed to decrease. Thus, what appeared to be a reasonable estimate of flow, may not be so.



YARRALUMLA CREEK

Figure 1

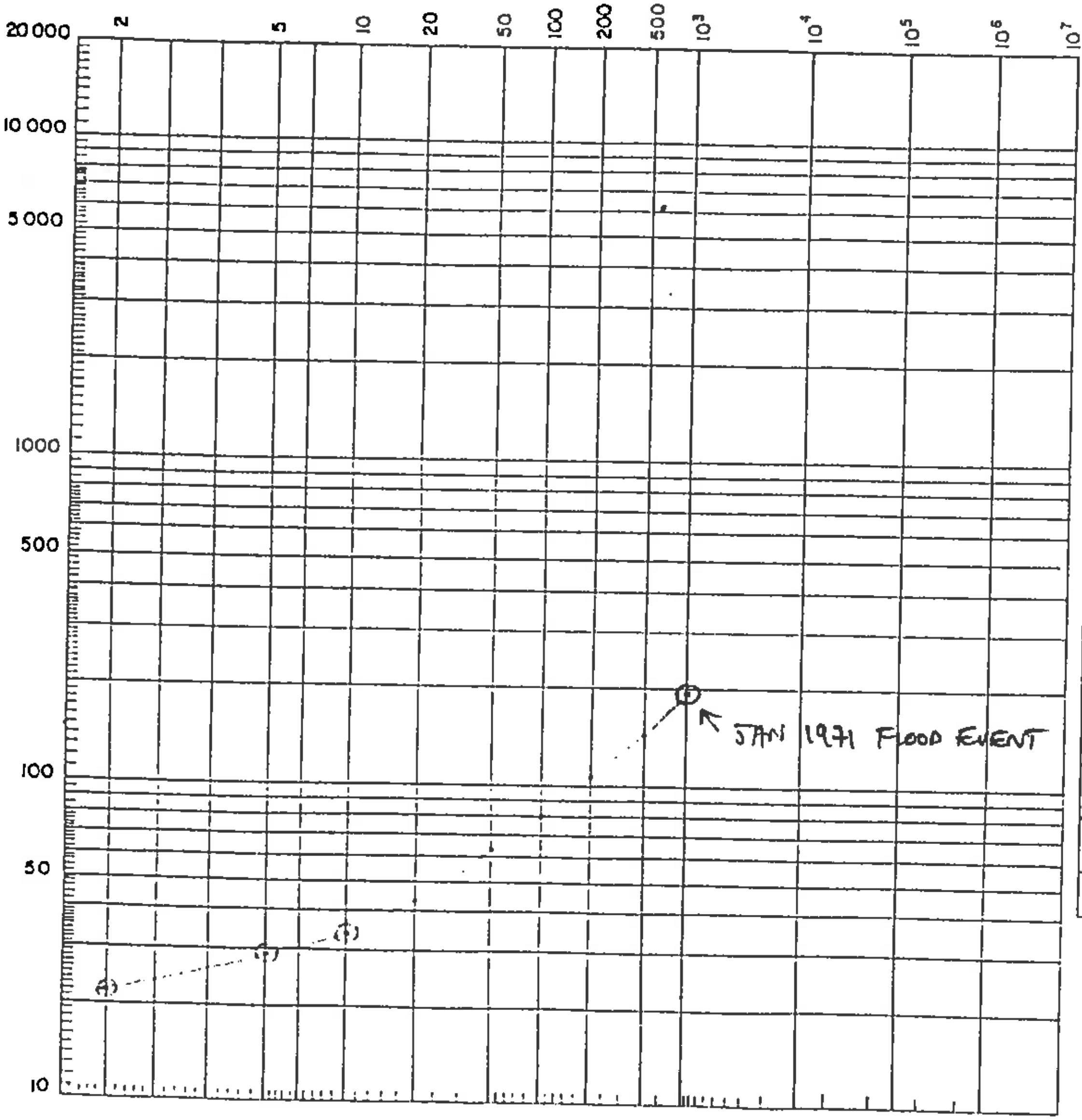


ANNUAL EXCEEDANCE PROBABILITY	PEAK FLOOD FLOW (cumecs)	METHOD OF DETERMINATION
1 in 2	68	LP3 PARTIAL SERIES EXCLUDING 'JAN.' 71 FLOOD EVENT.
1 in 5	83	
1 in 10	97	
1 in 20	119	Scaled from graph.
1 in 25	125	
1 in 50	150	
1 in 100	175	
1 in 200	210	
1 in 250	250	JAN. 1971 FLOOD EVENT.
1 in 500	260	Scaled from graph.
1 in 1000	315	
1 in 2000		
1 in 10^4		
1 in 2×10^4		
1 in 5×10^4		
1 in 10^5		
1 in 10^6		

ACT URBAN CATCHMENT FLOOD STUDY.

A RORB MODEL OF THE YARRALUMLA CK. CATCHMENT AREA.

FIGURE 2. RECOMMENDED COMPOSITE FFC FOR CURTIN G.C.

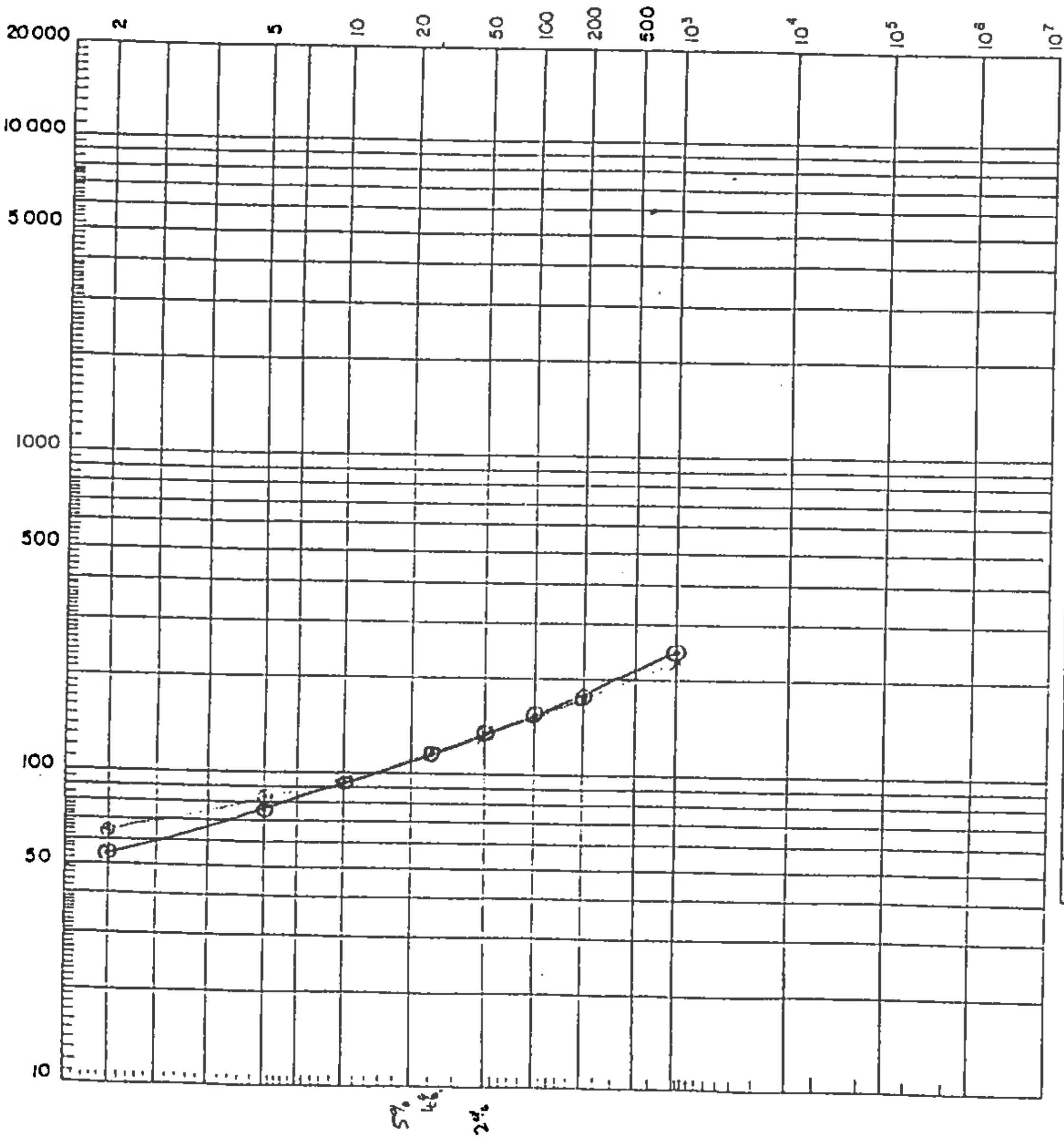


ANNUAL EXCEEDANCE PROBABILITY	PEAK FLOOD FLOW (cumecs)	METHOD OF DETERMINATION
1 in 2	22	LPIII PARTIAL SERIES EXCLUDING JAN. 1971 FLOOD EVENT.
1 in 5	29	
1 in 10	34	
1 in 20	43	SCALED FROM GRAPH.
1 in 25	50	
1 in 50	63	
1 in 100	80	
1 in 200	106	
1 in 500	145	
1 in 1000	195	JAN. 1971 FLOOD EVENT.
1 in 2000		
1 in 10^4		
1 in 2×10^4		
1 in 5×10^4		
1 in 10^5		
1 in 10^6		

ACT URBAN CATCHMENT FLOOD STUDY.

A RORB MODEL OF THE YARRALUMLA CK. CATCHMENT AREA.

FIGURE 3. RECOMMENDED DESIGN [illegible] FOR MAWSON G.S.

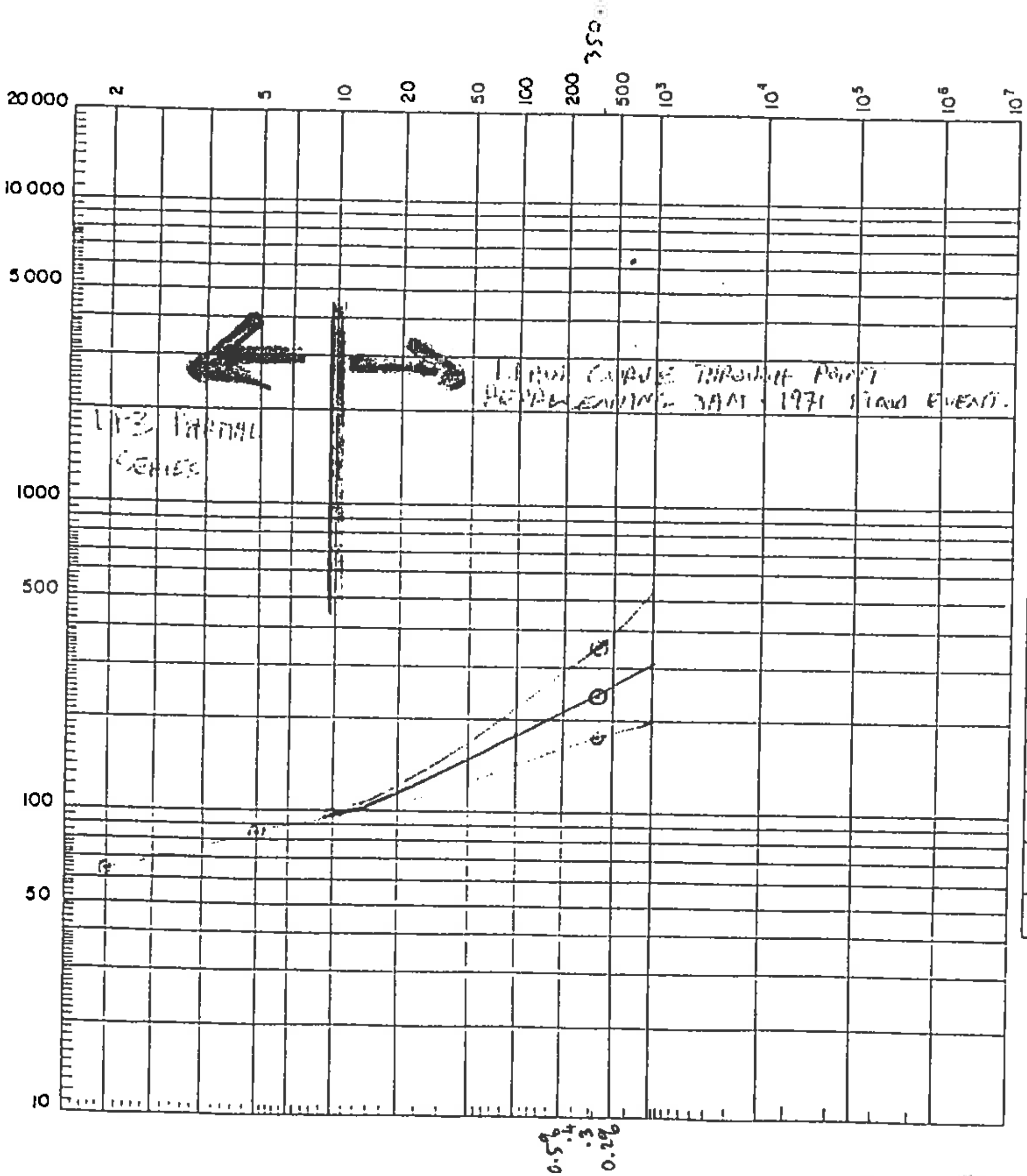


ANNUAL EXCEEDANCE PROBABILITY	PEAK FLOOD FLOW (cumecs) Partial Series	Annual Series
1 in 2	64	54
1 in 5	83	76
1 in 10	97	93
1 in 20		
1 in 25	118	115
1 in 50	135	135
1 in 100	154	156
1 in 200	175	179
1 in 500		
1 in 1000	232	240
1 in 2000		
1 in 10^4		
1 in 2×10^4		
1 in 5×10^4		
1 in 10^5		
1 in 10^6		

ACT URBAN CATCHMENT FLOOD STUDY

A RORB MODEL OF THE YARRALUMLA CK. CATCHMENT AREA

FIGURE [illegible] PARTIAL AND ANNUAL SERIES [illegible]

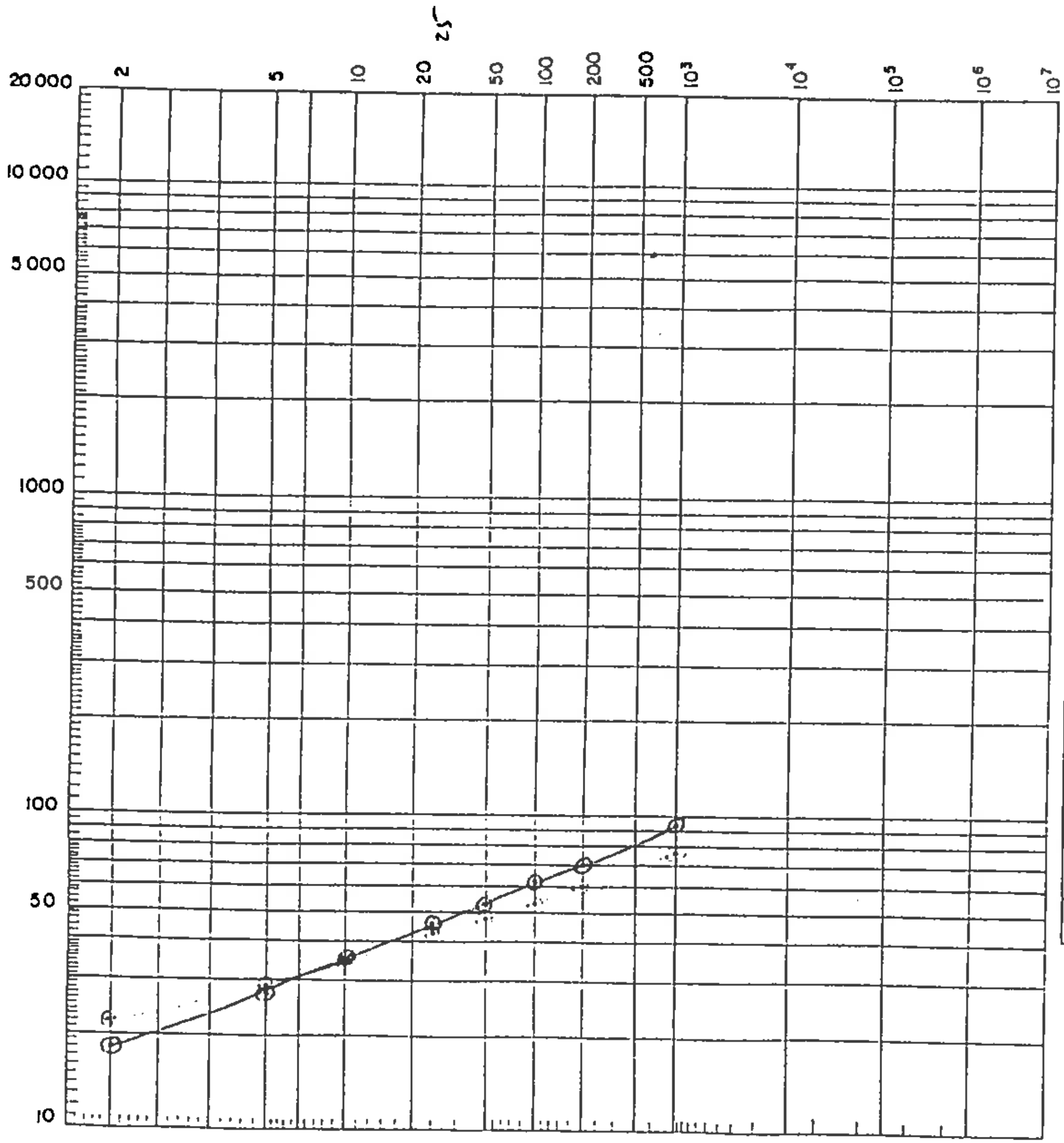


ANNUAL EXCEEDANCE PROBABILITY	PEAK FLOOD FLOW (cumecs)	METHOD OF DETERMINATION		
1 in 2	64	64	64	LPIII PARTIAL SERIES
1 in 5	83	83	83	
1 in 10	97	97	97	
1 in 20	110	119	122	SCALED FROM GRAPH
1 in 25	116	125		
1 in 50	130	150	169	
1 in 100	145	180	220	
1 in 200	160	210	300	
1 in 350	175	240	350	JAN '71 FLOOD
1 in 500	180	260	400	SCALED FROM GRAPH.
1 in 1000	200	315	520	
1 in 2000				
1 in 10^4	LOWER ESTIMATE OF JAN. '71 FLOW = 175 CUMECS.	ESTIMATED JAN. '71 FLOW = 240 CUMECS.	UPPER ESTIMATE OF JAN. '71 FLOW = 350 CUMECS.	
1 in 2×10^4				
1 in 5×10^4				
1 in 10^5				
1 in 10^6				

ACT URBAN CATCHMENT FLOOD STUDY

A RORB MODEL OF THE YARRALUMLA CK. CATCHMENT AREA

FIGURE [illegible] BASED ON EXTREME ESTIMATES OF JAN. 1971 FLOOD FLOW.

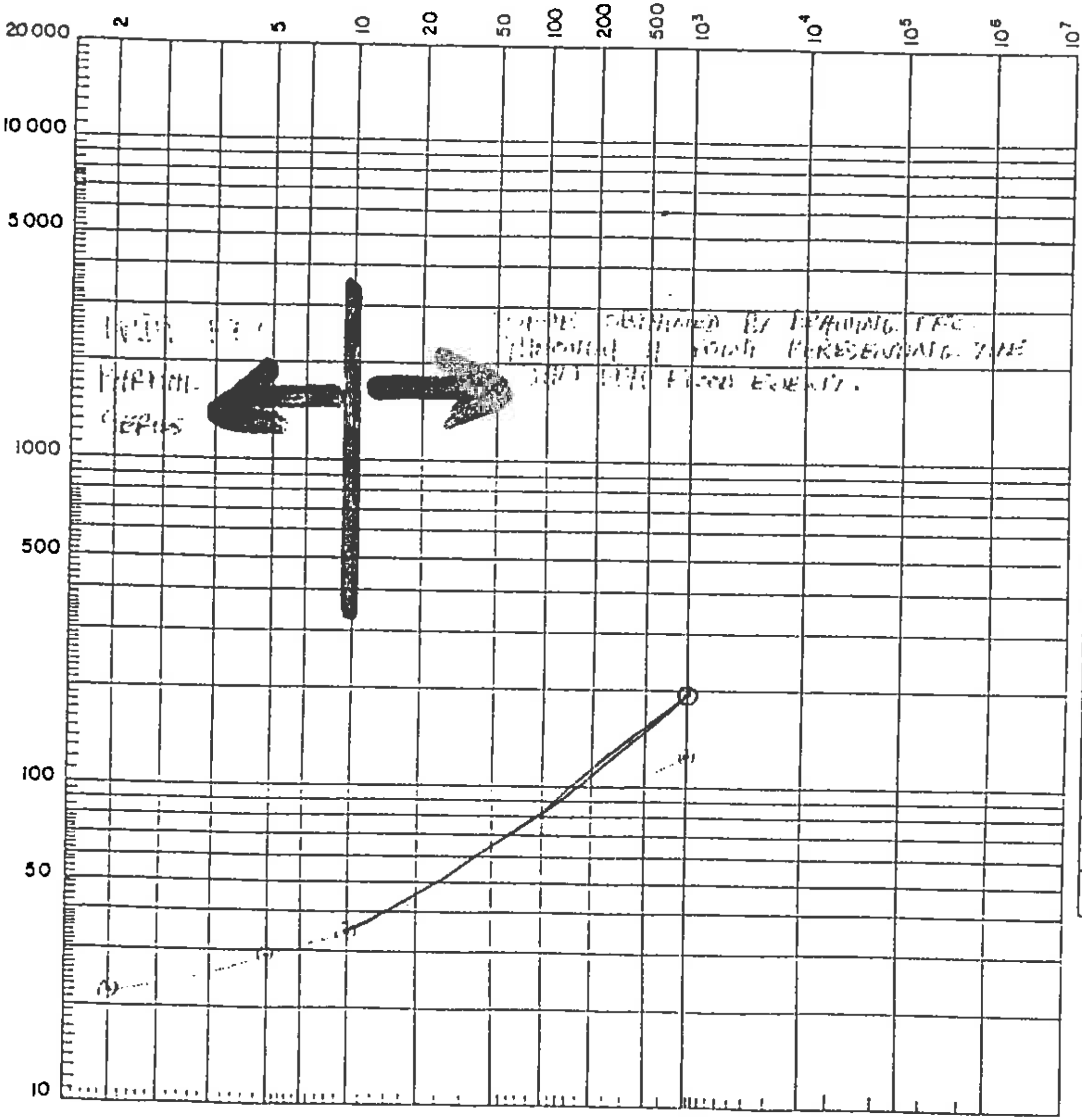


ANNUAL EXCEEDANCE PROBABILITY	PEAK FLOOD FLOW (cumecs) PARTIAL SERIES.	ANNUAL SERIES.
1 in 2	22	18
1 in 5	29	28
1 in 10	34	35
1 in 20	-	
1 in 25	42	45
1 in 50	47	53
1 in 100	53	61
1 in 200	60	70
1 in 500		
1 in 1000	77	93
1 in 2000		
1 in 10^4		
1 in 2×10^4		
1 in 5×10^4		
1 in 10^5		
1 in 10^6		

ACT URBAN CATCHMENT FLOOD STUDY.

A RORB MODEL OF THE YARRALUMLA CK. CATCHMENT AREA.

FIGURE ... PARTIAL AND ANNUAL SERIES ...

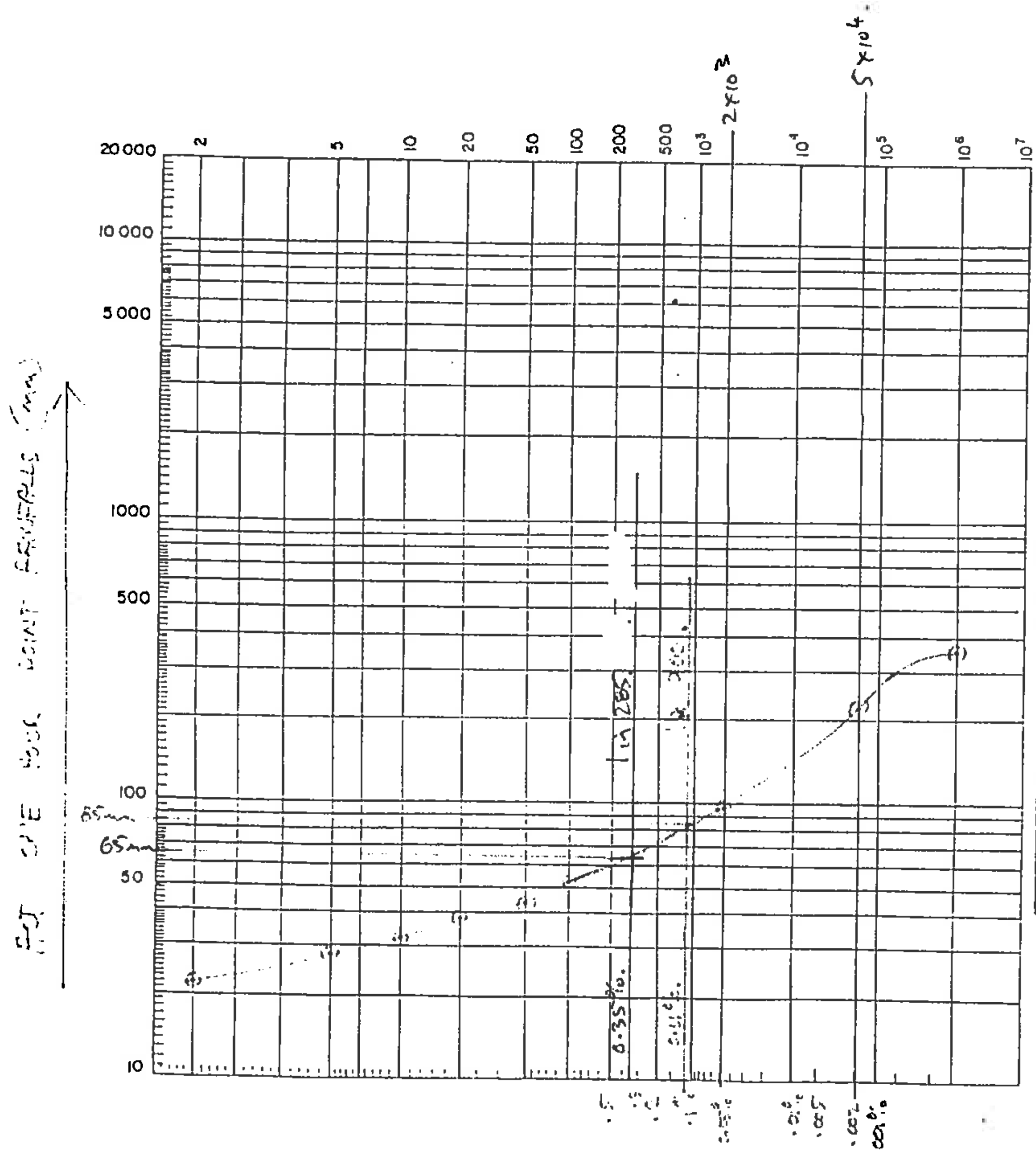


ANNUAL EXCEEDANCE PROBABILITY	PEAK FLOOD FLOW (cumecs)		METHOD OF DETERMINATION
1 in 2	22	22	LPIII ANNUAL SERIES.
1 in 5	24	29	
1 in 10	34	34	
1 in 20	43	43	SCALED FROM GRAPH.
1 in 25	46	50	
1 in 50	56	63	
1 in 100	70	80	
1 in 200	83	106	
1 in 500	105	145	
1 in 1000	127	195	JAN. 1971 FLOOD EVENT.
1 in 2000			
1 in 10^4			
1 in 2×10^4			
1 in 5×10^4			
1 in 10^5			
1 in 10^8			

ACT URBAN CATCHMENT FLOOD STUDY.

A RORB MODEL OF THE YARRALUMLA CK. CATCHMENT AREA.

FIGURE 4. ... BASED ON EXTREME ESTIMATES OF JAN. 1971 FLOOD FLOW.



ANNUAL EXCEEDANCE PROBABILITY	ONE HOUR RAINFALLS (mm)	METHOD OF DETERMINATION
1 in 2	22	
1 in 5	28	
1 in 10	32.	
1 in 20	38	
1 in 25	-	
1 in 50	44	
1 in 100	51	
1 in 200		
1 in 285	65 mm	Scaled from graph.
1 in 500		
1 in 900		
1 in 1000	85 mm	Scaled from graph.
1 in 2000	98	ARR 1987. Vol. 1. Rowbottom Method.
1 in 10^4		
1 in 2×10^4		
1 in 5×10^4	225	ARR 1987. Vol. 1 Rowbottom Method.
1 in 10^5		
1 in 10^6 PMP	360	Met. Bulletin 51, 1984. (Bureau of Meteorology, 1984.)

ACT URBAN CATCHMENT FLOOD STUDY.

A RORB MODEL OF THE YARRALUMLA CK. CATCHMENT AREA

FIGURE 8.? PROBABILITY-FREQUENCY DIAGRAM FOR ONE HOUR DURATION POINT RAINFALLS IN THE ACT

CURTIN CATCHMENT

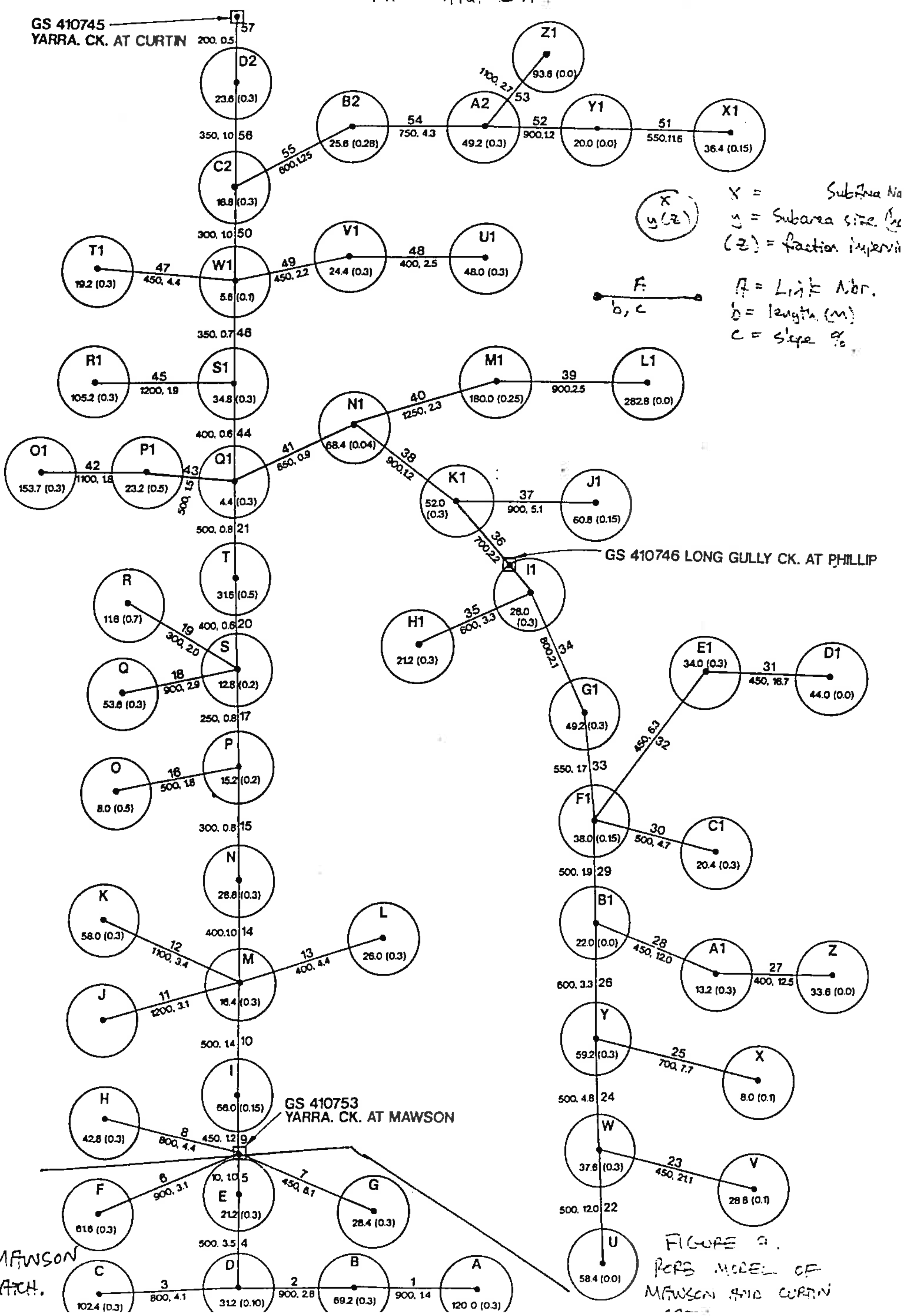
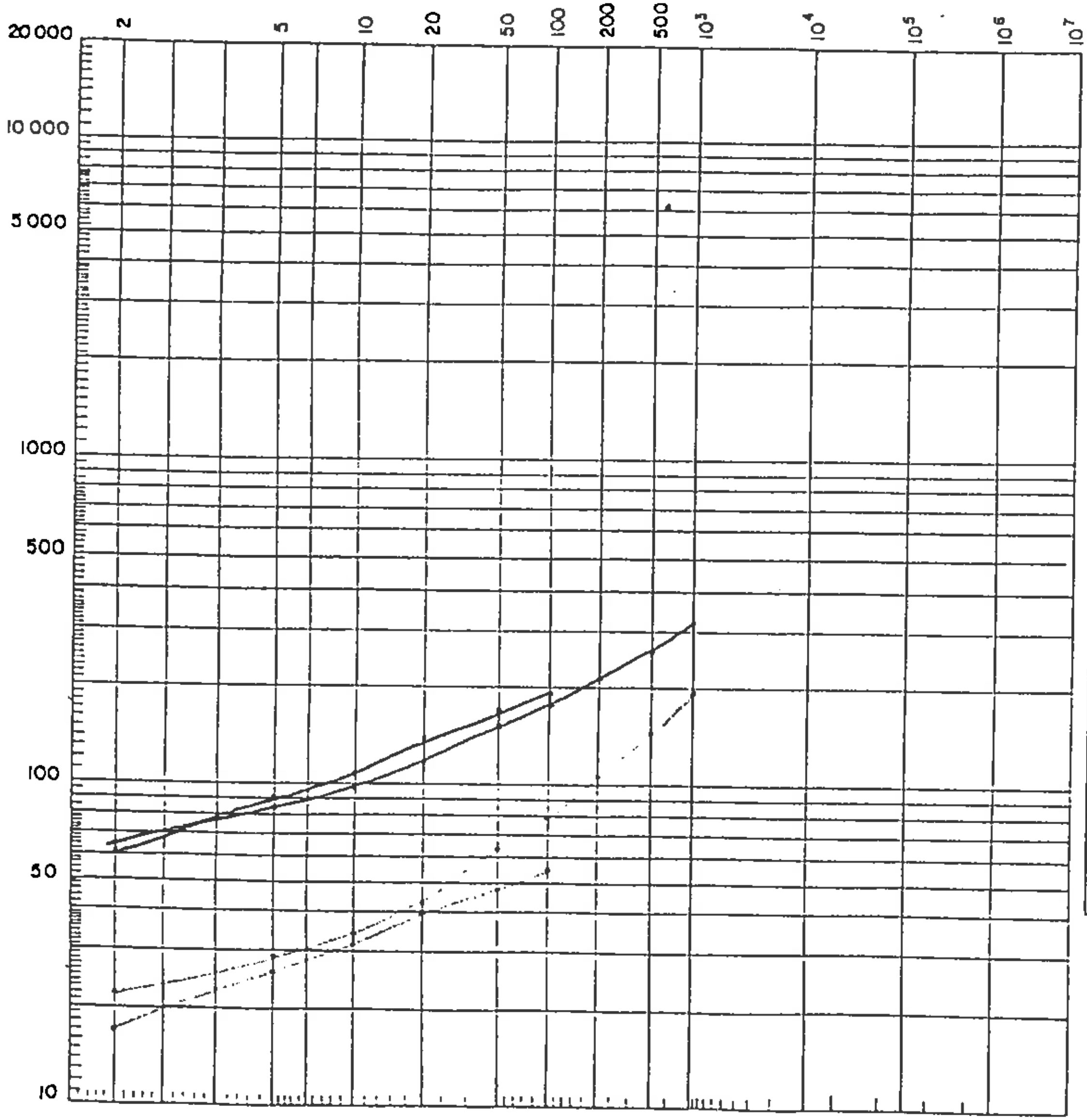


FIGURE 9.
RORB MODEL OF
MAWSON AND CURTIN
CATCH...

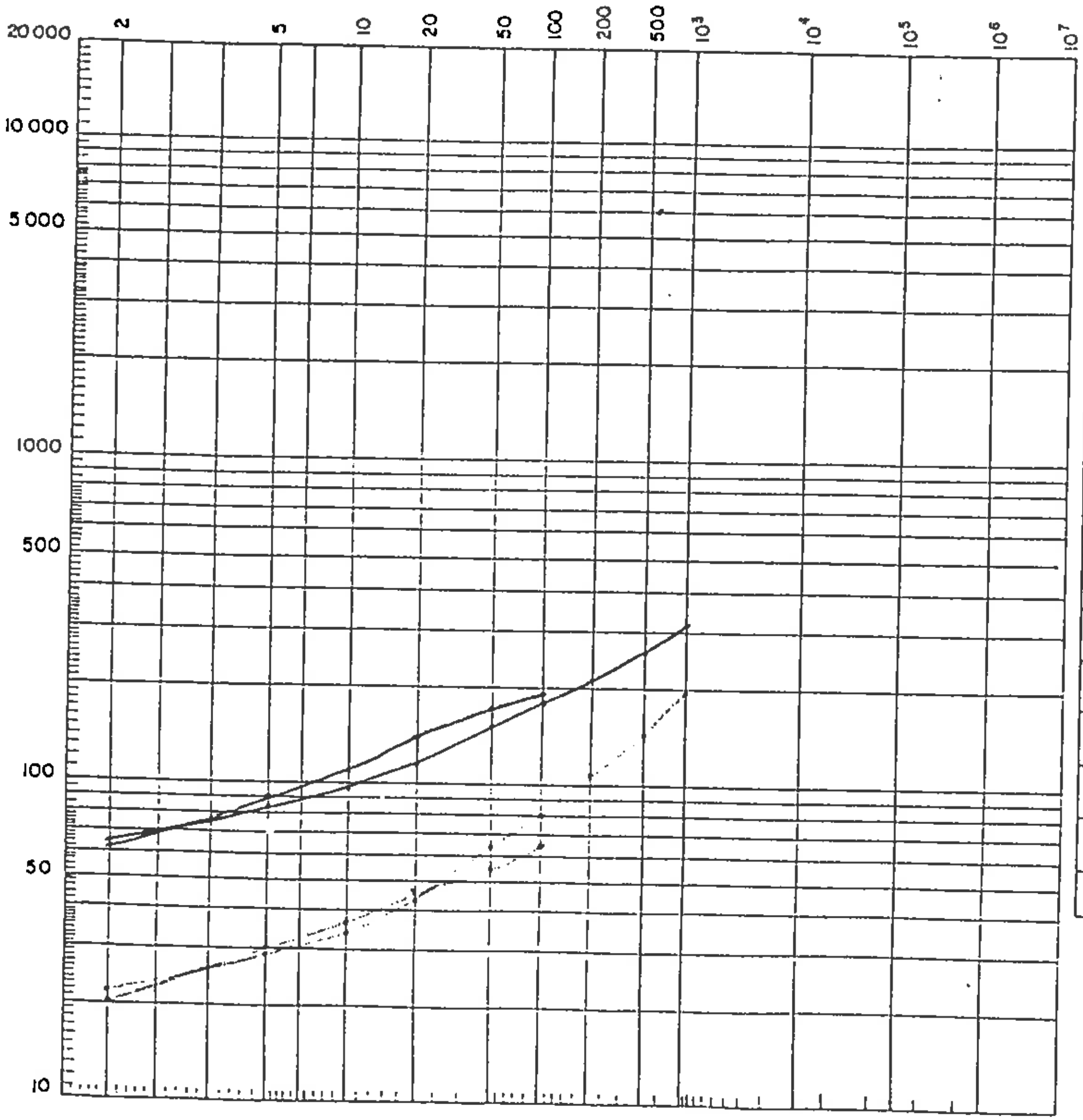


ANNUAL EXCEEDANCE PROBABILITY	MAWSON G.S.		CURTIN G.S.	
	Recommend FFC	45% FFC	RECOMMEND FFC	45% RUNOFF COEFF. FFC
1 in 2	22	17	64	61
1 in 5	29	26	83	90
1 in 10	34	31	97	108
1 in 20	43	40	119	140
1 in 25	50	—	125	—
1 in 50	63	47	150	174
1 in 100	80	55	175	190
1 in 200	106		210	
1 in 350			240	
1 in 500	145		260	
1 in 1000	195		315	
1 in 2000				
1 in 10^4				
1 in 2×10^4				
1 in 5×10^4				
1 in 10^5				
1 in 10^6				

ACT URBAN CATCHMENT FLOOD STUDY.

A RORB MODEL OF THE YARRALUMLA CK. CATCHMENT AREA.

FIGURE 16. COMPARISON OF FFCS OBTAINED USING RUNOFF COEFFICIENT = 45% WITH RECOMMENDED COMPOSITE FFCS FOR CURTIN AND MAWSON.



ANNUAL EXCEEDANCE PROBABILITY	MAWSON G.S. [illegible] FFC	MAWSON G.S. RUNOFF COEFF = 55%	CURTIN G.S. RECOMMENDED FFC	CURTIN G.S. RUNOFF COEFF = 45%
1 in 2	22	20	64	61
1 in 5	29	30	83	90
1 in 10	34	36	97	108
1 in 20	43	45	119	140
1 in 25	50	—	125	—
1 in 50	63	54	150	174
1 in 100	80	63	180	190
1 in 200	106		210	
1 in 350			240	
1 in 500	145		260	
1 in 1000	155		315	
1 in 2000				
1 in 10^4				
1 in 2×10^4				
1 in 5×10^4				
1 in 10^5				
1 in 10^6				

ACT URBAN CATCHMENT FLOOD STUDY.

A RORB MODEL OF THE YARRALUMLA CK. CATCHMENT AREA.

FIGURE 11. COMPARISON OF FFC'S OBTAINED USING RUNOFF COEFFICIENTS CALIBRATED AGAINST THE [illegible] FLOOD, WITH RECOMMENDED FFC'S FOR CURTIN AND MAWSON

APPENDIX D

DESIGN RAINFALL DATA

```
*****************************************************************
*                            I F D                              *
*                                                               *
*  Intensity - Frequency - Duration Design Rainfall Program    *
*                        (Version 2.0)                          *
*                                                               *
*      This software determines IFD design rainfall in         *
*    accordance  with the algebraic procedures presented       *
*         in Chapter 2 (Author : R.P. Canterford) of           *
*             Australian Rainfall & Runoff(1987)                *
*                                                               *
*                ****************************                   *
*                                                               *
*      This software is supplied as is and without any         *
*    warranties as to performance or any other warranties      *
*                   expressed or implied.                       *
*                                                               *
*                    (C) WP SOFTWARE 1988                       *
*                     Ph. (062) 815811                          *
*****************************************************************

*** INPUT DATA ECHO ***

7
 2 year,  1 hour intensity:   22.00 mm/hr
 2 year, 12 hour intensity:    4.40 mm/hr
 2 year, 72 hour intensity:    1.20 mm/hr
50 year,  1 hour intensity:   43.00 mm/hr
50 year, 12 hour intensity:    8.00 mm/hr
50 year, 72 hour intensity:    2.30 mm/hr
Skewness:     .24
Geographical factor for 6 minute,  2 yr storm:    4.28
Geographical factor for 6 minute, 50 yr storm:   15.55
Latitude :      .0000
Longitude:      .0000
```

*** OUTPUT IFD TABLE ***

Rainfall Intensity (mm/h) for 7

Duration	Average Storm Recurrence Interval (years)						
	1	2	5	10	20	50	100
5m	54.97	72.64	98.97	115.31	137.57	168.77	194.09
6	51.45	67.94	92.35	107.46	128.06	156.90	180.28
7	48.53	64.03	86.84	100.94	120.17	147.08	168.86
8	46.03	60.69	82.16	95.40	113.48	138.75	159.18
9	43.87	57.79	78.11	90.61	107.70	131.55	150.83
10	41.96	55.25	74.55	86.41	102.63	125.25	143.52
11	40.26	52.99	71.40	82.68	98.13	119.67	137.06
12	38.74	50.96	68.57	79.35	94.11	114.69	131.28
13	37.37	49.12	66.01	76.34	90.49	110.19	126.08
14	36.11	47.45	63.69	73.61	87.20	106.12	121.36
15	34.97	45.92	61.57	71.11	84.20	102.40	117.06
16	33.91	44.52	59.62	68.82	81.44	98.99	113.12
17	32.94	43.22	57.82	66.70	78.90	95.85	109.49
18	32.03	42.02	56.16	64.75	76.55	92.95	106.13
20	30.40	39.85	53.16	61.23	72.34	87.74	100.11
25	27.12	35.50	47.16	54.20	63.91	77.34	88.12
30	24.62	32.18	42.62	48.88	57.54	69.51	79.10
35	22.64	29.56	39.03	44.69	52.54	63.36	72.02
40	21.02	27.42	36.10	41.28	48.47	58.38	66.29
45	19.67	25.63	33.67	38.45	45.10	54.25	61.55
50	18.52	24.12	31.61	36.06	42.25	50.76	57.54
55	17.53	22.81	29.84	34.00	39.80	47.76	54.11
60	16.66	21.66	28.29	32.20	37.67	45.16	51.12
75	14.51	18.84	24.54	27.89	32.58	39.00	44.11
90	12.93	16.78	21.80	24.75	28.89	34.54	39.04
2.0h	10.76	13.94	18.06	20.46	23.84	28.45	32.11
3.0	8.28	10.71	13.80	15.59	18.13	21.58	24.30
4.0	6.87	8.87	11.39	12.85	14.91	17.71	19.93
5.0	5.95	7.67	9.82	11.06	12.82	15.20	17.08
6.0	5.28	6.81	8.70	9.78	11.33	13.42	15.07
8.0	4.39	5.65	7.19	8.07	9.32	11.03	12.36
10.0	3.80	4.88	6.20	6.95	8.02	9.47	10.61
12.0	3.38	4.34	5.50	6.15	7.09	8.36	9.36
14.0	3.04	3.91	4.96	5.55	6.41	7.57	8.48
16.0	2.77	3.57	4.53	5.08	5.87	6.94	7.77
18.0	2.56	3.29	4.19	4.70	5.43	6.42	7.20
20.0	2.38	3.06	3.90	4.38	5.06	5.99	6.72
22.0	2.23	2.87	3.66	4.11	4.75	5.63	6.31
24.0	2.09	2.70	3.45	3.87	4.48	5.31	5.96
36.0	1.57	2.02	2.60	2.93	3.40	4.03	4.54
48.0	1.26	1.63	2.10	2.38	2.76	3.29	3.70
60.0	1.06	1.37	1.77	2.01	2.33	2.78	3.14
72.0	.91	1.18	1.53	1.73	2.02	2.41	2.72

APPENDIX E

RAFTS SENSITIVITY ANALYSIS

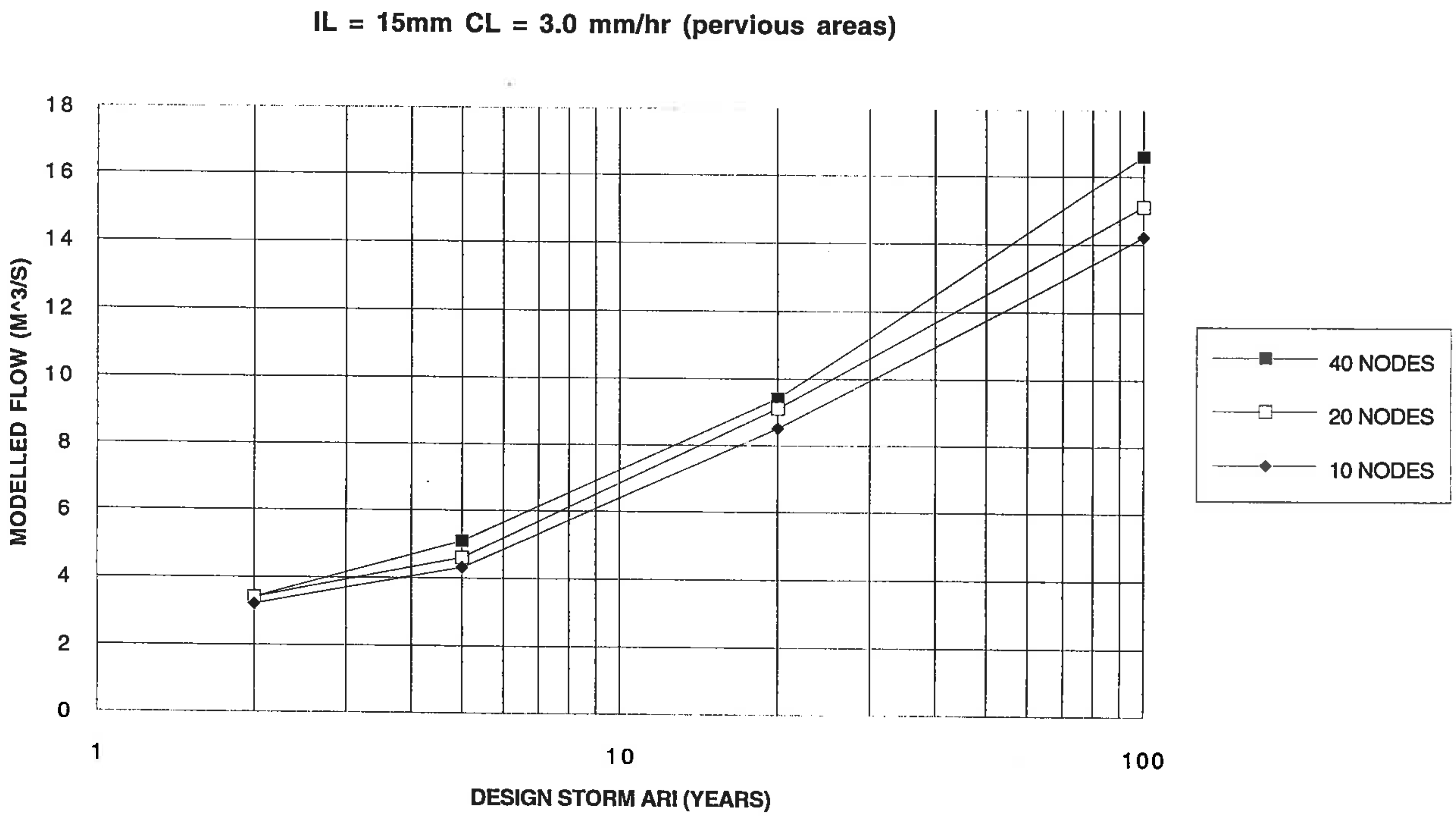


Figure E1

Giralang Catchment

Design Flows - IL = 15mm & CL = 3.0 mm/hr (pervious surfaces)

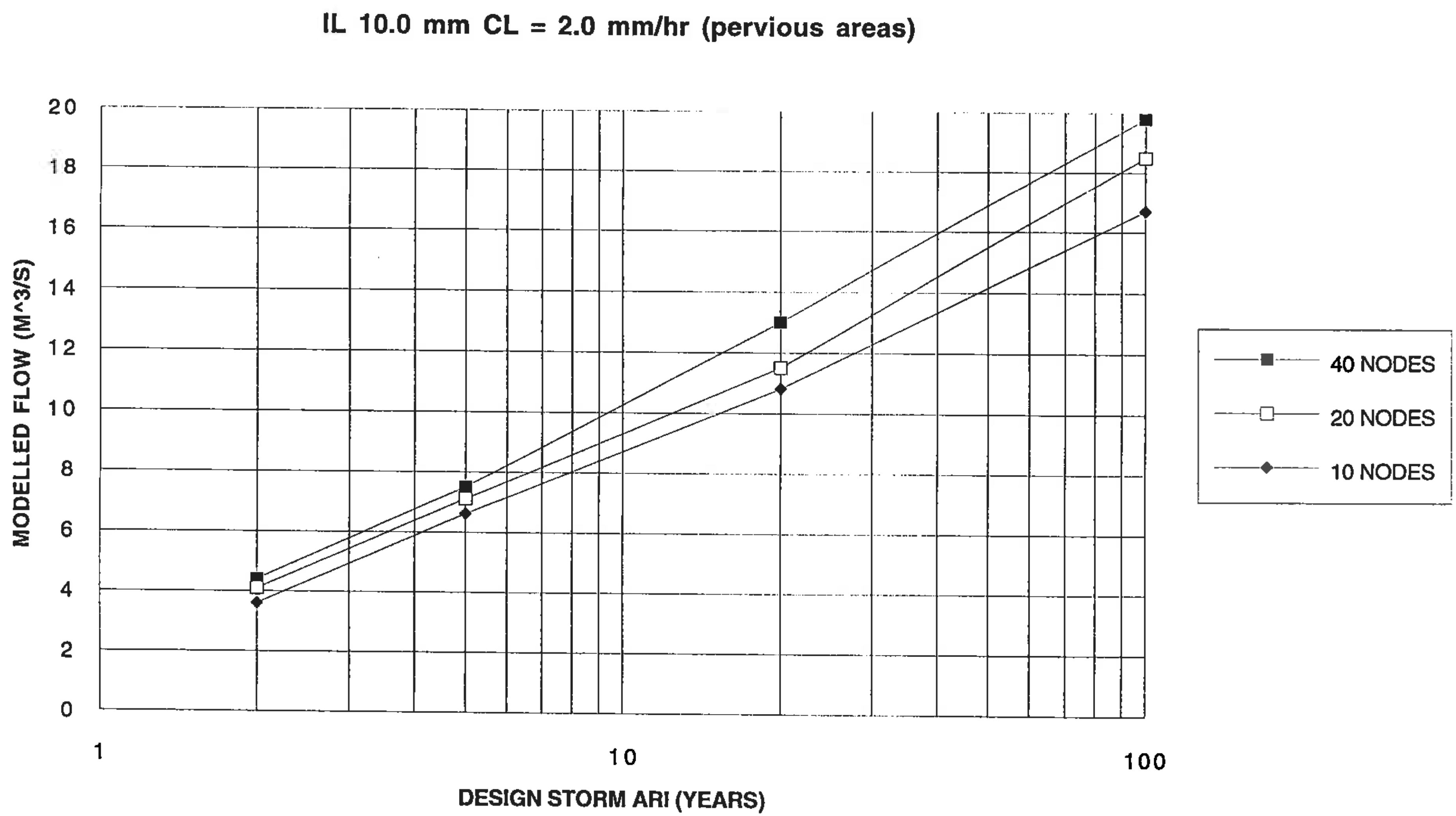


Figure E2

Giralang Catchment

Design Flows - IL = 10mm & CL = 2.0 mm/hr (pervious surfaces)

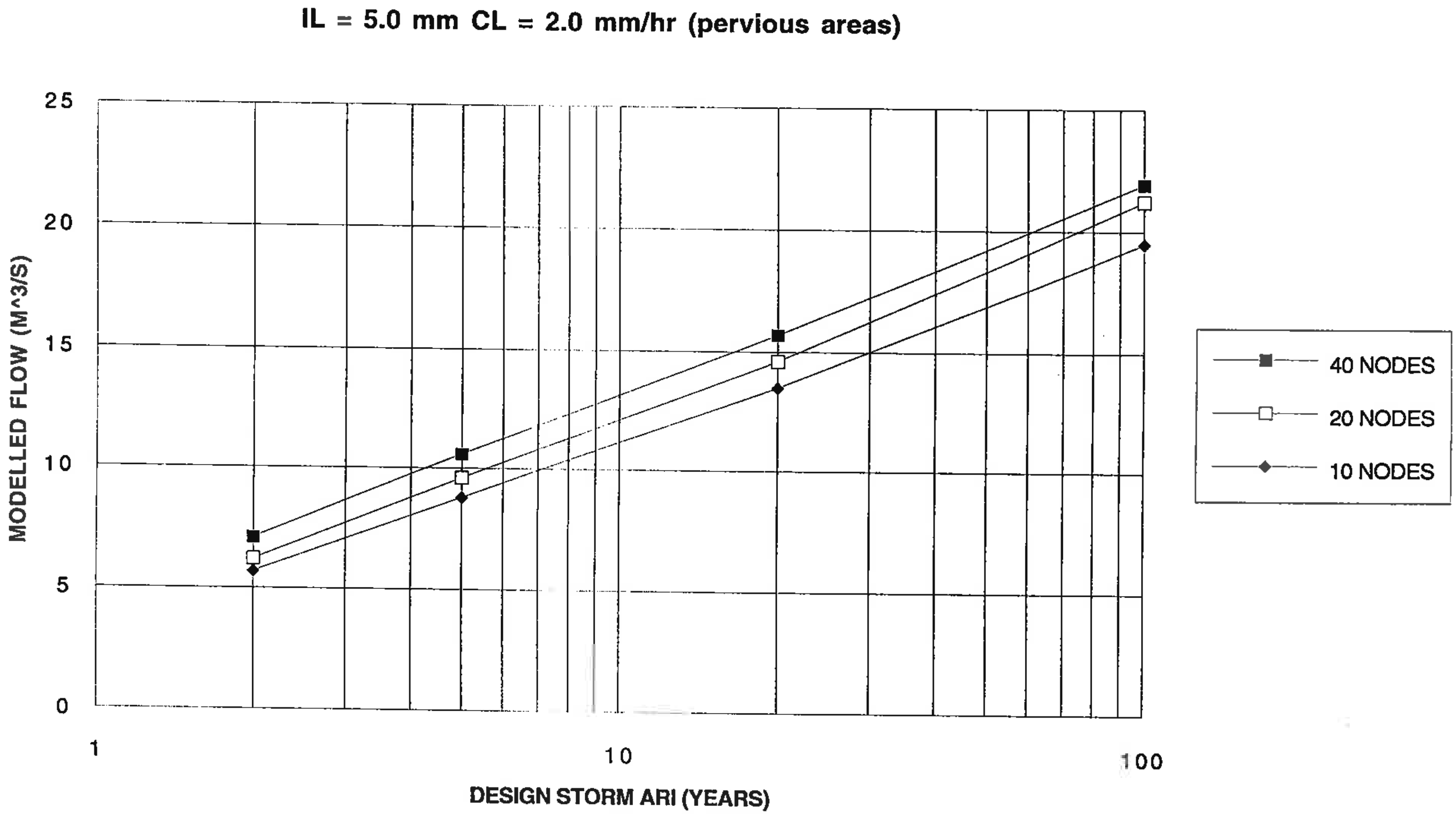


Figure E3

Giralang Catchment

Design Flows - IL = 5mm & CL = 2.0 mm/hr (pervious surfaces)

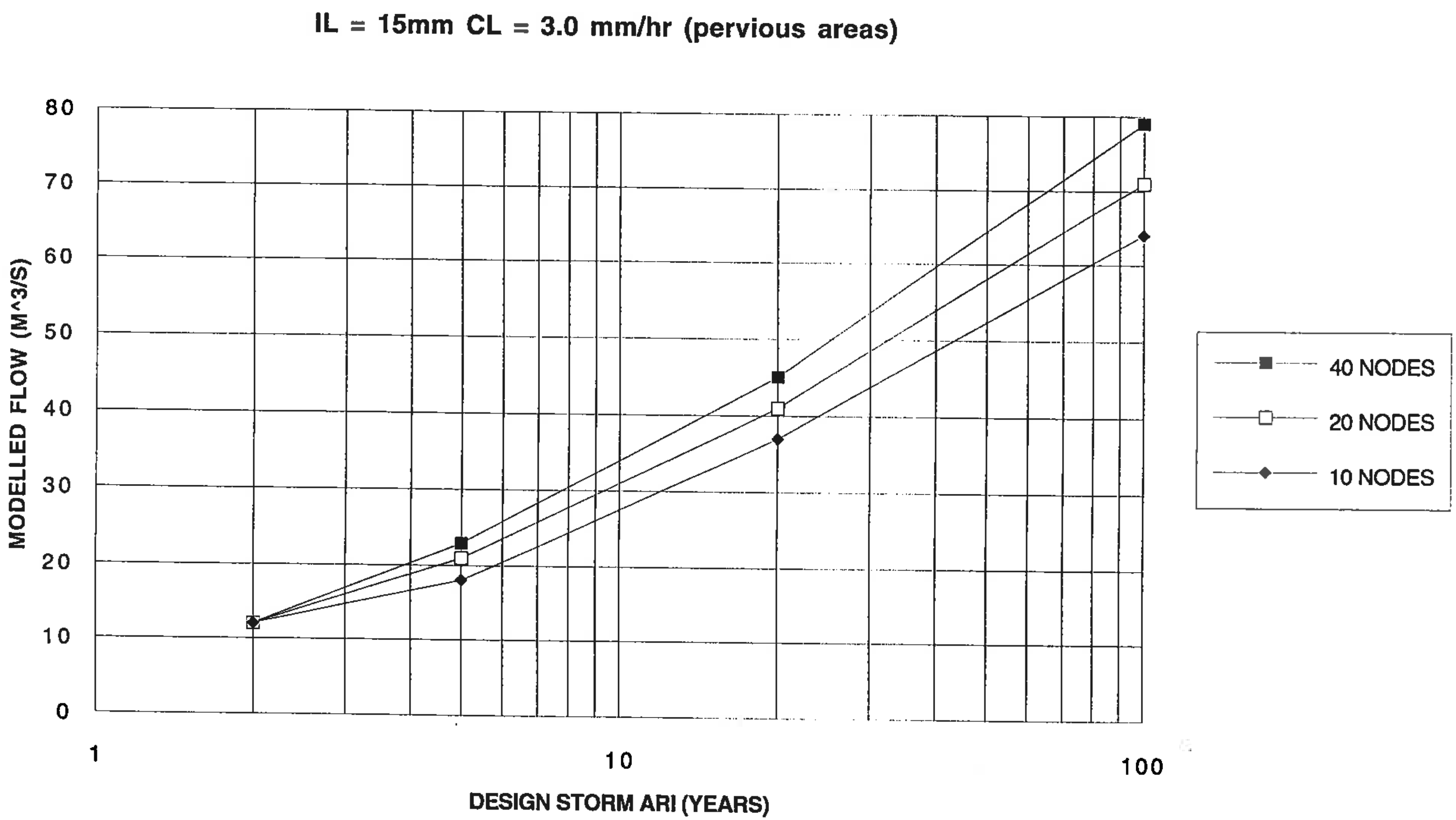


Figure E4

Mawson Catchment

Design Flows - IL = 15mm & CL = 3.0 mm/hr (pervious surfaces)

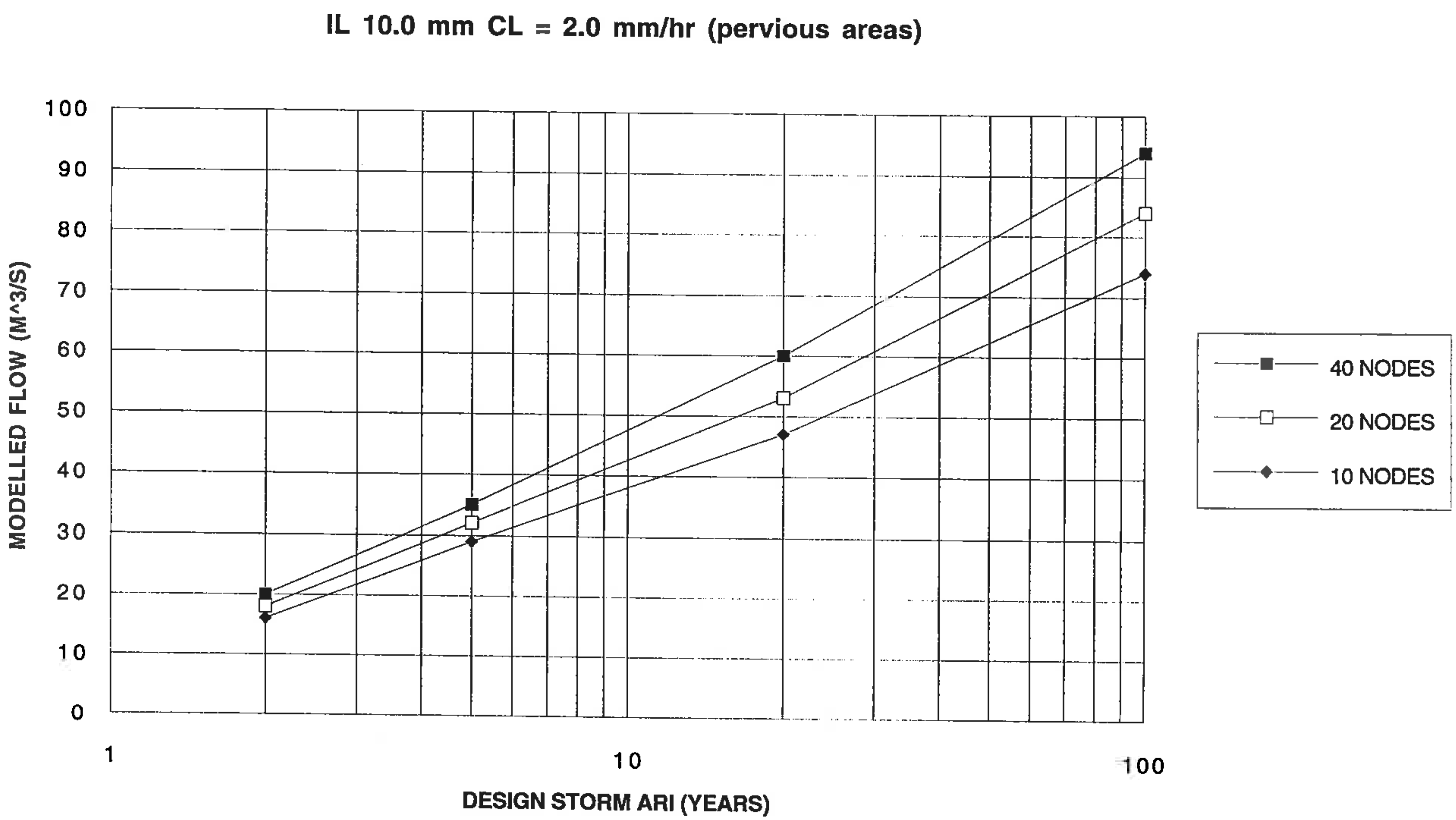


Figure E5

Mawson Catchment

Design Flows - IL = 10mm & CL = 2.0 mm/hr (pervious surfaces)

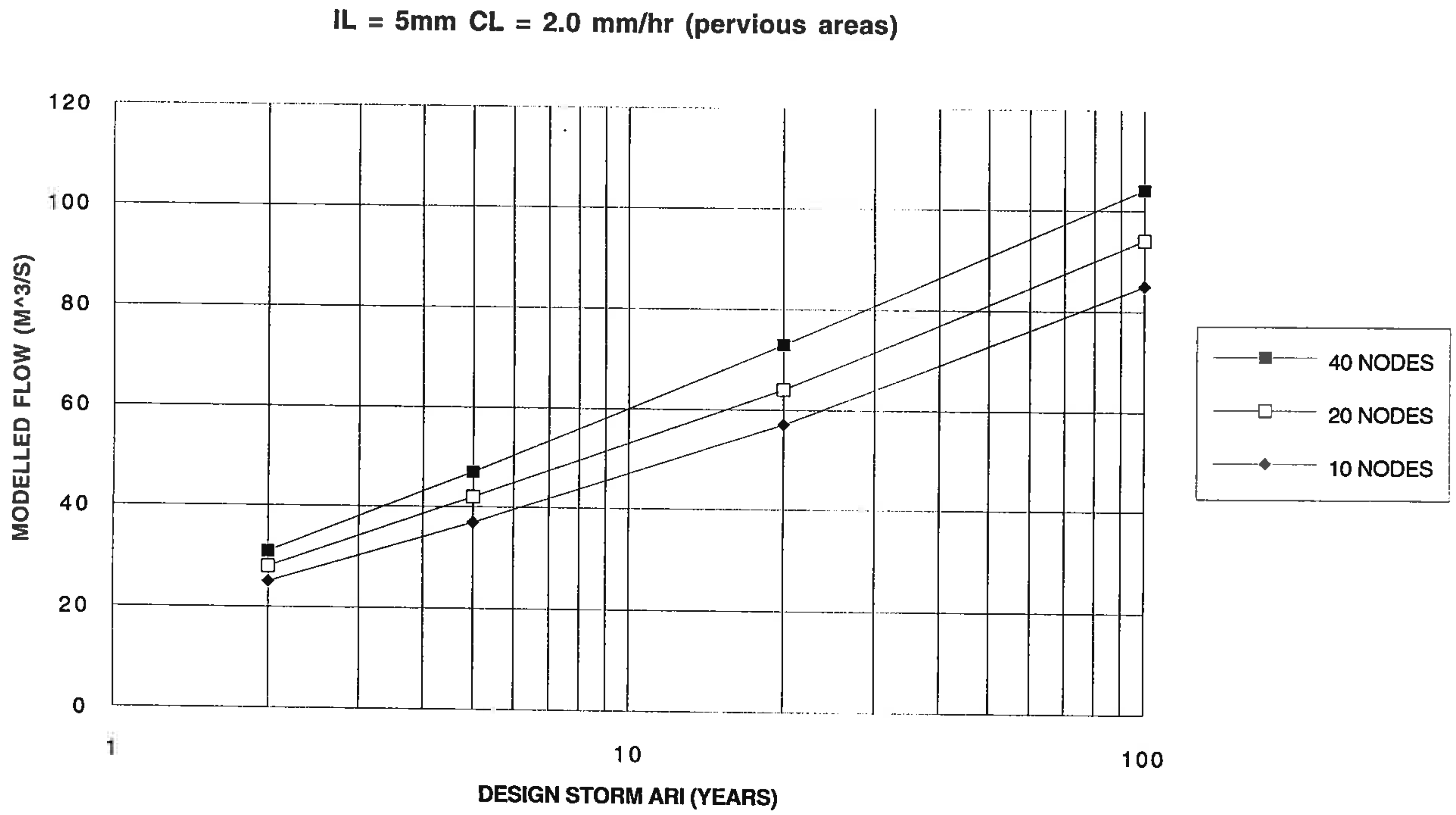


Figure E6

Mawson Catchment

Design Flows - IL = 5mm & CL = 2.0 mm/hr (pervious surfaces)