TESTING OF IMPROVED INPUTS FOR DESIGN FLOOD ESTIMATION IN SOUTH-EASTERN AUSTRALIA

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COOPERATIVE RESEARCH CENTRE FOR
CATCHMENT HYDROLOGY

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PREFACE

In common with all of the core projects of the CRC, Project D1 "Improved Loss Modelling for Design Flood Estimation and Flood Forecasting" was identified and developed by industry and research representatives working together. They saw the need, and potential, for research to address the question, "How much rain becomes runoff?"

This report is related to design hydrograph estimation, in which the starting point is a (hypothetical) storm of a given level of severity. It deals with the specific problem of determining the proportion of the applied storm which becomes streamflow (the remainder being 'loss'), such that the design level of severity of the resultant hydrograph is preserved.

This reports documents the testing program which was used to evaluate the new values of design losses (see CRC Report 96/5). The comparison is based on a 'before and after' approach, in which design hydrographs are generated using old and new values. The clear superiority of the new values is demonstrated in these tests.

I'd like to take this opportunity to acknowledge the efforts of Peter Hill (Project Leader) and his project team for their fine achievements in this work.

Russell Mein
Program Leader

SUMMARY

This report describes the testing on 11 catchments of improved inputs for design flood estimation. The parameters currently recommended in Australian Rainfall and Runoff (IEAust, 1987; AR&R), new design losses derived by Hill et al. (1996) and new areal reduction factors (ARFs) derived by Siriwardena and Weinmann (1996) are tested based upon a comparison with results from a flood frequency analysis. The purpose of these tests was to assess the performance of the new design information in typical applications, in a limited benchmarking exercise.

The application of the design parameters in AR&R consistently overestimated the peak flows; for an AEP of 1 in 10, by an average of 47 percent. The use of the new ARFs resulted in an average reduction in the peak flow of 6 percent for an AEP of 1 in 10 and of 9 percent for an AEP of 1 in 50.

The use of the new initial loss-continuing loss (IL/CL) values with the new ARFs and unfiltered temporal patterns removed the bias encountered using the AR&R guidelines.

The application of the new initial loss-proportional loss (IL/PL) values with the new ARFs and unfiltered temporal patterns produced peak flows which were consistently lower than those obtained from the flood frequency analysis. For an AEP of 1 in 10, the runoff coefficient (one minus the PL) had to be increased by 50 percent to remove the bias. For an AEP of 1 in 50, the runoff coefficient had to be increased by 80 percent. This is an impediment to the use of the IL/PL model for design.

When applied in conjunction with unfiltered temporal patterns, the new ARFs, and a non-linear runoff-routing model, the new IL/CL values were shown to produce peak flows that are generally consistent with the results of flood frequency analysis. However, the verification of design losses is dependent upon the choice of all of the key inputs in the modelling process; different assumptions about any of them could affect the conclusions about the others.

ACKNOWLEDGMENTS

Much of the creation and calibration of the RORB models was undertaken by Upula Maheepala, Ephraim Goldberg and Tam Hoang who worked as assistants in the CRC for Catchment Hydrology. Their efforts are gratefully acknowledged.

Hydrometric data was provided by the Bureau of Meteorology, the (former) Rural Water Corporation and ACT Electricity and Water.

Valuable guidance and suggestions were provided by the project reference panel which included Tom McMahon, Rory Nathan, Jim Elliott, Leon Soste and Sri Srikanthan.

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1. INTRODUCTION

This reports presents an evaluation of the impact of the proposed new design losses, and areal reduction factors for design rainfalls, on the prediction of design floods. These new design values are two of the outcomes from research performed in the Flood Hydrology Program of the CRC for Catchment Hydrology.

An estimate of a design flood hydrograph is required for a variety of engineering works. The procedure for estimating a design flood hydrograph with specified annual exceedance probability (AEP) for a catchment starts with a point design rainfall of the desired AEP. However, the actual probability of the calculated design flood peak will depend upon the choice of the critical storm duration, areal reduction factor, storm temporal pattern, *design losses*, runoff-routing model, model parameters and the baseflow.

Australian Rainfall and Runoff (IEAust., 1987 - hereafter referred to as AR&R) recommends design procedures and values for each of these key inputs. In that document, least guidance is available on appropriate values for design losses; this constitutes one of the greatest weaknesses in Australian flood design (Pilgrim and Robinson, 1988).

A complementary report by Hill et al. (1996) details the research done, using an empirical analysis of data, to re-derive losses suitable for design flood estimation. New areal reduction factors (Siriwardena and Weinmann, 1996) have been derived as part of CRC Project D3.

In this verification stage, the guidelines for design flood estimation in AR&R were followed to reproduce best practice. The purpose of this phase of the project is to provide an objective evaluation of the new input data for the design process.

This report of the work is structured as follows; the selection of the catchments is briefly dealt with in Chapter 2, and flood frequency analysis of the data is described in Chapter 3. This flood frequency analysis is used to test the rainfall based design flood estimates calculated in the subsequent chapters

The derivation and calibration of the RORB runoff-routing models is discussed in Chapter 4. New models were generated for all catchments to ensure that the models were consistent and that the quality of the models could be controlled.

Chapter 5 details the design guidelines in AR&R and illustrates the limitations of the current recommendations. The following chapter outlines the effects of some possible improvements by the use of new areal reduction factors and filtered temporal patterns.

Chapter 7 calculates design flows using the new design losses derived in Hill et al. (1996). Some conclusions are then drawn and recommendations made as to the suitability of the new losses for design flood estimation (Chapter 8).

2. DATA USED FOR THIS STUDY

The primary selection criterion used for the catchments was the existence of concurrent pluviograph and streamflow data so that the RORB models could be calibrated (Chapter 4). The catchments were selected from Hill (1994) with one additional catchment selected from the ACT. The second selection criterion was a long record of instantaneous peak flows for the flood frequency analysis (Chapter 3).

The 11 selected catchments are listed in Table 2-1; eight of them were also used in the empirical analysis of data undertaken in Hill et al. (1996). The catchment areas vary from 32 to 332 km² and the mean annual rainfall ranges from 540 to 1880 mm. The location of the catchments (Figure 2-1) represents a geographic spread covering central Victoria.

Table 2-1 Summary of Selected Catchments

Catchment	Code	Area (km^2)	Gauging Station	Rainfall (mm)
Goodman Ck above Lerderderg Tunnel	GO	32	231219	800
Ford River @ Glenaire	FO	56	235229	1520
Orroral River @ Crossing	OR	90	410736	750
Aire River @ Wyelangta	AI	90	235219	1880
Moonee Ck @ Lima	MO	91	404208	1060
Wanalta Ck @ Wanalta	WN	108	405229	540
Tarwin R East Branch @ Dumbalk Nth	TE	127	227226	1140
Lerderderg River @ Sardine Ck	LE	153	231213	1080
Moe River @ Darnum	ME	214	226209	1050
Avon River @ Beazley's Bridge	AV	259	415224	565
Seven Cks @ Euroa Township	SE	332	405237	925

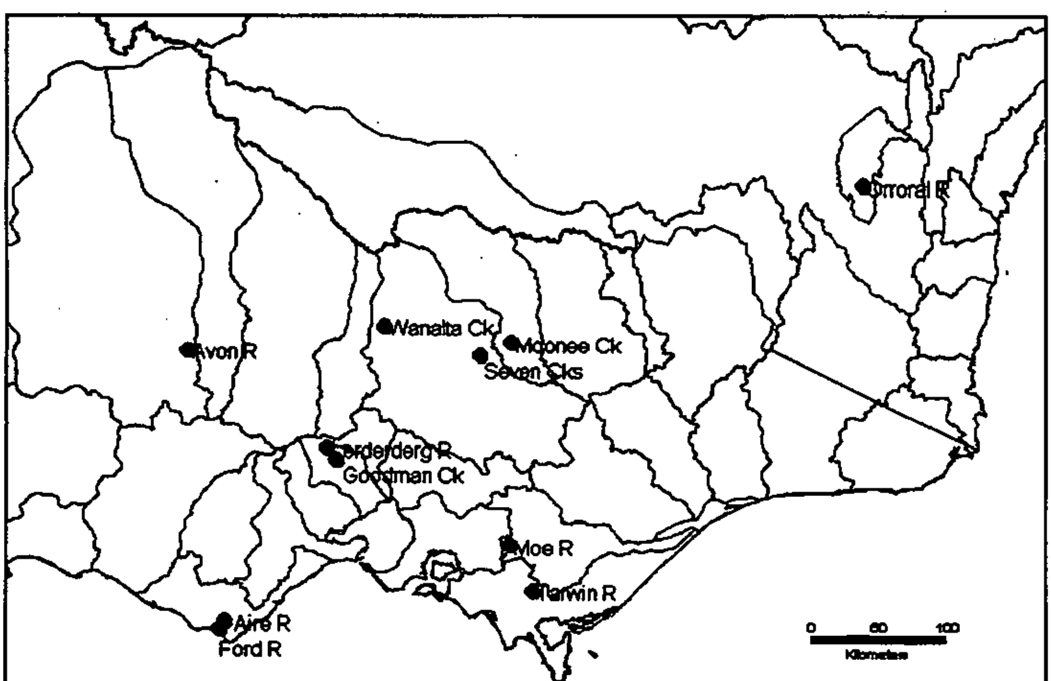

Figure 2-1 Location of Study Catchments

3. Flood Frequency Analysis

Flood frequency analysis (FFA) of the recorded peak flows was undertaken for each catchment. The flood quantiles from the analysis were needed to test rainfall based design peak flows in Chapters 5, 6 and 7.

A large number of different theoretical distributions have been used for flood frequency analysis. However, following the recommendations of AR&R, the log-Pearson III distribution was adopted for all catchments for consistency.

3.1 Annual Series

For each catchment, the annual series of instantaneous peak flows was constructed by selecting the largest peak flow in each year of record. For some catchments, additional information was available from Rural Water Commission of Victoria (1990) which summarises streamflow data for Victorian gauging stations to 1987. In these instances, information on the maximum peak flow in each month was available before the commencement of continuous streamflow data. These additional peak flows were used to extend the annual series.

Table 3-1 shows a summary of the available data. The annual series used for the flood frequency analysis are given in Appendix A.

Table 3-1 Summary of Availability of Instantaneous Peak Flows for Flood Frequency Analysis

Catchment	Code	Area	Streamflow Data		
		(km^2)	start	end	years
Goodman Ck	GO	32	1971	1995	25
Ford River	FO	56	1970	1986	17
Orroral River	OR	90	1968	1995	28
Aire River	AI	90	1968	1995	28
Moonee Ck	MO	91	1963	1995	33
Wanalta Ck	WN	108	1961	1995	35
Tarwin River	TE	127	1971	1995	25
Lerderderg River	LE	153	1960	1995	36
Moe River	ME	214	1961	1988	28
Avon River	AV	259	1965	1995	31
Seven Cks	SE	332	1964	1995	32

In some instances, the instantaneous peak flow was not recorded for a given month, but the maximum daily flow was available. Relationships developed between the instantaneous peak flow and the maximum daily flow for each catchment were used to infill the missing months.

The available streamflow data was extended, where possible, by relating the peak flows at the gauge of interest to other gauges either up or downstream. For Goodman Creek, (231219; 32 km^2) instantaneous peak flows are only available for 12 years (1971 to 1982). A linear relationship (r^2=0.97) was developed between the monthly instantaneous peak flow at 231219 and 231218 (located downstream; 39 km^2). This extended the annual series to 25 years (1971 to 1995).

For Avon River 415224 (259 km^2) there was only 17 years of peak flows available (1970 to 1986). A relationship (r^2=0.90) was developed between the peak flows at 415224 and 415220 (596 km^2) which has data from 1965 to 1995.

3.2 Low Flows

The occurrence of low flows in the annual series can have a significant effect on fitting a frequency distribution to an annual series of flood peaks. This is particularly relevant to south-eastern Australia where there have been a number of very dry years in the last few decades. In particular, in 1982 there was a severe drought over much of south-eastern Australia and for many catchments the 'peak flow' in 1982 is very small or even zero.

3.2.1 Omission of Low Flows

AR&R includes statistical tests for both high and low outliers (p216-7). The test for low outliers very rarely indicates the existence of any low outliers. Even if low flows are not shown to be statistical outliers, their occurrence can distort the fitting of a frequency distribution, particularly when the interest is on the upper tail of the distribution.

AR&R suggests that low flows be omitted and recommends a conditional probability adjustment. The adjusted exceedance probabilities are calculated by multiplying by the ratio of the number of years in the truncated series to the number of years in the total series.

Given that the low flows are not shown to be statistical outliers (by the criteria suggested in AR&R), judgement is required to decide whether there are low flows which need to be omitted and if so, how many. A large negative skew of the logarithms of the peak flows is often an indication that there are low flows which need to be omitted. A plot of the data and the fitted distribution is also valuable in ascertaining the existence of low flows.

An Excel spreadsheet was developed (CRCCHFFA.XLS) based upon the guidelines in AR&R for flood frequency analysis. The spreadsheet allows the inclusion of historical data and the omission of low flows. In this study, no historical data was considered, due to the time consuming nature of estimating historical flows.

The annual series were checked for low flows and any low flows were omitted using the probability adjustment recommended in AR&R. No low flows were omitted for Aire River and Moonee Creek. Between 1 and 3 low flows were omitted for each of the remaining 9 catchments.

3.2.2 Effect of Omitting Low Flows

The omission of low flows can have a major effect on the fitted frequency distribution, because of the change in the calculated skew and thus the shape parameter of the distribution. The effect of omitting the low flows on skew is shown in Appendix A and illustrated in Figure 3-1. For all but the Aire River the calculated skews for the full series are negative. Omitting the low flows increases skew to a more plausible value; generally in the range between -0.5 and +0.5. It is important to note the significance of this; the log Pearson III distribution has an upper bound if the skew is negative, and a large negative skew indicates a low value of the upper bound.

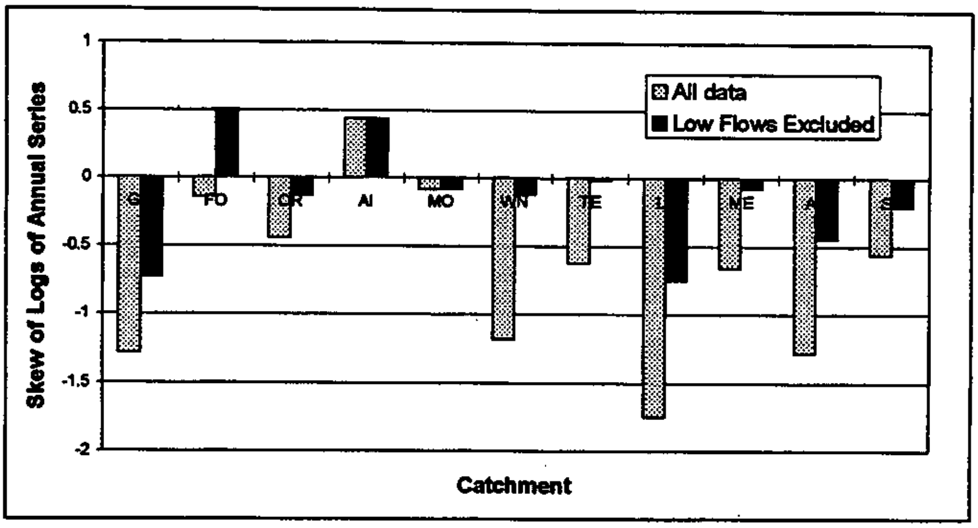

Figure 3-1 Effect of Omitting Low Flows on the Skew of the Logarithms of the Annual Series (Acronyms are given in Table 3-1)

The effect of omitting low flows on the calculated flood peaks is shown in Figure 3-2 (values are given in Appendix A). Data for the Aire and Moonee are not shown because no low flows were omitted for these catchments.

The omission of the low flows slightly reduces the estimate of the 1 in 10 AEP flow (with the exception of the Moe River for which there is no change). With the exception of Orroral, the omission of low flows increases the estimates of peak flows for the lower AEPs. The difference in the peak flows for an AEP of 1 in 100 varies from 10 to 50 percent. It is thus clear that estimates involving extrapolation beyond the range of the data depend to a considerable degree on subjective decisions made in the treatment of low flows.

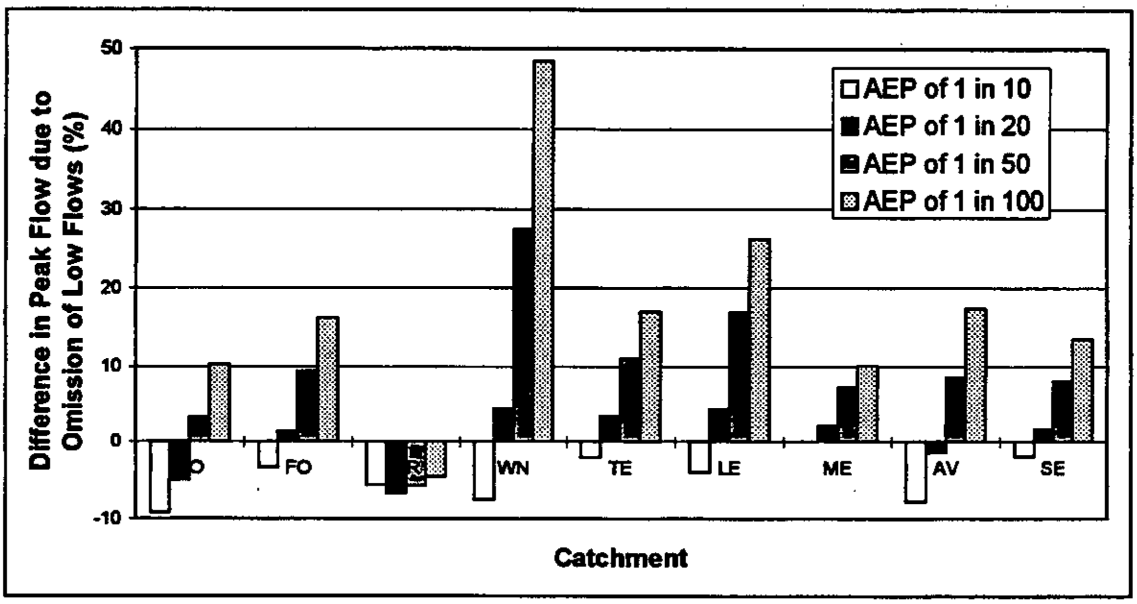

Figure 3-2 Effect of Omitting Low Flows on the Calculated Peak Flow

3.3 Results of Flood Frequency Analysis

The adopted plot of the fitted log Pearson III distribution for each catchment is shown in Appendix A. These plots incorporate the conditional probability adjustment for the omission of low flows. From these fitted distributions, flood peaks for AEPs of 1 in 10 and 1 in 50 were chosen for comparing the rainfall based design peak flow estimates from Chapters 5, 6 and 7. The results of the flood frequency analysis for AEPs of 1 in 10 and 1 in 50 are shown in Table 3-2.

Table 3-2 Results of Flood Frequency Analysis (low flows omitted)

Catchment	Area	No. Low	Peak Flow (m^3/s)					
			AEP of 1 in 10			AEP of 1 in 50		
	(km^2)	Flows	95%	Q_{10}	5%	95%	Q_{50}	5%
Goodman Ck	32	2	45	69	107	57	126	278
Ford River	56	1	35	57	94	47	116	284
Orroral River	90	3	33	50	76	52	99	191
Aire River	90	0	89	130	189	135	263	512
Moonee Ck	91	0	21	29	40	33	55	94
Wanalta Ck	108	3	32	49	76	56	111	220
Tarwin River	127	1	69	95	131	97	161	267
Lerderderg River	153	3	96	120	150	115	173	260
Moe River	214	1	37	44	52	45	59	77
Avon River	259	3	80	108	147	110	178	289
Seven Cks	332	1	170	257	388	288	549	1043

The confidence limits shown in Table 3-2 indicate a moderate degree of uncertainty in the 1 in 10 AEP peak flow and a larger error margin in the 1 in 50 AEP estimate.

The extremely low peak flows for the Moe River should be noted. Such low peak flows and high levels of baseflow, are common to other rivers in the Gippsland region (for example the La Trobe River). However, it seems likely that the 1 in 50 AEP event for the Moe River is underestimated by the frequency analysis of recorded peak flows.

4. RORB MODELLING

RORB models were created for each of the 11 catchments given in Table 2-1. Each model was then calibrated using the recorded concurrent streamflow and rainfall data. This chapter briefly outlines the formulation and calibration of the RORB models.

4.1 Model Formulation

4.1.1 RORB Catchment File

A RORB catchment file was created for each catchment following the guidelines in Laurenson and Mein (1995). The number of sub-areas for each catchment is shown in Table 4-1. The catchment sub-divisions and the RORB catchment data files used are included in Appendix B.

Table 4-1 Summary of RORB Catchment Files

Catchment	Area (km^2)	No. of Sub-Areas	d_{av} (km)
Goodman Ck	32	10	9.06
Ford River	56	14	7.88
Orroral River	90	11	11.2
Aire River	90	14	12.0
Moonee Ck	91	11	9.49
Wanalta Ck	108	14	8.72
Tarwin River	127	11	6.33
Lerderderg River	153	11	10.1
Moe River	214	20	10.1
Avon River	259	15	14.4
Seven Cks	332	14	23.8

4.1.2 Extraction of Baseflow

Prior to calibrating the RORB models, the surface runoff component of the total streamflow needs to be identified from the total recorded streamflow hydrograph. For this study, baseflow was separated using a recursive digital filter of the form of Equation 4.1. This filter has been widely used for yield studies (eg. Boughton, 1988; Nathan and McMahon, 1990) and has also been used to separate hydrographs for flood studies (eg. O'Loughlin et al., 1982; Hill et al., 1996). For this study, a filter factor of 0.925, 3 passes and a 1 hour time increment were used.

$$f_k = a.f_{k-1} + \frac{1+a}{2}\left(y_k - y_{k-1}\right) \tag{4.1}$$

where: f_k is the filtered quick response at the k^{th} sampling instant;
y_k is the total streamflow; and
a is the filter parameter (or factor).

4.2 Calibration

The RORB models were calibrated using the largest recorded flood events which had concurrent streamflow and pluviograph data; for each catchment, at least 6 events were selected for calibration. For some of these events there were data errors or data inconsistencies which affected the calibration and these events were discarded. The final number of events used for calibration is shown in Table 4-2.

RORB allows for spatially non-uniform rainfall data and daily rainfall data was used to supplement the pluviograph data to determine the spatial variation of each storm. The depth of rainfall for each sub-area was estimated from these data.

The RORB model was calibrated for both the initial loss-continuing loss (IL/CL) and the initial loss-proportional loss (IL/PL) models. The fitting strategy was that the initial loss was varied to make the rising limb of the calculated hydrograph match that of the recorded hydrograph. The k_c was then varied so that the peak flow was matched.

The calibrated parameters for each event are shown in Appendix B. Events with poor fits, or parameters which differed significantly from the others, were excluded and the mean parameters calculated for each catchment. Despite this, for some catchments there was still a spread of k_c values and this uncertainty will be reflected in the design floods estimated in Chapters 5, 6 and 7. The mean values are shown in Table 4-2; note that the runoff coefficient (RC) shown in Table 4-2 is equal to one minus the proportional loss.

Table 4-2 Summary of RORB Calibrations ($m = 0.8$)

Catchment	Area (km^2)	Baseflow (%)	No. of Events	IL/CL			IL/PL			Preferred Model
				IL	CL	k_c	IL	RC	k_c	
Goodman Ck	32	0.43	5	15	2.1	8.3	16	0.54	4.8	-
Ford River	56	6.0	8	29	2.7	13.4	48	0.40	7.9	-
Orroral River	90	5	6	8	4.3	17.6	20	0.22	14.5	IL/PL
Aire River	90	9	4	24	2.3	18.5	44	0.50	12.3	IL/PL
Moonee Ck	91	21	5	4	4.3	24.5	8	0.21	17.6	-
Wanalta Ck	108	4.8	7	14	2.2	17.2	14	0.45	13.7	-
Tarwin River	127	7.7	6	16	0.75	19.6	19	0.64	13.0	-
Lerderderg River	153	3.2	4	19	1.2	19.2	21	0.58	12.7	IL/PL
Moe River	214	20	6	3	2.1	20.2	11	0.33	16.3	IL/PL
Avon River	259	3.4	6	22	1.3	15.9	22	0.63	14.1	-
Seven Cks	332	5.9	5	22	1.5	17	23	0.62	14.9	-

As mentioned above, each event was calibrated for both IL/CL and IL/PL models. From a comparison of the calculated and recorded hydrographs it was evident that, for different events, different loss models sometimes resulted in different degrees of fits; the last column in Table 4-2 indicates if one model appeared to produce clearly better fits. For 7 of the catchments, there is no preferred loss model. For 4 of the catchments, the IL/PL model was superior over the range of events. There were no catchments for which IL/CL model consistently outperformed the IL/PL model.

It is important to note that the calibrated initial loss can be different for the IL/CL and IL/PL models. It would be reasonable to assume that the initial loss was the same for the two loss models, but the initial loss for the IL/CL model was consistently lower than for the IL/PL model. This is because the fitted initial loss depends on the pattern of the ongoing loss, which is different for the two loss models.

The calibrated k_c value for RORB also depends on the loss model for similar reasons as given above, and k_c for the IL/PL model is consistently lower than that for the IL/CL model. Figure 4-1 compares the k_c values derived for the two loss models and Equation 4.2 relates the k_c for the IL/CL model to the k_c for the IL/CL model found from linear regression.

$$k_c(IL/PL) = 0.79k_c(IL/CL) - 0.85 \qquad r^2=0.79 \qquad (4.2)$$

The different k_c for the different loss models is particularly important for this project, as loss parameters have been evaluated for both the IL/CL and IL/PL models. The appropriate k_c value must be used for each of the loss models.

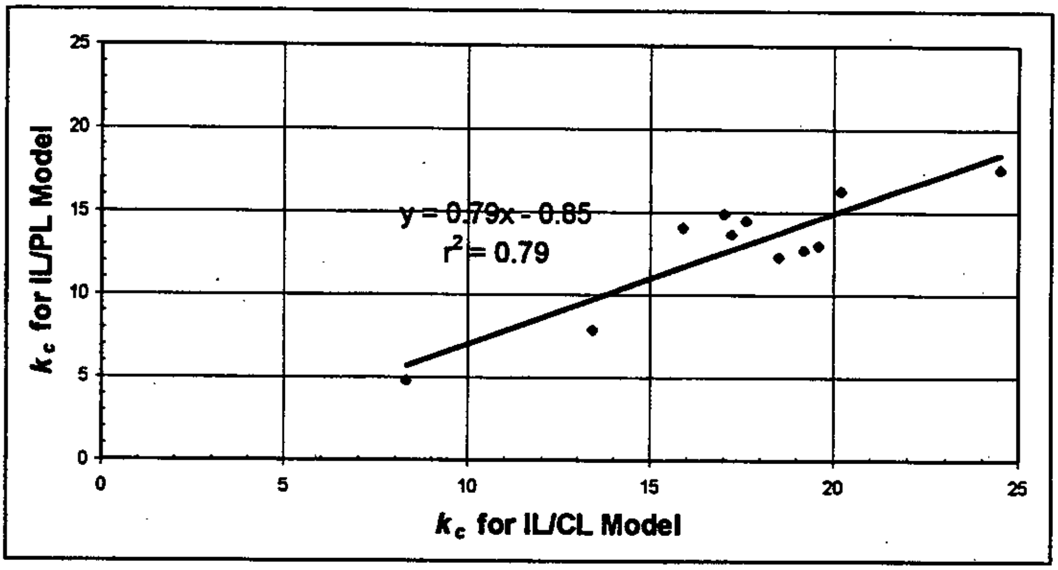

Figure 4-1 Comparison of k_c Values for Different Loss Models

5. Flood Estimates using Australian Rainfall and Runoff

Prior to testing the suitability of the new design losses derived in Hill et al. (1996), the existing loss parameters contained in AR&R are used to derive design flood estimates. These estimates are compared to those obtained from the flood frequency analysis undertaken in Chapter 3.

5.1 Choice of Parameters from Australian Rainfall and Runoff

A summary of the recommendations of design losses contained in AR&R is shown in Table 5-1 (no recommendation is made for initial loss for Tasmania). The limitations of these losses are discussed by Walsh et al. (1991), Hill and Mein (1996) and Hill et al. (1996).

Table 5-1 AR&R Recommended Design Losses

Location	Median Values of Parameters
ACT	Initial loss zero Continuing loss 1.0-3.6 mm/h (depending on ARI)
New South Wales East of the western slopes	Initial loss 10 to 35 mm, varying with catchment size and mean annual rainfall. Continuing loss 2.5 mm/h
Arid Zone, mean annual rainfall ≤ 300 mm	Initial loss 15 mm Continuing loss 4 mm/h
Victoria South and east of the Great Dividing Range	Continuing loss 2.5 mm/h Initial loss 25-35 mm Initial loss 15-20 mm
North and west of the Great Dividing Range	Probably as for similar areas of NSW

From Table 5-1 it is evident that AR&R recommend a range of values, with little guidance as to how the losses vary with catchment characteristics. There is no information on suitable values of proportional loss for design. For this study, an initial loss of 20 mm and a continuing loss of 2.5 mm/h were adopted for all catchments as being representative of the losses recommended in AR&R.

The information on design rainfall depths (IFD data), temporal patterns and areal reduction factors were obtained from AR&R and used without modification. The losses were applied to the design storms and the resulting rainfall excess routed through the calibrated RORB models using the parameters shown in Table 4-2. Design storms, of durations of 1 to 72 hours, were routed through the RORB model; the critical duration being estimated as the duration which gave the largest peak flow.

Following AR&R, the design surface runoff was then converted to a design total flow by adding an estimate of the baseflow to the surface runoff. The magnitude of this baseflow (as a percentage of the peak surface runoff) is shown in Table 4-2 and was calculated as the average of the events used in calibrating the RORB models in Chapter 4. The resulting peak flow was taken as the design peak flow for the given annual exceedance probability, and compared to that obtained from the flood frequency analysis undertaken in Chapter 3.

5.2 Results using Currently Recommended Parameters

5.2.1 Peak Flows

Design peak flows were derived as outlined above for AEPs of 1 in 10 and 1 in 50 and are documented in Appendix C. In Figure 5-1 the differences between the peak flows obtained using the AR&R methodology and flood frequency analysis (Chapter 3) are shown.

The design peak flow for Moe River is grossly over-estimated (by over 300% for an AEP of 1 in 10 and over 500% for an AEP of 1 in 50). The results for Moe River are therefore not shown in Figure 5-1 because they distort the figure.

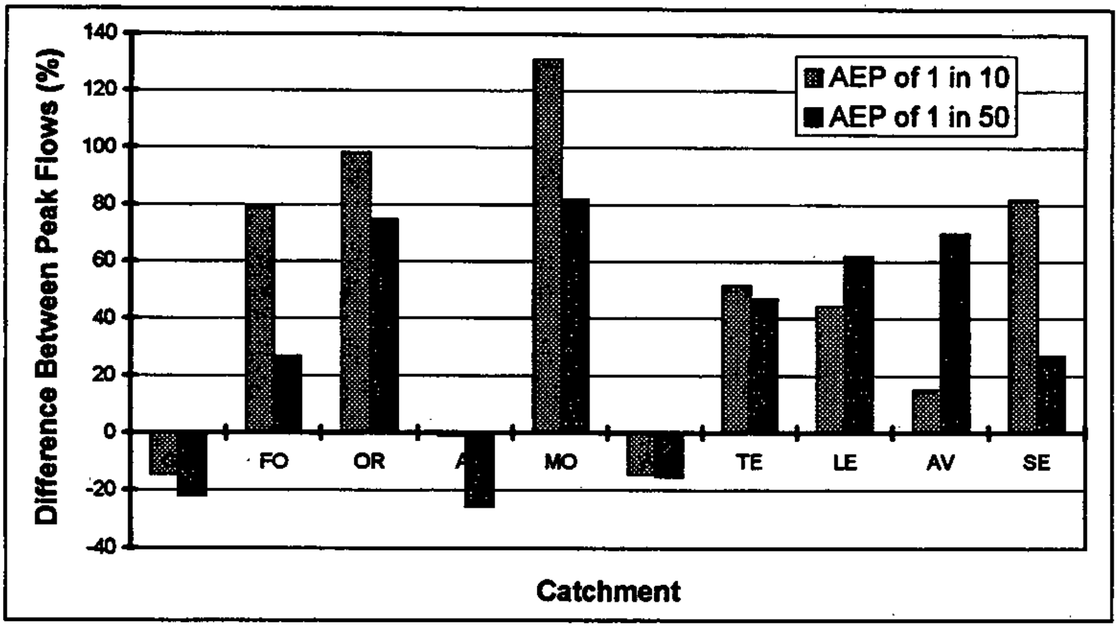

Figure 5-1 Difference Between Peak Flow obtained using AR&R and a Flood Frequency Analysis

It is clear from Figure 5-1 that the use of the AR&R design values results in over-estimation of the peak flows for 8 of the 11 catchments. Excluding the results for Moe River, the average over-prediction is 47 percent for an AEP of 1 in 10 and 32 percent for an AEP of 1 in 50.

5.2.2 Critical Durations

It would be reasonable to assume that the critical duration should increase with catchment area, on the principle that this parameter is related to catchment time of concentration. It would also be expected that d_{av} increases with catchment area, but there is almost no trend (Table 4-1).

The critical durations obtained for AEPs of 1 in 10 and 1 in 50 are shown in Figure 5-2. It is clear from this figure, that there is no relationship between the critical duration and the catchment area, a similar result to that reported by Walsh et al. (1992) and Hill and Mein (1996). With the exception of Seven Creeks, the critical duration for an AEP of 1 in 50 was equal to, or less than, the critical duration for an AEP of 1 in 10; this is due to losses being a smaller proportion of the rainfall hyetograph for the larger design storms.

For many catchments, the critical durations are larger than expected and reasons for this are discussed in Section 6.2.

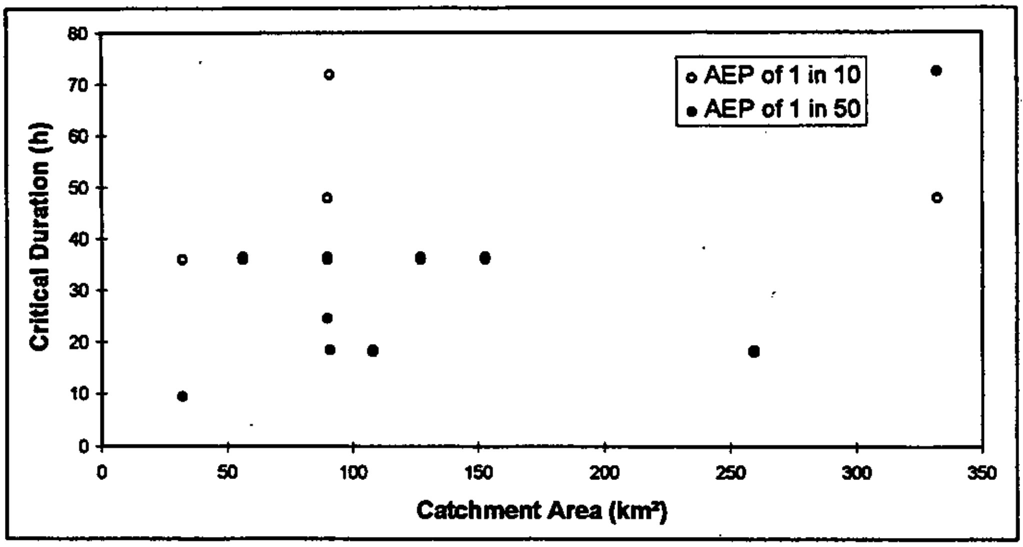

Figure 5-2 Critical Durations Using AR&R Methodology

5.3 Summary

The application of the design losses recommended in AR&R, with the design rainfall information contained in that document, generally results in the overestimation of design peak flows and produces critical durations which are not related to the catchment area. Authors such as Walsh et al. (1992) and Hill and Mein (1996) have reported similar findings.

The results represent application on ungauged catchments, where no opportunity exists for calibration of design losses against the results from flood frequency analysis. There is considerable subjectivity in the selection of design loss values from the range presented in Table 5-1 which translates into substantial uncertainty in the design flood estimates.

6. MODIFICATIONS TO THE CURRENT METHODOLOGY FOR DESIGN FLOOD ESTIMATION

This chapter outlines two modifications to the AR&R methodology for rainfall-based design flood estimation; the first is the use of new areal reduction factors which have been derived from Australian data and the second is the filtering of the design temporal patterns. The purpose is to overcome some of the inaccuracies found when the AR&R methodology was applied in Chapter 5.

6.1 Areal Reduction Factors

The areal reduction factor (ARF) converts a point rainfall intensity to the areal average rainfall intensity over a catchment of a given area. It takes into account that larger catchments are less likely than small catchments to have high intensity rainfall over the whole of the catchment area.

6.1.1 New ARFs for Victoria

Because of a lack of work done on Australian data, the ARFs contained in AR&R are based upon studies done in Chicago and Arizona in the United States. Some recent work using Australian data has indicated that the ARF values contained in AR&R may be too high. A major study was therefore initiated as part of CRC for Catchment Hydrology Project D3 to derive new ARFs for Victoria (Siriwardena and Weinmann, 1996).

Using the Victorian daily rainfall database, ARFs were derived for areas from 1 to 10,000 km², durations from 18 to 120 hours and AEPs from 1 in 2 to 1 in 2,000. The ARFs derived in the study were 5 to 8 percent lower than those contained in AR&R.

Because of the significant difference in the new ARFs for durations of more than 18 hours, it is likely that ARFs in AR&R for shorter durations are also too high. Until a detailed analysis can produce ARFs for short durations, Siriwardena and Weinmann (1996) recommend (in Appendix C) some interim short duration ARFs which are consistent with the new ARFs. The short duration ARFs are related to those used in the U.K. Flood Studies Report (NERC, 1975), and an equation was developed to merge the short duration ARFs with the new long duration ARFs at a duration of 18 hours.

Siriwardena and Weinmann (1996) recommend that the new ARFs are applicable for Victoria and for regions with 'similar hydrometeorological characteristics'. They derive expressions for the ARF for an average recurrence interval (ARI) of 2 years and an adjustment for calculating ARFs for other ARIs. The ARFs for an ARI of 2 years are shown in Appendix D for areas from 1 to 10,000 km² and durations of 1 to 48 hours.

Table 6-1 compares the new ARFs for an ARI of 2 years with those in AR&R. The new ARFs are approximately 5 percent lower for a duration of 24 hours and approximately 10 percent lower for a duration of 2 hours.

Table 6-1 Comparison of New ARFs with values from AR&R for an ARI of 2 years

Duration	% Reduction in ARF for Area (km²)			
(h)	50	100	500	1000
2	11.5	10.9	9.9	9.2
12	8.2	9.3	9.8	10.1
24	5.1	5.2	6.4	6.5

6.1.2 Effect of using New ARFs

The effect of using the new ARFs was tested by repeating the design flood estimation undertaken in Chapter 5 (with the same losses) but using the new ARFs. The resultant peak flows are shown in Appendix D for AEPs of 1 in 10 and 1 in 50. The percentage reduction in peak flows for each catchment is shown in Figure 6-1. The average reduction is 6 percent for an AEP of 1 in 10 and 9 percent for an AEP of 1 in 50 (although the reduction can be as high as 18 percent for some specific catchments); there was no reduction for Goodman Creek and Ford River for an AEP of 1 in 10.

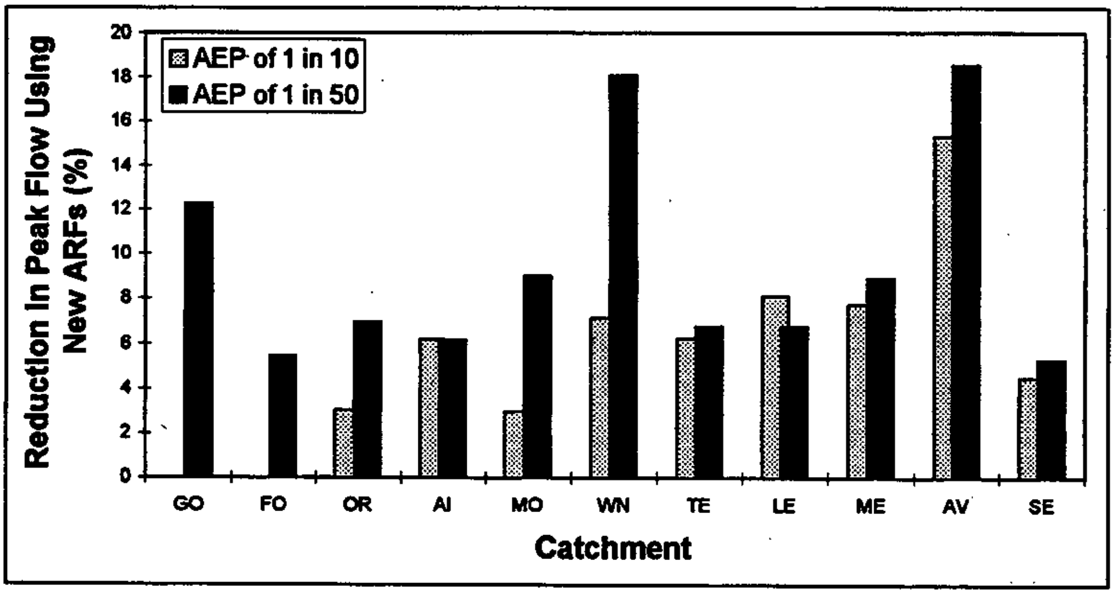

Figure 6-1 Effect of using New ARFs on the Design Flood Peak

6.2 Modified Temporal Patterns

In Section 5.2 long critical durations which were not related to the catchment area were noted when applying the AR&R methodology, and other studies such as Walsh et al. (1992) and Hill and Mein (1996) have also reported similar findings. Although 'reasonable' critical durations are not absolutely necessary for design flood estimation, they are certainly desirable.

According to Hill and Mein (1996), the excessively long critical durations are caused by:

- the occurrence of sub-intervals within the temporal patterns which have ARIs which exceed the ARI of the complete patterns;
- the use of a constant design initial loss for all durations; and
- the time increment used for the longer duration temporal patterns is too long, and for the smaller catchments, it approaches the response time of the catchment.

The treatment of temporal patterns is discussed in Section 6.2.1, the need for an initial loss which increases with duration is outlined in Hill et al. (1996), and the results of using a varying initial loss are discussed in Chapter 7 of this report.

6.2.1 Filtering Temporal Patterns

One method of reducing the occurrence of long critical durations is to 'filter' the temporal patterns to ensure that no sub-interval rainfall has an ARI which is greater than the ARI of the complete pattern. It should be noted here that the design temporal patterns presented in AR&R have *already been filtered* to ensure that no sub-interval has an ARI which is more than 10 percent (up to 20 percent for a very small number of patterns) greater than the ARI of the complete pattern; ie. the filtering was only a partial one. The recommendation in AR&R (p48) is for users to adopt further filtering if anomalies occur.

Temporal patterns can be filtered manually, or with available computer packages (Laurenson and Mein, 1995; Hydro Expert Software, 1993) in which incremental rainfall totals in the sub-interval are reduced by an amount proportional to their values and the excess rainfall is distributed equally over the remaining time increments.

For this study, the temporal patterns were filtered using the RORB for Windows Interface (Laurenson and Mein, 1995). The effect of filtering a 36 hour, 10 year ARI temporal pattern is shown in Figure 6-2 for Tarwin River and in Figure 6-3 for Wanalta Creek.

It can be seen that the temporal pattern for Tarwin River (from Zone 1 - coastal region) has its peak intensity in the middle of the pattern. The filtering of the temporal pattern has reduced the peak by approximately 30 percent and redistributed the rainfall over the remaining time increments.

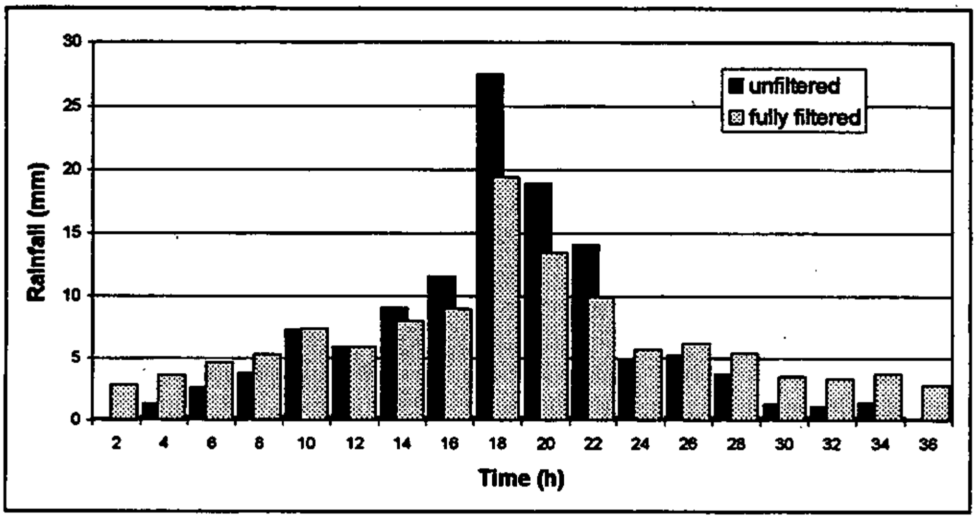

Figure 6-2 Fully Filtered Temporal Pattern for Tarwin River, 10 year ARI, 36 hour

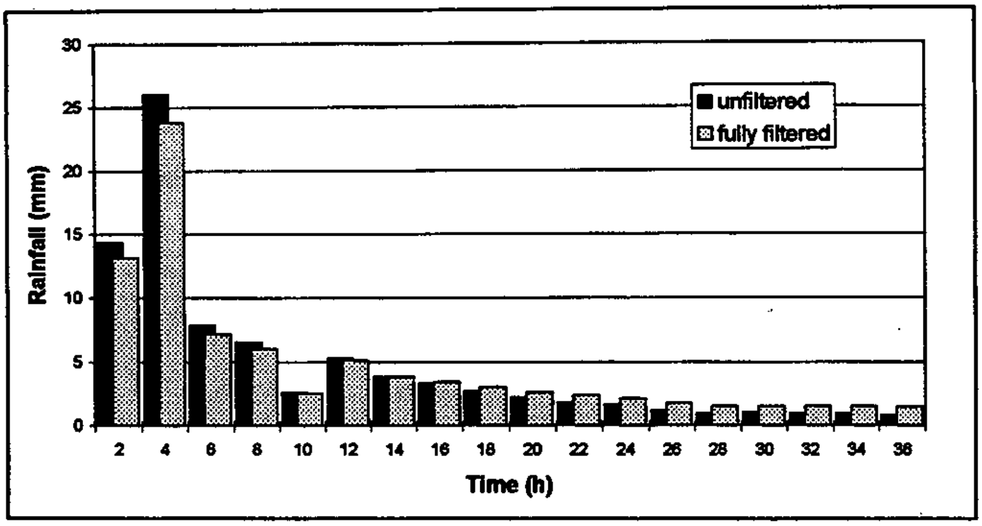

Figure 6-3 Fully Filtered Temporal Pattern for Wanalta Creek, 10 year ARI, 36 hour

In Figure 6-3 the temporal pattern for Wanalta Creek (from Zone 2 - inland region) has its peak intensity in the second time increment. The filtering of the temporal pattern only reduces the peak by approximately 8 percent; ie. the effect of filtering is not a consistent one for different durations and zones. The effect is most pronounced for durations greater than 6 to 12 hours.

It is important to consider the interaction of design losses with the temporal patterns. An initial loss will have a larger effect on an early peak temporal pattern (eg. Wanalta Creek in Zone 2) than for a temporal pattern which has a peak in the middle portion of the pattern (eg. Tarwin River in Zone 1).

6.2.2 Effect of Using Filtered Temporal Patterns

The effect of filtering on the temporal pattern has been demonstrated in the previous section. However, the most important result of filtering is on the calculated peak flows. The effect of filtering is shown in Figure 6-4 for Tarwin River and in Figure 6-5 for Wanalta Creek for a 1 in 10 AEP flood estimate.

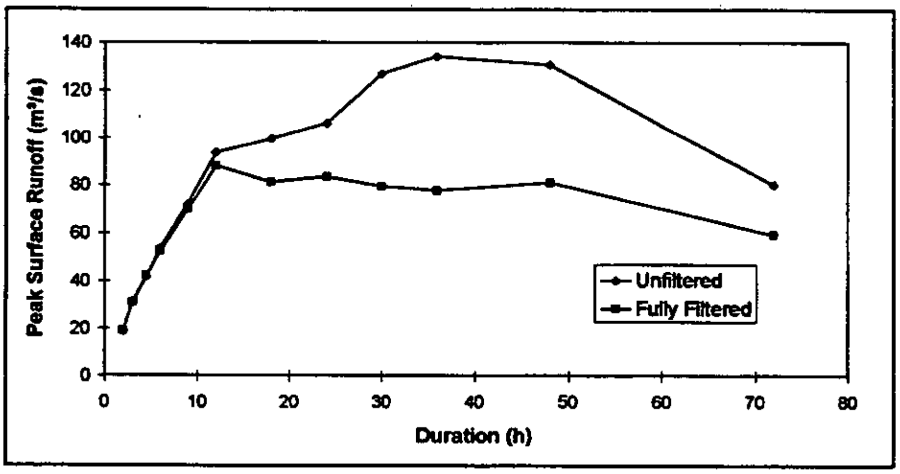

Figure 6-4 Effect of Filtering Temporal Patterns on Calculated Flows for Tarwin River (1 in 10 AEP)

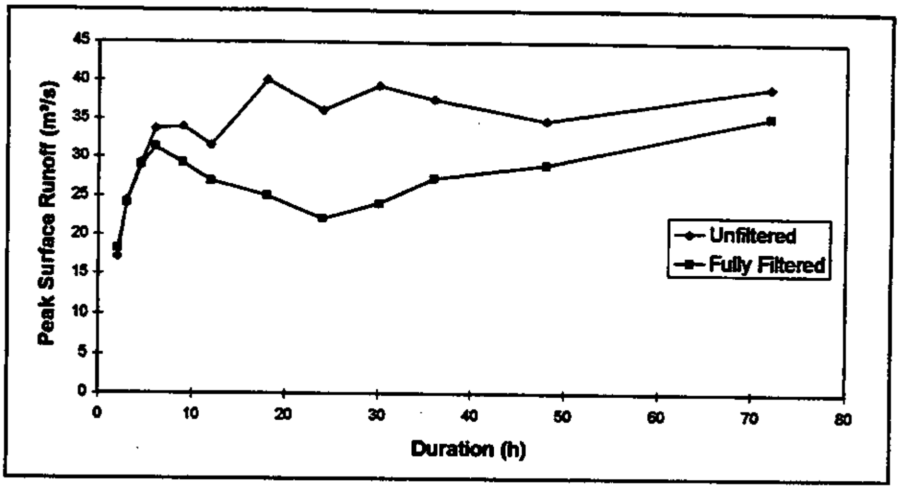

Figure 6-5 Effect of Filtering Temporal Patterns on Calculated Flows for Wanalta Creek (1 in 10 AEP)

For the Tarwin River, filtering reduces the critical duration from 36 to 12 hours. The curve for Wanalta Creek shown in Figure 6-5 is not as smooth and the filtering actually increases the critical duration from 18 to 72 hours.

The effect of filtering on the calculated critical durations for all of the catchments is shown in Figure 6-6 and Table 6-2. Filtering results in a reduction in the critical duration for 7 of the catchments, although for 1 catchment there is no change and for 3 catchments the filtering actually increases the critical duration.

Although filtering of temporal patterns produces more consistent curves for most catchments, it does not remove the occurrence of long critical durations. This is because there are still sub-intervals with ARIs equal (or nearly equal) to that of the complete pattern. The temporal patterns which contain these intense sub-intervals will still be critical because of the rainfall prior to the sub-interval.

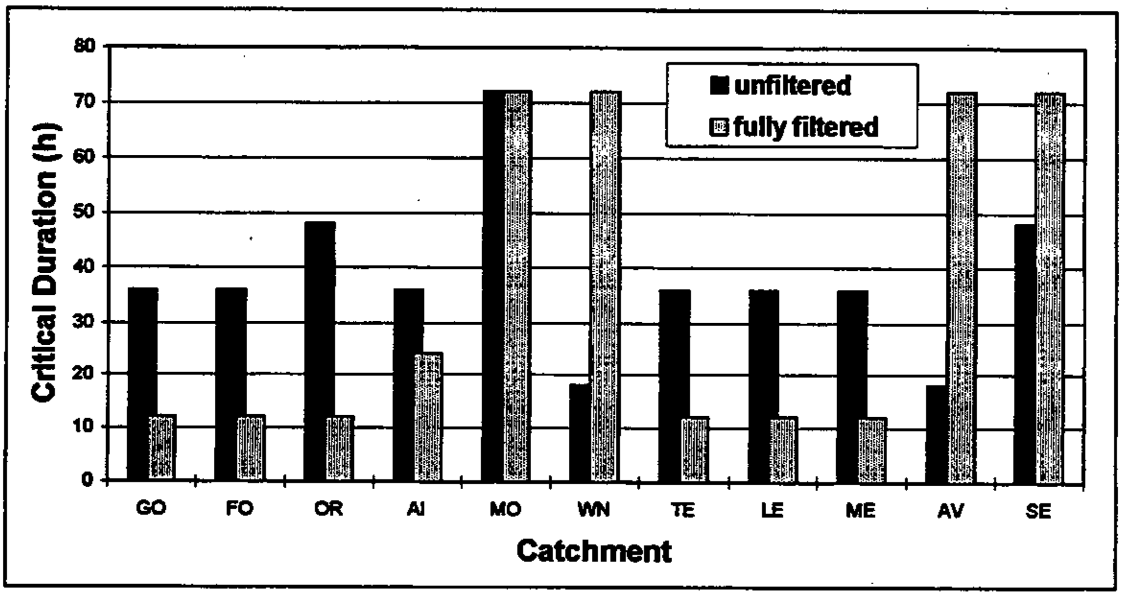

Figure 6-6 Effect of Filtering Temporal Patterns on the Critical Duration

Although, for most catchments, the critical durations have been reduced, the critical durations calculated using the fully filtered temporal patterns still do not exhibit any relationship with the catchment area.

It is also important to note the effect of filtering on the calculated peak flow. Table 6-2 shows the 1 in 10 AEP peak flows calculated with AR&R losses and areal reduction factors, and with unfiltered and fully filtered temporal patterns. The reduction in the 1 in 10 AEP peak flow varies from 8.5 to 39 percent.

Table 6-2 Effect of Filtering Temporal Patterns on 1 in 10 AEP Design Flows

Catchment	Code	Zone	Area (km^2)	FFA (m^3/s)	Peak Flow (m^3/s) unfiltered	Peak Flow (m^3/s) fully filtered	Reduction (%)	Critical Duration (h) unfiltered	Critical Duration (h) fully filtered
Goodman Ck	GO	1	32	69	59	54	8.5	36	12
Ford River	FO	1	56	57	102	67	34	36	12
Orroral River	OR	2	90	50	99	66	33	48	12
Aire River	AI	1	90	130	129	79	39	36	24
Moonee Creek	MO	2	91	29	67	43	36	72	72
Wanalta	WN	2	108	49	42	37	12	18	72
Tarwin River	TE	1	127	95	144	95	34	36	12
Lerderderg	LE	1	153	120	173	137	21	36	12
Moe River	ME	1	214	44	194	146	24	36	12
Avon River	AV	2	259	108	124	102	18	18	72
Seven Cks	SE	2	332	257	468	284	39	48	72

Figure 6-7 shows the difference between the peak flows calculated using unfiltered and fully filtered temporal patterns with the estimates from flood frequency analysis for an AEP of 1 in 10. The peak flow for Moe River is overestimated by over 200 percent even with fully filtered temporal patterns and is therefore not shown in Figure 6-7 as it would distort the scale.

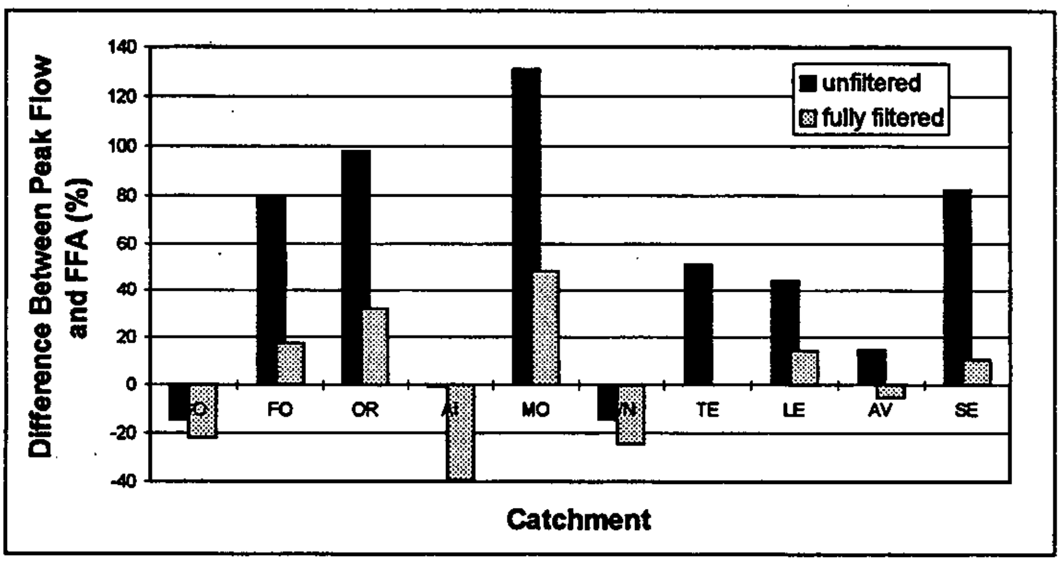

Figure 6-7 Difference Between Peak Flow obtained using AR&R and a Flood Frequency Analysis for an AEP of 1 in 10

From Figure 6-7 it is clear that, when applied with AR&R losses and areal reduction factors, the use of filtered rather than unfiltered temporal patterns results in design flows which are more in line with flood frequency analysis. The consistent bias (over-estimation) of design flows observed in Chapter 5 using unfiltered temporal patterns has been significantly reduced, although there is still up to 40 percent difference between the modelled flows and those from the flood frequency analysis.

7. Flood Estimates using New Design Losses

Hill et al. (1996) derived new losses for south-eastern Australia from the empirical analysis of data for 22 catchments. Loss values were derived for both the IL/CL and IL/PL models. This chapter applies the new design losses in place of those used in Chapter 5. The design peak flows are then compared to those obtained from the flood frequency analysis undertaken in Chapter 3.

As discussed in Section 3.3, the recorded floods for the Moe River seem unusually low; all attempts at using a rainfall-based design estimate over-estimate the flood peak obtained from flood frequency analysis. For this reason, the Moe River has been excluded from further consideration in this chapter.

The new areal reduction factors derived by Siriwardena and Weinmann (1996) and discussed in Section 6.1 have been used in this chapter.

7.1 New Design Losses

The prediction equations 7.1 to 7.4 were derived by Hill et al. (1996) from the empirical analysis of data. The losses are recommended for use with unfiltered temporal patterns and the new ARFs for long duration with the interim recommendations for short durations (Siriwardena and Weinmann, 1996).

The storm initial loss should first be estimated along with the continuing or proportional loss and then the burst initial loss (IL_b) calculated for each duration using Equation 7.2 which accounts for the embedded nature of the bursts used to calculate the design rainfalls in AR&R.

Storm Initial Loss

The storm initial loss should be estimated using Equation 7.1.

$$IL_s = -25.8BFI + 33.8 \qquad r^2{=}0.55 \qquad SE{=}5.1 \tag{7.1}$$

where: *BFI* is the baseflow index

Burst Initial Loss

The burst initial loss should be calculated for each duration using Equation 7.2.

$$IL_b = IL_s\left\{1 - \frac{1}{1 + 142\dfrac{\sqrt{(duration)}}{MAR}}\right\} \tag{7.2}$$

where: IL_s is the storm initial loss estimated using Equation 7.1
MAR is the mean annual rainfall (mm)
duration is the design duration (hours)

Continuing Loss

The continuing loss should then be calculated using Equations 7.3.

$$CL = 7.97BFI + 0.00659PET - 6.00 \qquad r^2{=}0.60 \qquad SE{=}1.5 \qquad (7.3)$$

where: *BFI* is the baseflow index;
PET is the mean annual potential evapotranspiration (mm);

Proportional Loss

The proportional loss should be estimated using Equation 7.4.

$$PL = 0.621BFI - 0.000175MAR + 0.662 \qquad r^2{=}0.71 \qquad SE{=}0.063 \qquad (7.4)$$

where: *BFI* is the baseflow index;
MAR is the mean annual rainfall (mm).

Adjustment for Season

Hill et al. (1996) showed that the storm initial loss and continuing loss vary significantly depending on the season in which the event occurs. The regional prediction equations have been derived using losses which have been adjusted to account for this seasonal variation. If the distribution of events is considered to be uniform throughout the year, the given loss values can be used without correction. However, if the uneven distribution of events found in the sample is considered to be typical, then the storm initial loss should be increased by 8 percent and the continuing loss should be increased by 5 percent.

The baseflow index (BFI) is defined as the volume of baseflow divided by the total streamflow volume (Nathan and Weinmann, 1993).

The relevant catchment characteristics and the predicted losses for each catchment are summarised in Table 7-1. The storm initial loss and the continuing loss were adjusted as described above. The burst initial loss for different durations was calculated using Equation 7.2.

Table 7-1 Predicted Design Losses (Hill et al., 1996)

Catchment	Code	Area (km^2)	BFI	PET (mm)	MAR (mm)	IL_s (mm)	CL (mm/h)	PL	RC (1-PL)
Goodman Creek	GO	32	0.13	1080	800	33	2.3	0.60	0.40
Ford River	FO	56	0.58	1050	1520	20	5.8	0.75	0.25
Orroral River	OR	90	0.54	1410	750	22	8.0	0.86	0.14
Aire River	AI	90	0.58	1050	1880	20	5.8	0.69	0.31
Moonee Creek	MO	91	0.65	1125	1060	18	7.0	0.88	0.12
Wanalta Creek	WN	108	0.08	1175	540	34	2.5	0.62	0.38
Tarwin River	TE	127	0.39	1000	1140	26	3.9	0.71	0.29
Lerderderg River	LE	153	0.41	1100	1080	25	4.8	0.73	0.27
Moe River	ME	214	0.48	980	1050	23	4.5	0.78	0.22
Avon River	AV	259	0.09	1110	565	34	2.1	0.62	0.38
Seven Creeks	SE	332	0.47	1150	925	23	5.6	0.79	0.21

Rather than refer to IL/PL it is easier to consider the loss model as an initial loss-runoff coefficient model (IL/RC), where the runoff coefficient is equal to one minus the proportional loss. If it is assumed that the there is 100 percent runoff from the saturated portion of the catchment and zero runoff from the remainder of the catchment, the runoff coefficient is equal to the percentage of the catchment that is saturated.

7.2 Effect of Using New Initial Loss-Continuing Loss

The new IL/CL values shown in Table 7-1 were applied to the design rainfalls depths from AR&R (with the new ARFs outlined in Section 6.1) and the excess routed through the catchment to produce design flows for AEPs of 1 in 10 and 1 in 50.

The peak flows for an AEP of 1 in 10 are summarised in Appendix E, and in Figure 7-1 they are compared with the flows obtained from the flood frequency analysis undertaken in Chapter 3. It is clear from Figure 7-1 that the consistent overestimation of peak flows using the AR&R losses has been alleviated.

The peak flow for the Aire River was substantially underestimated (58% for unfiltered and 76% for fully filtered temporal patterns). For the remaining catchments, the peak flow is estimated to within approximately 25 percent using unfiltered temporal patterns. However, the use of fully filtered temporal patterns however, results in a consistent under-prediction of the peak flow.

The underestimation of the design flow for the Aire River could possibly be caused by the design rainfall intensity being too low, or the design losses being too high.

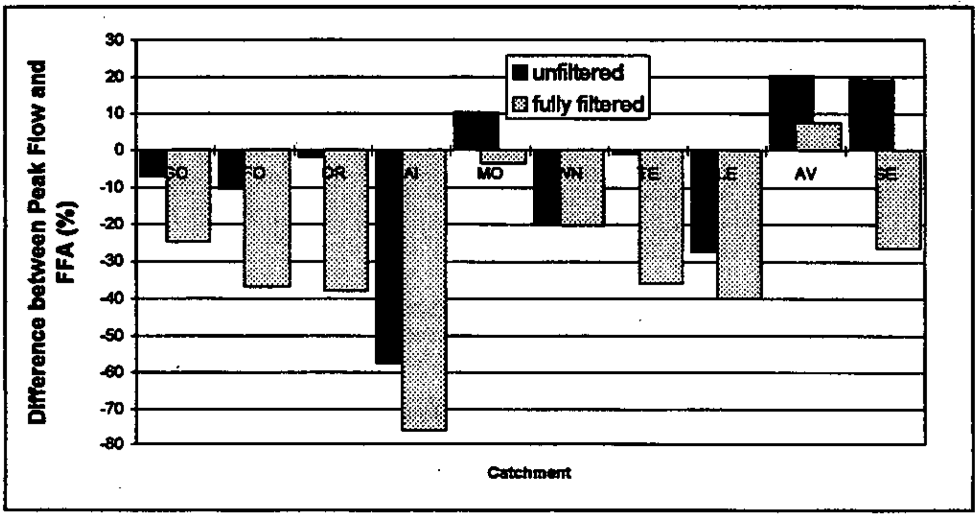

Figure 7-1 Difference between FFA and Peak Flow calculated using New IL/CL for an AEP of 1 in 10 (new ARFs)

The critical durations resulting from the use of the new IL/CL values with the new ARFs are shown in Table 7-2 for the case of unfiltered and fully filtered temporal patterns; the latter producing shorter critical durations. The combination of the new losses with fully filtered temporal patterns results in unrealistically short critical durations (ie 4.5 and 3 hours for 2 catchments larger than 200 km²) and is not recommended. It should be noted that the critical durations for neither the unfiltered or fully filtered case appear to be related to the catchment area.

Table 7-2 Critical Durations using New IL/CL for an AEP of 1 in 10 (new ARFs)

Catchment	Area (km^2)	Critical Duration (h) unfiltered	fully filtered
Goodman Ck	32	9	9
Ford River	56	36	12
Orroral River	90	24	3
Aire River	90	48	12
Moonee Ck	91	24	3
Wanalta	108	3	3
Tarwin River	127	48	12
Lerderderg	153	12	12
Avon River	259	18	4.5
Seven Cks	332	24	3

The relationship between peak surface runoff and duration is shown in Figure 7-2 and Figure 7-3 for Tarwin River and Wanalta Creek. For Tarwin River the large reduction resulting from the use of fully filtered temporal patterns is evident. For Wanalta Creek and unfiltered temporal patterns the irregular relationship between flow and duration is evident. The use of fully filtered temporal patterns for this catchment results in a very short critical duration (3 hours).

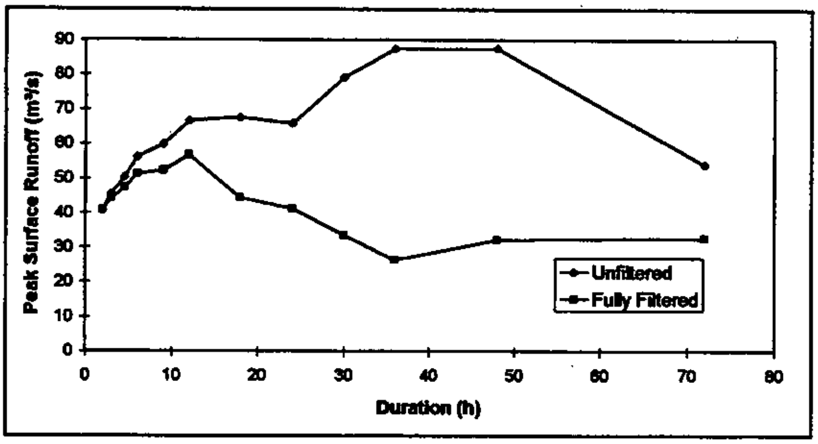

Figure 7-2 Effect of Filtered Temporal Patterns for Tarwin River (AEP of 1 in 10, new IL/CL)

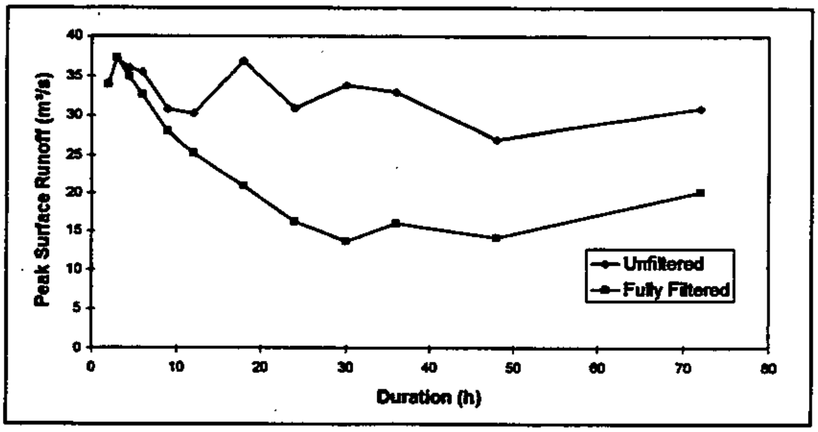

Figure 7-3 Effect of Filtered Temporal Patterns for Wanalta Creek (AEP of 1 in 10, new IL/CL)

The peak flows calculated for an AEP of 1 in 50, using the new ARFs and unfiltered temporal patterns, are shown in Table 7-3. Here, there does not appear to be any consistent bias in the flows when compared with those obtained from the flood frequency analysis, although the peak flows differ by up to 62 percent. As noted for an AEP of 1 in 10, the design flow for Aire River is underestimated. There does appear to be a general trend to underestimate the flow for smaller catchments, although a larger number of catchments would be necessary to verify this trend.

Table 7-3 Peak Flow calculated using New IL/CL for an AEP of 1 in 50 (new ARFs, unfiltered temporal patterns)

Catchment	Area (km^2)	FFA (m^3/s)	peak flow (m^3/s)	difference (%)	Critical Duration (h)
Goodman Ck	32	126	93	-26	9
Ford River	56	116	83	-28	36
Orroral River	90	99	85	-14	24
Aire River	90	263	101	-62	36
Moonee Creek	91	55	52	-5	4.5
Wanalta	108	111	85	-23	18
Tarwin River	127	161	173	7	36
Lerderderg	153	173	175	1	12
Avon River	259	178	289	62	18
Seven Cks	332	549	495	-10	48

7.3 Effect of Using New Initial Loss-Runoff Coefficient

The new IL/RC values shown in Table 7-1 were applied to the design rainfalls depths from AR&R (with the new ARFs outlined in Section 6.1) and the excess routed to produce design floods for AEPs of 1 in 10 and 1 in 50.

In Table 7-4 the peak flows for an AEP of 1 in 10 are compared to the flows obtained from the flood frequency analysis undertaken in Chapter 3. The peak flow is underestimated for every catchment by between 18 and 58 percent for unfiltered temporal patterns and by between 28 and 70 percent for fully filtered temporal patterns. The critical durations calculated using both unfiltered and fully filtered temporal patterns show no relationship with catchment area.

Table 7-4 Peak Flow calculated using New IL/RC for an AEP of 1 in 10

			unfiltered			fully filtered		
Catchment	Area (km^2)	FFA (m^3/s)	peak flow (m^3/s)	difference (%)	Crit. Dur. (h)	peak flow (m^3/s)	difference (%)	Crit. Dur (h).
Goodman Ck	32	69	44	-36	9	37	-46	9
Ford River	56	57	43	-25	36	34	-40	12
Orroral River	90	50	21	-58	72	15	-70	18
Aire River	90	130	69	-47	36	48	-63	12
Moonee Ck	91	29	15	-48	72	11	-62	72
Wanalta	108	49	35	-29	18	30	-39	9
Tarwin River	127	95	75	-21	36	54	-43	30
Lerderderg	153	120	81	-33	36	67	-44	12
Avon River	259	108	89	-18	18	78	-28	9
Seven Cks	332	257	134	-48	72	93	-64	9

If it is assumed that the there is 100 percent runoff from the saturated portion of the catchment and zero runoff from the remainder of the catchment, the runoff coefficient is equal to the percentage of the catchment that is saturated. It would be expected that the percentage of the catchment which is saturated would increase during an event and would also increase with storm severity.

The under-prediction of design flows using the IL/RC model is therefore not unexpected. Over half of the rainfall bursts used to derive the IL/PL parameters have an ARI of less than 2 years and only 4 percent have an ARI of more than 20 years (Hill et al, 1996).

Design flows were again estimated using the IL/RC model, but the RC was increased by a fixed proportion until there was no consistent bias in the estimated flows (across all catchments) when compared to those obtained from the flood frequency analysis. Figure 7-4 shows that the use of the IL/RC values results in consistently underestimated peak flows. By increasing the RC by 50 percent, the bias is removed, although there is still up to 36 percent difference between the calculated flows and those obtained from a flood frequency analysis.

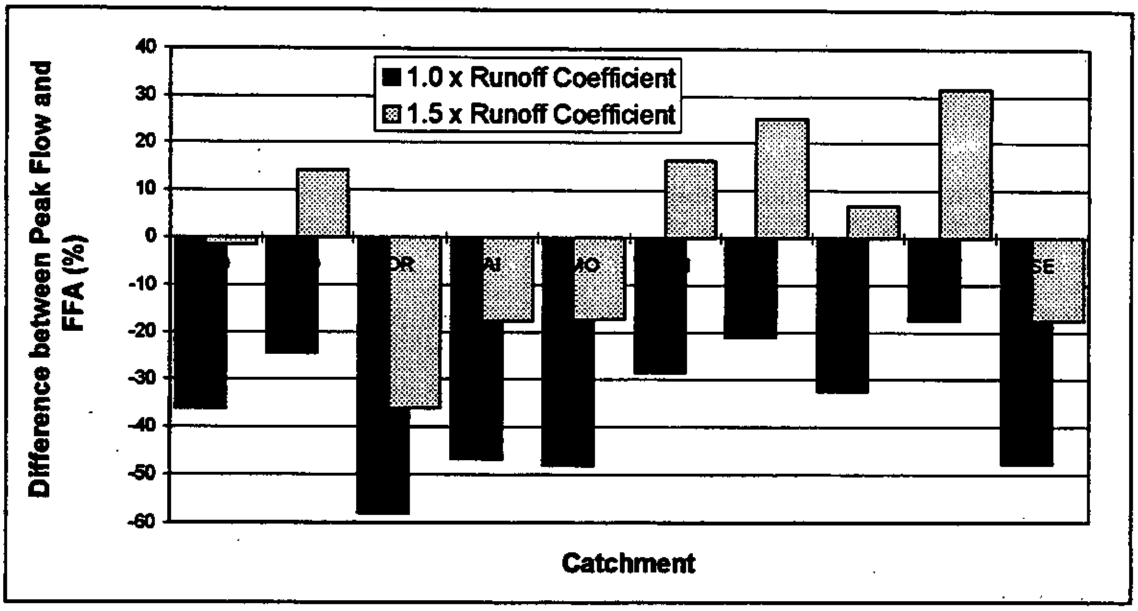

Figure 7-4 Difference between FFA and Peak Flow calculated using New IL/PL for an AEP of 1 in 10 (new ARFs, unfiltered temporal patterns)

In Figure 7-5 the peak flows obtained using the IL/RC values with the new ARFs and unfiltered temporal patterns for an AEP of 1 in 50 are compared to the flows obtained from the flood frequency analysis undertaken in Chapter 3. The peak flow is again underestimated for every catchment. The runoff coefficient was increased until the bias was removed. Figure 7-5 shows the peak flows resulting from a 50 and 80 percent increase in the runoff coefficient. The use of a runoff coefficient increased by 80 percent removes the bias. The peak flow for the Avon River is overestimated by 66 percent and for the remaining catchments, the peak flows are within 45 percent.

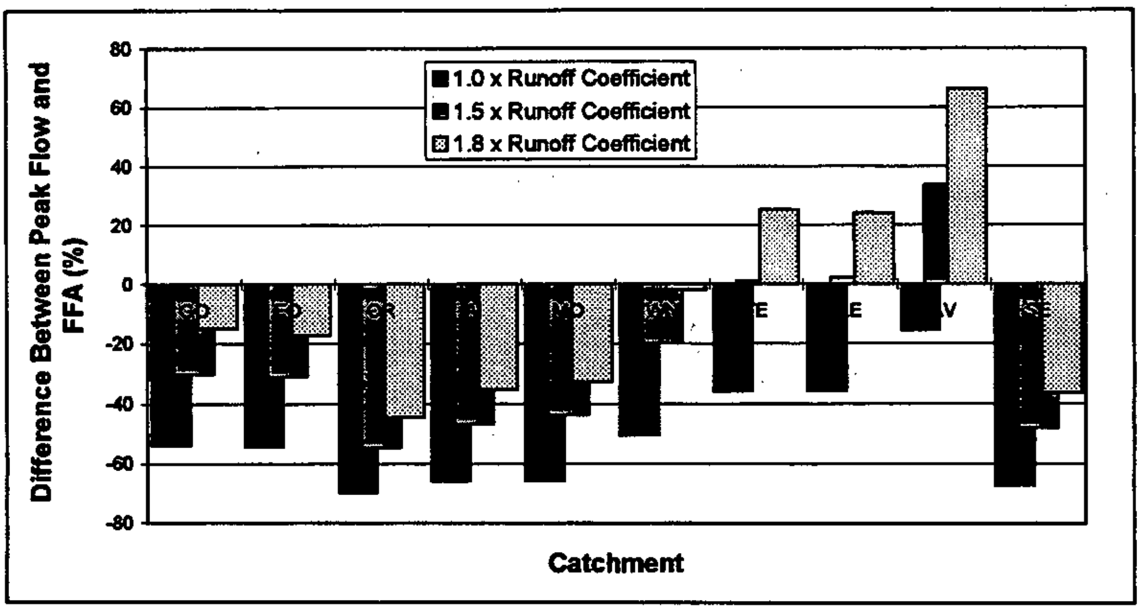

Figure 7-5 Difference between FFA and Peak Flow calculated using New IL/RC for an AEP of 1 in 50 (new ARFs, unfiltered temporal patterns)

8. CONCLUSION

8.1 Summary

This report describes the application of the existing AR&R guidelines for design flood estimation, and the testing of the new design losses derived by Hill et al. (1996) and the new areal reduction factors (ARFs) derived by Siriwardena and Weinmann (1996) based upon a comparison with results from a flood frequency analysis.

The design flood estimates from runoff-routing modelling are the result of a number of factors. This study has not fully explored the effect of different runoff-routing model parameters but has concentrated on the major factors involved in runoff production, ie. design point rainfall depths, areal reduction factors, temporal patterns and design losses.

A flood frequency analysis of the recorded flood peaks was undertaken for 11 catchments. RORB models were calibrated for each catchment for both the initial loss-continuing loss (IL/CL) and initial loss-proportional loss (IL/PL) models.

The currently recommended design parameters in AR&R were used to estimate design hydrographs for each catchment and the peaks compared with those from the flood frequency analysis. The effects of using the new areal reduction factors, filtered temporal patterns and the new design losses were examined in a similar fashion.

8.2 Conclusions

From the limited scope benchmark testing on 11 catchments, the following conclusions are drawn:

- The application of RORB with the currently recommended design parameters in AR&R consistently overestimates the peak flows. For an AEP of 1 in 10, the peak flow was overestimated by an average of 47 percent and for an AEP of 1 in 50, the peak flow was overestimated by an average of 32 percent.

- The application of the new IL/CL values with the new ARFs and unfiltered AR&R temporal patterns removes the bias encountered using the design values from AR&R. For an AEP of 1 in 10, 9 of the 11 catchments had peak flows that were within 25 percent of those from the flood frequency analysis (Figure 8-1). For an AEP of 1 in 50, 8 of the catchments had a difference in flows of less than 28 percent.

- The application of the new IL/PL values with the new ARFs and unfiltered temporal patterns produced peak flows which were consistently lower than those obtained from the flood frequency analysis. For an AEP of 1 in 10, the runoff coefficient (one minus the PL) had to be increased by 50 percent to remove the bias. For an AEP of 1 in 50, the runoff coefficient had to be increased by 80 percent. This is an impediment to the use of the IL/PL model for design.

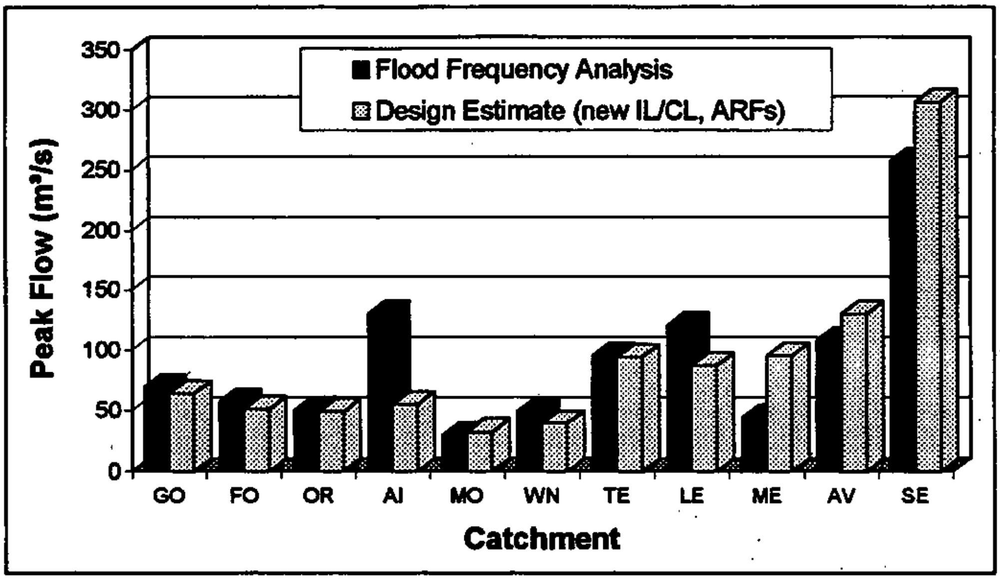

Figure 8-1 Design Peak Flows Calculated using new IL/CL and new ARFs for an AEP of 1 in 10

- The use of the AR&R parameters produced excessively long critical durations, unrelated to catchment area. The use of fully filtered temporal patterns with the same parameters led to reductions in the peak flow of between approximately 10 and 40 percent; critical durations were generally shorter than those obtained using unfiltered temporal patterns, but some inconsistencies remain.

- The use of the new ARFs by themselves resulted in an average reduction in the peak flow of 6 percent for an AEP of 1 in 10, and of 9 percent for an AEP of 1 in 50.

- Low flows should be omitted from the annual series when undertaking a flood frequency analysis. Although these low values may not be outliers in a statistical sense, they can have a large influence on the fitted distribution. When fitting a log Pearson III distribution, the omission of between 1 and 3 low flows made up to a 50 percent difference in the estimate of the 1 in 100 AEP flow.

- During the calibration of the RORB models, both the initial loss and k_c were dependent upon the loss model (IL/CL or IL/PL). The initial loss was higher and the k_c was lower for the IL/PL model. A relationship between the k_c values for the two loss models was developed.

8.3 Recommended Losses for Design Flood Estimation

The new design IL/CL values derived by Hill et al. (1996) are recommended for design flood estimation for south-eastern Australia on the basis that:

- they are based on a detailed study using methodology that is consistent with the derivation of design rainfalls;
- they incorporate plausible relationships with catchment and climatic characteristics, and rainfall duration;
- they produce satisfactory results when tested on 11 representative catchments in the region.

The losses are recommended for use with unfiltered temporal patterns and the new ARFs for long duration with the interim recommendations for short durations (Siriwardena and Weinmann, 1996). However, the verification of design losses is dependent upon the choice of all of the key inputs in the modelling process. Different assumptions about any of the key inputs, such as the filtering of temporal patterns, ARFs or runoff-routing model characteristics, could affect the conclusions.

8.4 Further Work

This report has outlined the verification of design losses on 11 catchments (10 Victorian and 1 in the ACT). There is a need to benchmark the methodology by applying it on more catchments in south-eastern Australia, particularly in southern NSW. There is also a need to apply the methodology on large catchments; the largest catchment in this study was only 332 km^2.

This study did not fully explore the effects of different runoff routing model characteristics on the calculated design floods, and their interaction with loss parameters. A broader scope benchmarking study should also address this issue.

Design losses should also be derived using the methodology outlined in Hill et al. (1996) for other regions of Australia. Further testing will then also be required.

There still remains up to a 50 percent misclosure between the rainfall based estimate and that from a flood frequency analysis at an AEP of 1 in 50. Further work is required on uncertainties in rainfall-based design flood estimates to investigate the reason for unexpectedly large discrepancies.

The relatively low design flood estimates from the flood frequency analysis for the Moe River identified in this study are common to other rivers in the Gippsland region (for example, the upper part of the La Trobe River). Further work is required to identify the catchment characteristics which cause this type of hydrologic behaviour.

It has been shown in this study that the runoff coefficient needs to increase substantially with storm severity. Further work is required to examine the variation of losses with AEP.

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Appendix A
Flood Frequency Analysis

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Table A.1 Annual Series used for Flood Frequency Analysis

Year	Annual Peak Flow (m^3/s)					
	Goodman	Ford	Orroral	Aire	Moonee	Wanalta
1960						
1961						0.9
1962						17.6
1963					3.2	24.9
1964					25.6	28.8
1965					19.7	7.2
1966					59.5	7.0
1967					2.3	2.6
1968			5.6	22.5	29.6	31.0
1969			58.2	22.9	7.0	12.3
1970		53.9	12.0	82.5	12.6	3.1
1971	32.8	18.5	28.9	49.4	20.9	3.6
1972	8.3	7.4	13.3	18.9	2.3	3.7
1973	33.0	18.1	8.9	84.1	22.8	158
1974	49.1	17.6	40.8	66.1	29.2	75.2
1975	14.7	15.6	48.4	38.0	26.7	16.4
1976	28.1	54.1	27.8	203	2.2	0.2*
1977	8.2	11.1	8.9	36.5	3.6	1.7
1978	51.5	86.3	20.9	241	8.1	7.6
1979	1.4	19.4	4.7	32.4	4.5	15.4
1980	0.2*	26.7	3.1	51.7	6.8	6.5
1981	3.1	11.7	8.6	17.7	23.5	13.0
1982	0*	3.3*	1.1*	21.7	1.8	0*
1983	69.9	33.2	18.2	136	9.9	18.9
1984	13.5	66.9	41.8	94.4	11.1	20.5
1985	40.6	12.4	5.3	49.1	4.6	11.2
1986	15.7	18.7	9.2	24.9	9.1	12.2
1987	69.0		1.5*	49.5	12.1	11.2
1988	10.4		38.7	29.8	13.9	10.2
1989	2.8		50.6	32.0	8.6	9.2
1990	78.2		12.8	88.7	7.6	14.3
1991	11.4		1.6*	71.8	4.5	21.6
1992	11.9		36.9	34.6	8.1	53.8
1993	41.1		13.0	70.5	24.1	55.1
1994	1.2		3.4	16.8	6.5	0.1*
1995	61.1		71.6	132	11.4	47.0

* Low flow excluded from flood frequency analysis

Table A.1 Annual Series used for Flood Frequency Analysis (cont.)

	Annual Peak Flow (m^3/s)				
Year	Tarwin	Lerderderg	Moe	Avon	Seven
1960		81.6			
1961		23.4	28.1		
1962		9.3	21.1		
1963		132	17.2		
1964		72.9	28.4		85.0
1965		75.6	16.1	30.9	45.9
1966		23.6	30.0	10.5	82.6
1967		0.7*	12.2	8.8	12.2
1968		80.6	21.1	59.4	376
1969		22.4	33.0	16.6	17.8
1970		74.2	25.2	10.7	44.8
1971	36.9	91.7	38.2	63.9	58.3
1972	13.0	11.4	19.0	2.9*	6.6
1973	24.2	76.7	17.5	147	99.5
1974	60.7	178	33.1	78.8	314
1975	50.6	54.5	46.8	92.5	364
1976	59.4	89.0	28.1	0*	10.5
1977	50.0	44.4	49.5	19.4	27.8
1978	139	79.4	28.4	48.5	46.0
1979	13.5	55.1	14.3	118	64.0
1980	107	12.6	55.3	57.9	53.4
1981	35.3	28.0	28.5	111	180
1982	4.3*	0.9*	7.0*	0*	2.3*
1983	58.8	84.4	40.3	88.9	84.6
1984	98.2	42.1	41.2	30.2	261
1985	16.4	107	36.0	27.5	31.2
1986	20.7	46.8	27.5	28.6	280
1987	17.8	76.3	16.2	54.9	49.8
1988	43.1	28.3	20.2	83.6	82.4
1989	48.3	31.1		23.9	35.7
1990	44.9	71.2		26.3	72.2
1991	32.2	11.7		37.0	47.1
1992	18.1	59.9		80.1	198
1993	101	122		102	285
1994	28.3	4.6*		8.3	9.7
1995	83.1	95.9		48.7	89.1

* Low flow excluded from flood frequency analysis

Table A.2 Effect of Excluding Outliers on the Skew of the Logarithms of the Annual Series

Catchment	Total No. of Years	No. Low Flows Excluded	Skew of the Logs All data	Skew of the Logs Low flows excluded
Goodman Ck	25	2	-1.283	-0.733
Ford R	17	1	-0.144	0.501
Orroral R	28	3	-0.445	-0.134
Aire River	28	0	0.440	0.440
Moonee Ck	33	0	-0.087	-0.087
Wanalta Ck	35	3	-1.186	-0.125
Tarwin R	25	1	-0.627	-0.022
Lerderderg R	36	3	-1.748	-0.751
Moe R	28	1	-0.657	-0.077
Avon R	31	3	-1.279	-0.452
Seven Cks	32	1	-0.564	-0.211

Table A.3 Effect of Excluding Outliers on the Calculated Peak Flow

Catchment	Peak Flow (m³/s)							
	AEP of 1 in 10		AEP of 1 in 20		AEP of 1 in 50		AEP of 1 in 100	
	all data	low flows omitted	all data	low flows omitted	all data	low flows omitted	all data	low flows omitted
Goodman Ck	76	69	99	94	122	126	136	150
Ford R	59	57	78	79	106	116	130	151
Orroral R	53	50	74	69	105	99	131	125
Aire R	130	130	179	179	263	263	343	343
Moonee Ck	29	29	39	39	55	55	70	70
Wanalta Ck	53	49	69	72	87	111	99	147
Tarwin R	97	95	118	122	145	161	165	193
Lerderderg R	125	120	138	144	148	173	152	192
Moe R	44	44	49	50	55	59	59	65
Avon R	117	108	140	138	164	178	178	209
Seven Cks	262	257	363	369	508	549	624	709

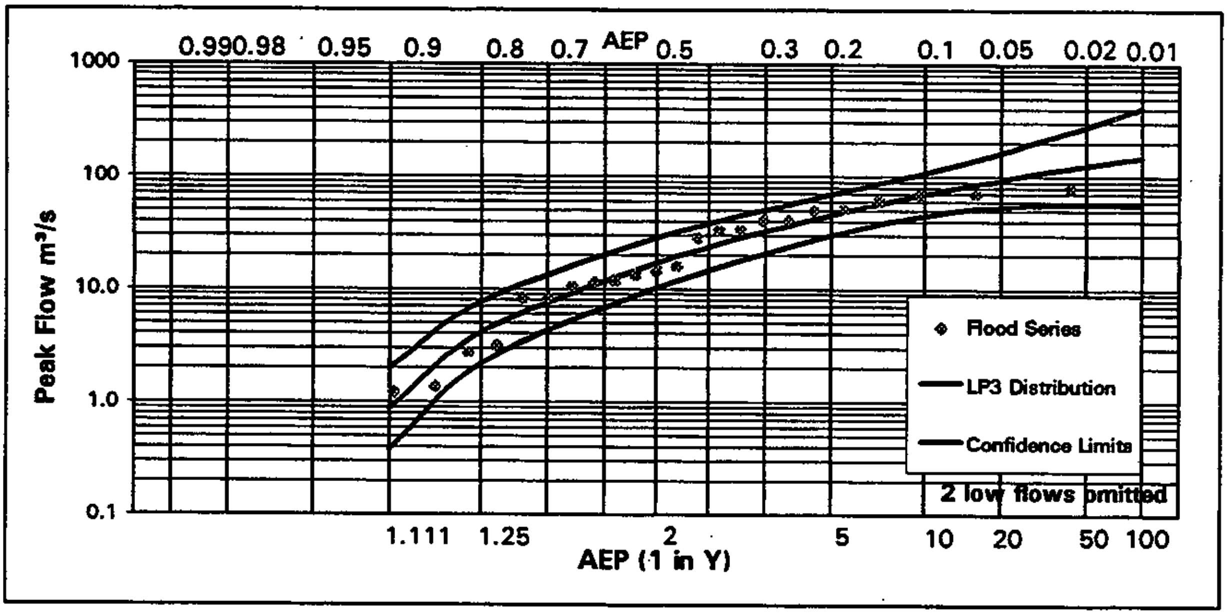

Figure A.1 Flood Frequency Curve for Goodman Creek

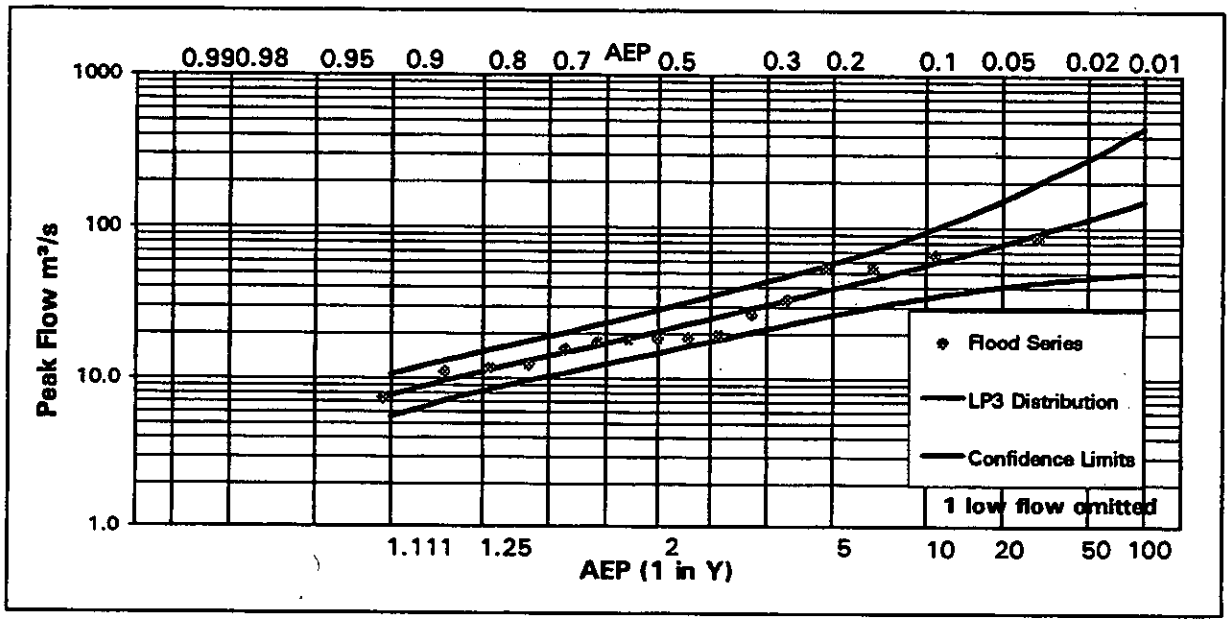

Figure A.2 Flood Frequency Curve for Ford River

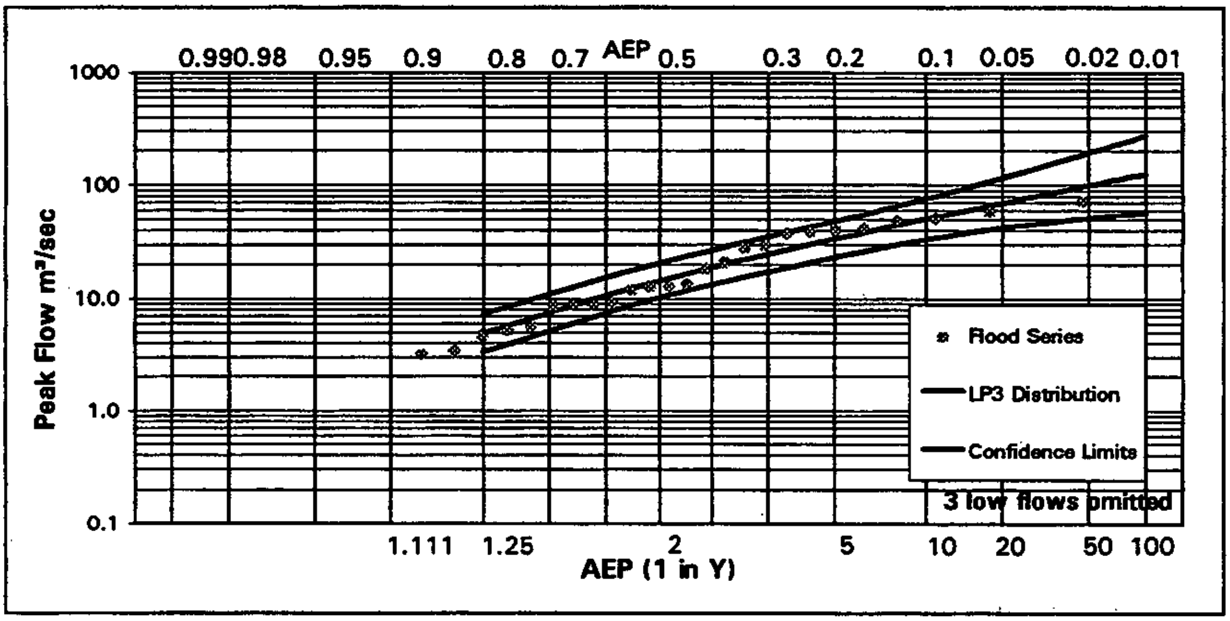

Figure A.3 Flood Frequency Curve for Orroral River

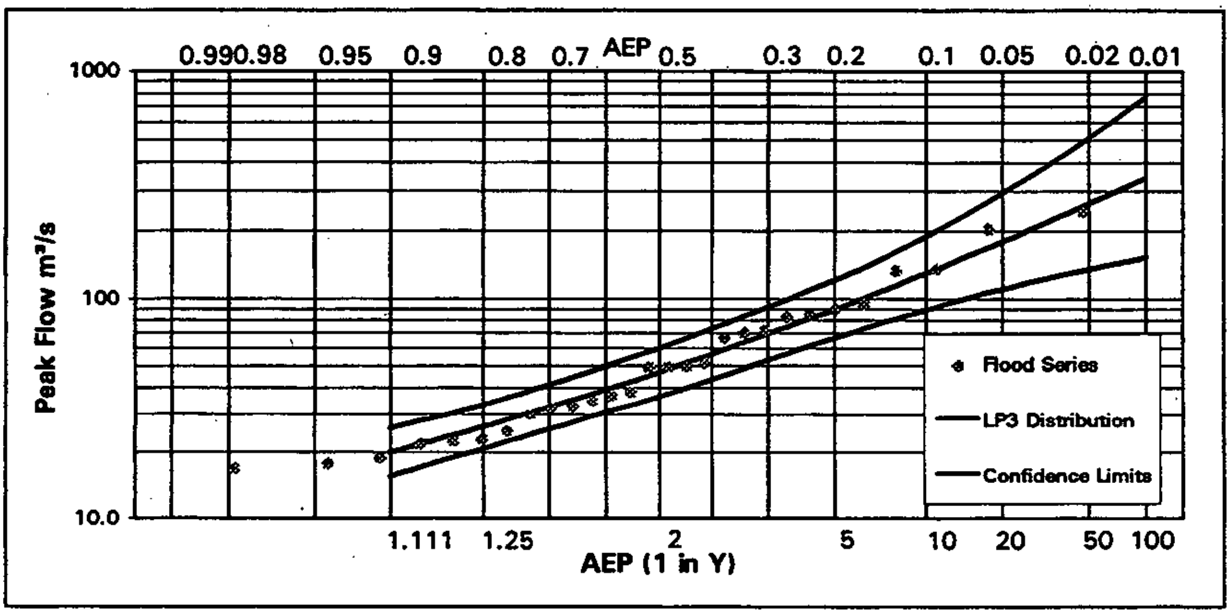

Figure A.4 Flood Frequency Curve for Aire River

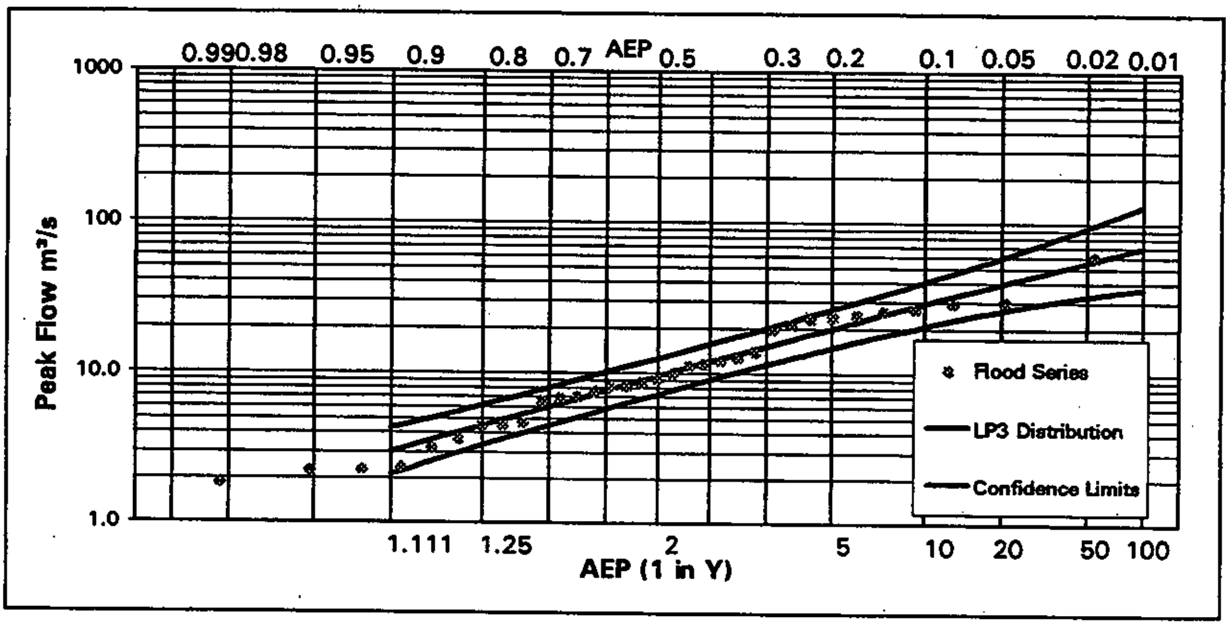

Figure A.5 Flood Frequency Curve for Moonee Creek

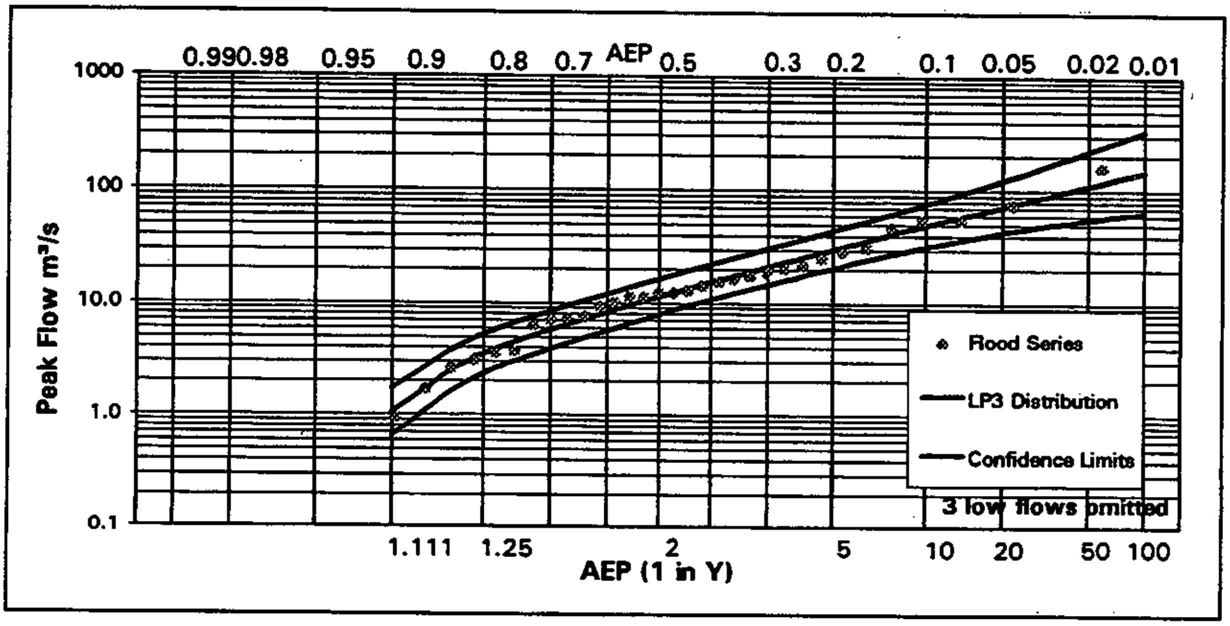

Figure A.6 Flood Frequency Curve for Wanalta Creek

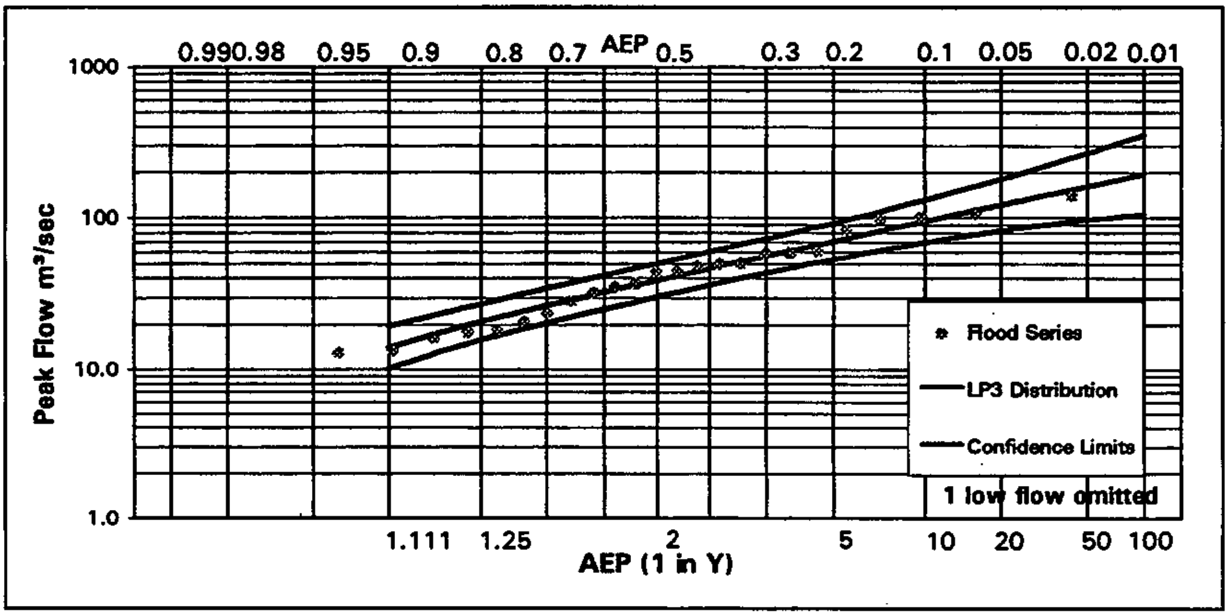

Figure A.7 Flood Frequency Curve for Tarwin River

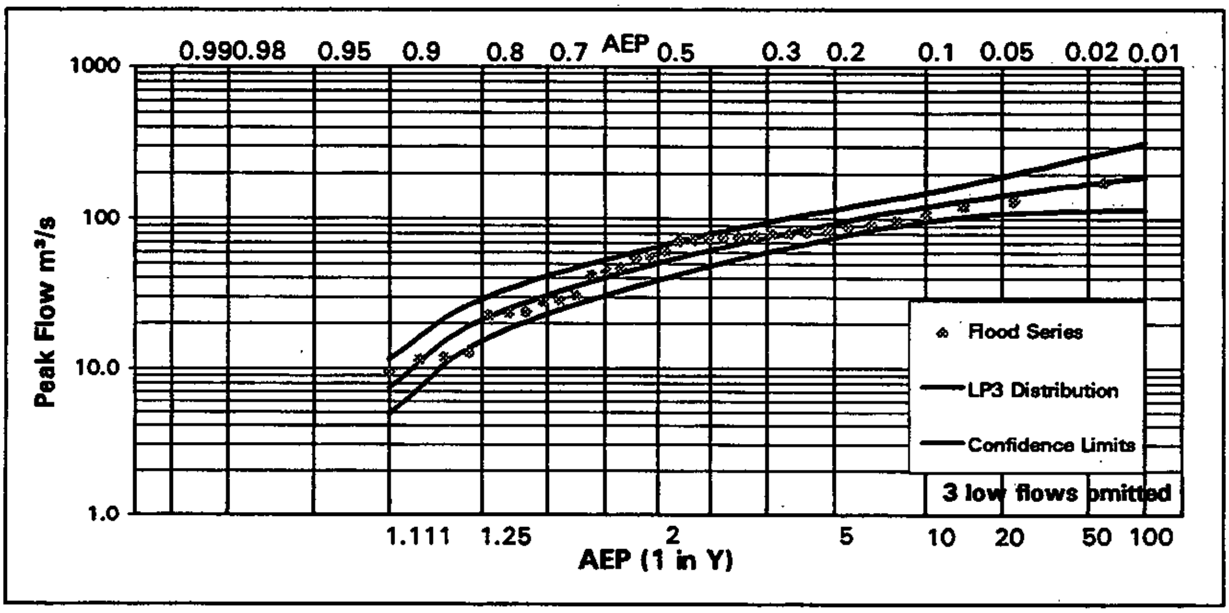

Figure A.8 Flood Frequency Curve for Lerderderg River

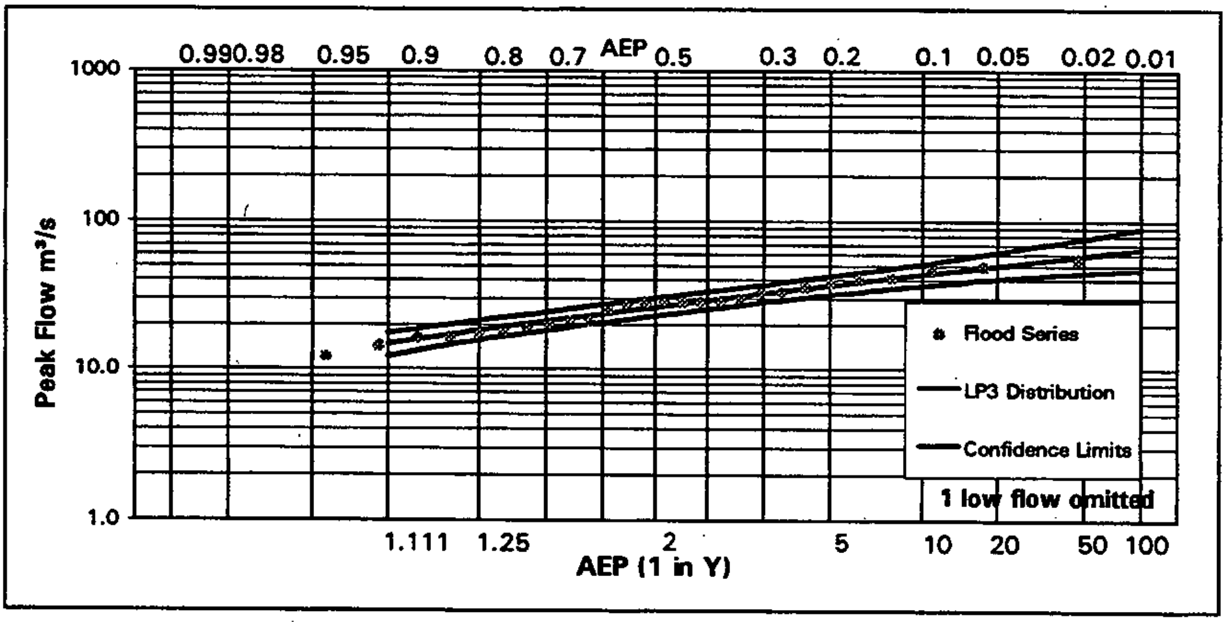

Figure A.9 Flood Frequency Curve for Moe River

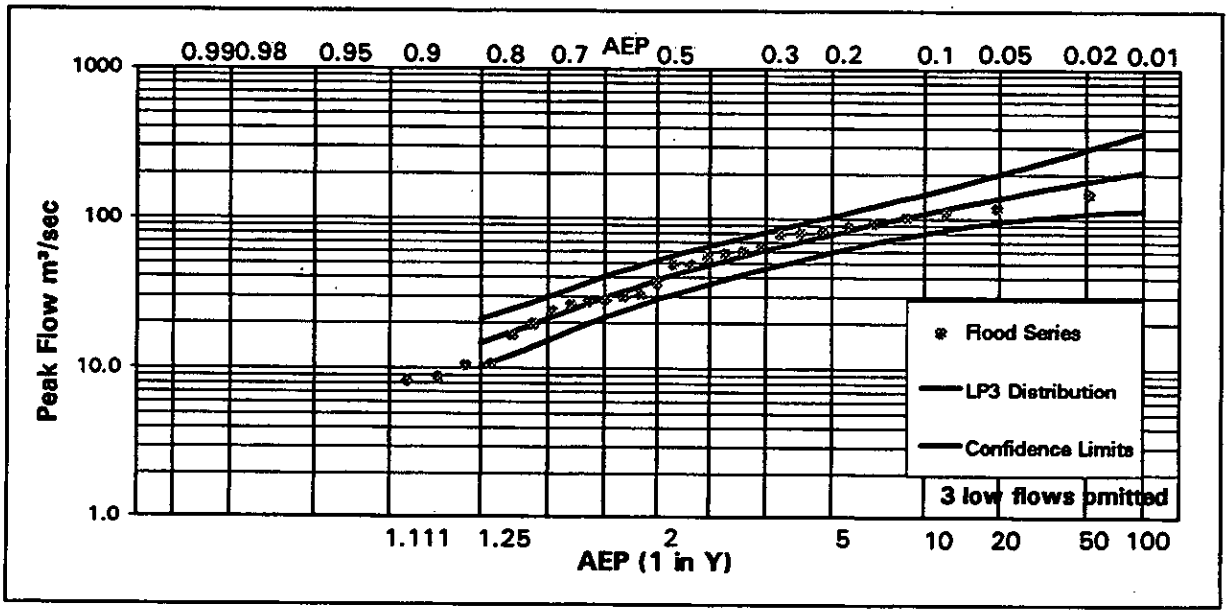

Figure A.10 Flood Frequency Curve for Avon River

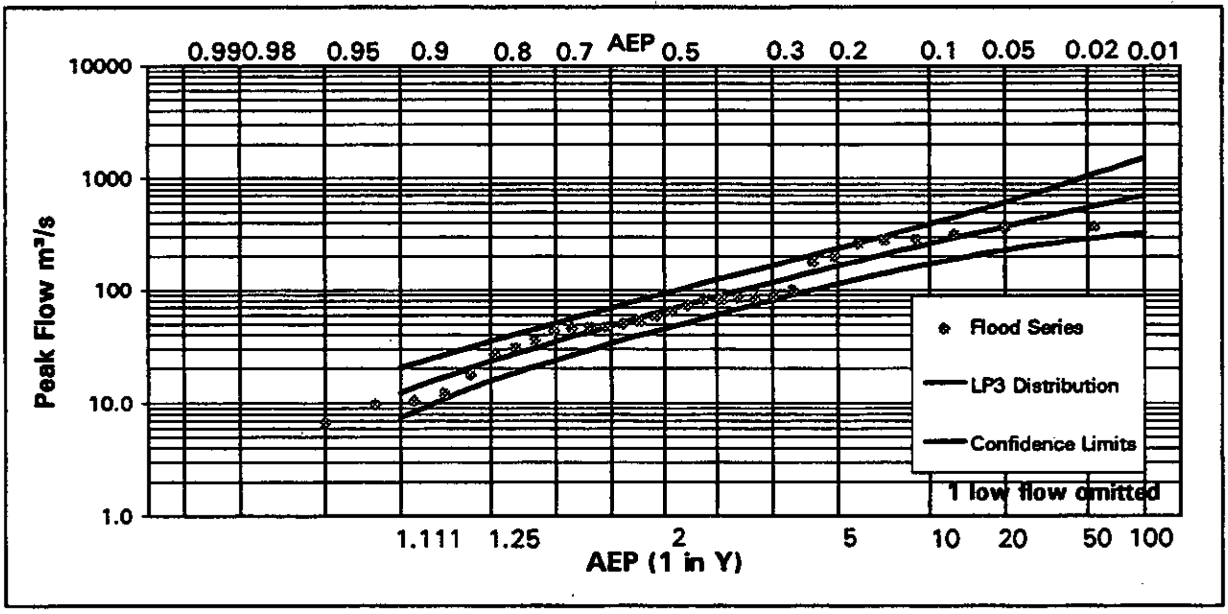

Figure A.11 Flood Frequency Curve for Seven Creeks

Appendix B
RORB Modelling

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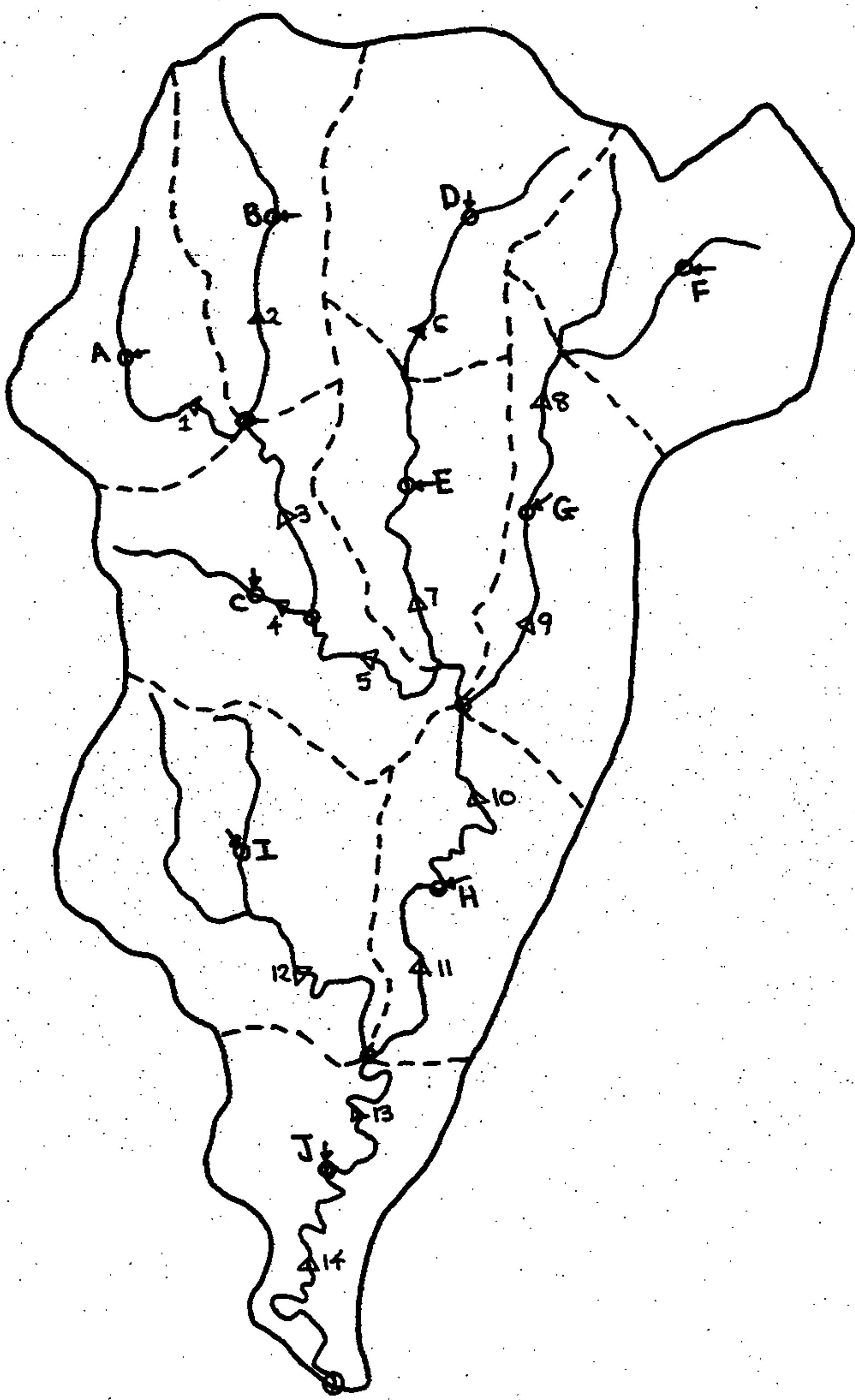

Figure B.1 RORB Model for Goodman Creek

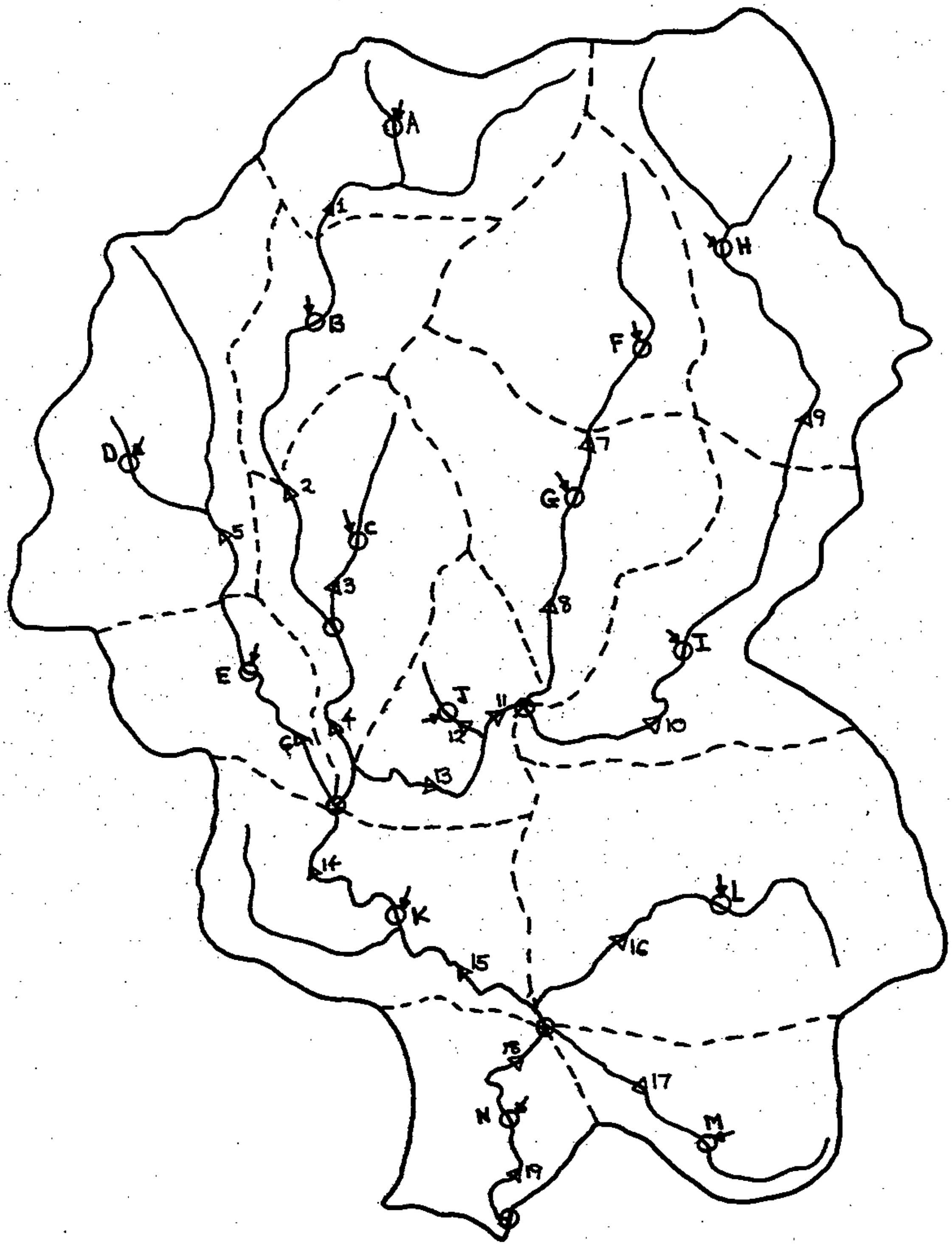

Figure B.2 RORB Model for Ford River

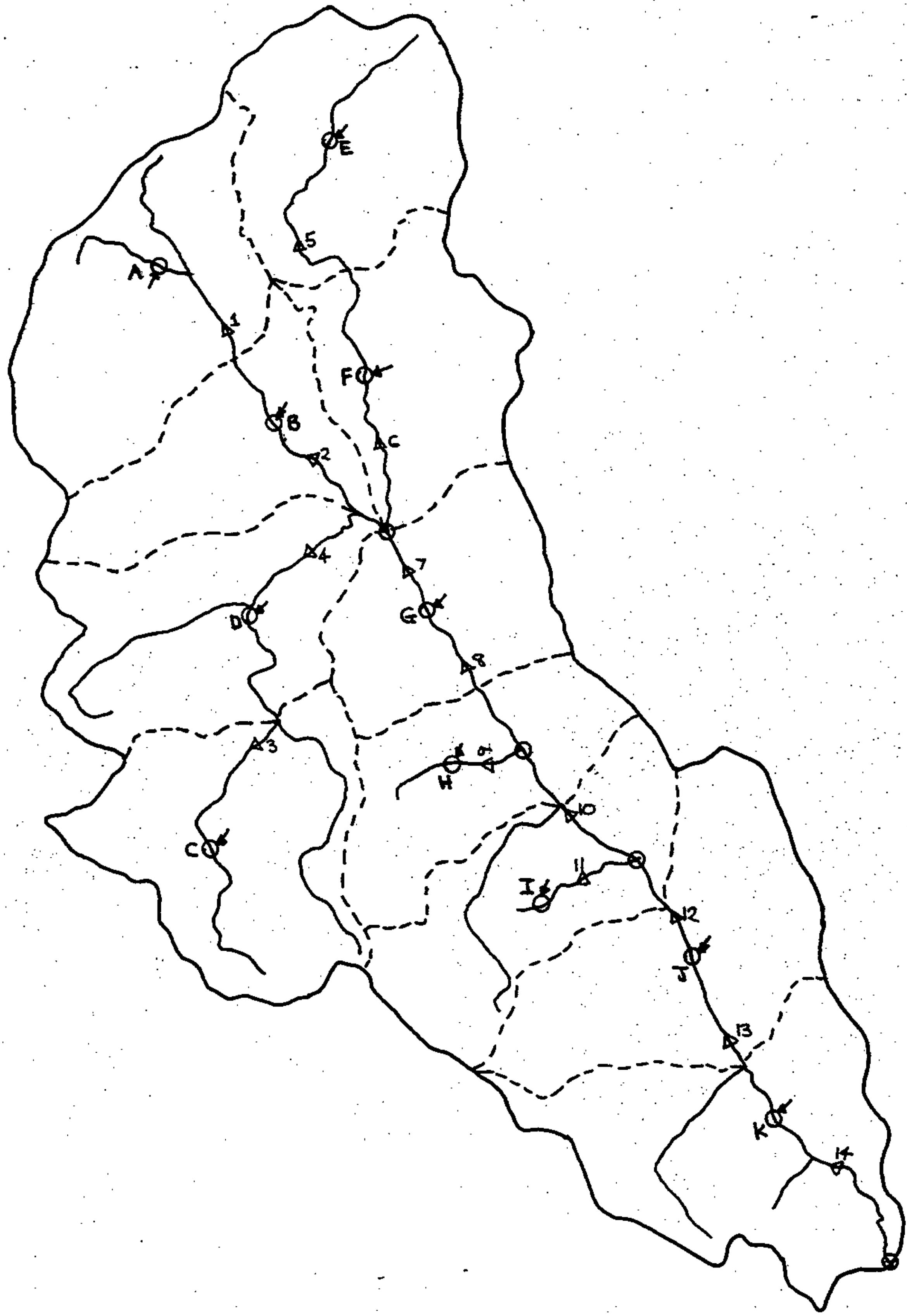

Figure B.3 RORB Model for Orroral River

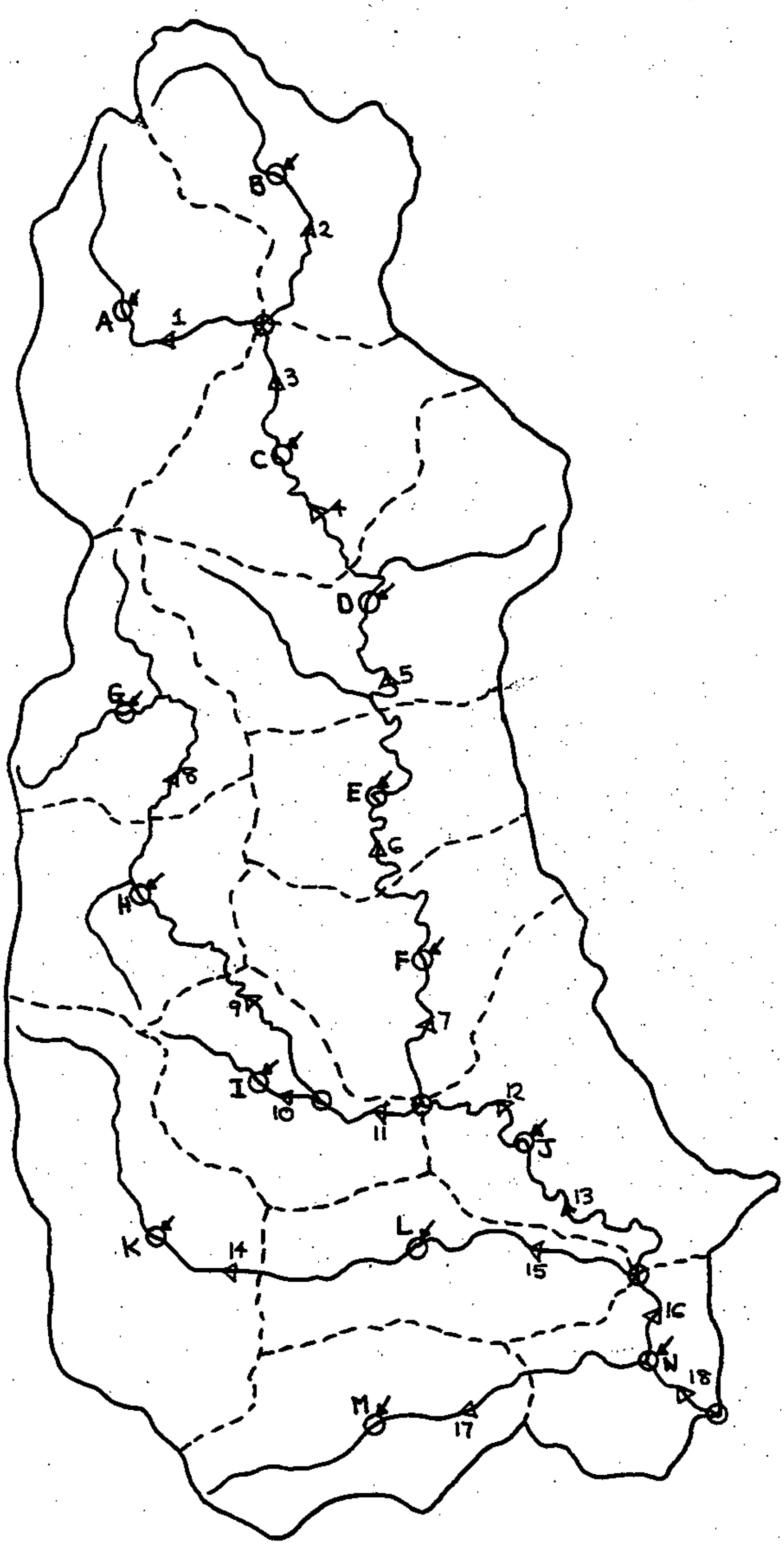

Figure B.4 RORB Model for Aire River

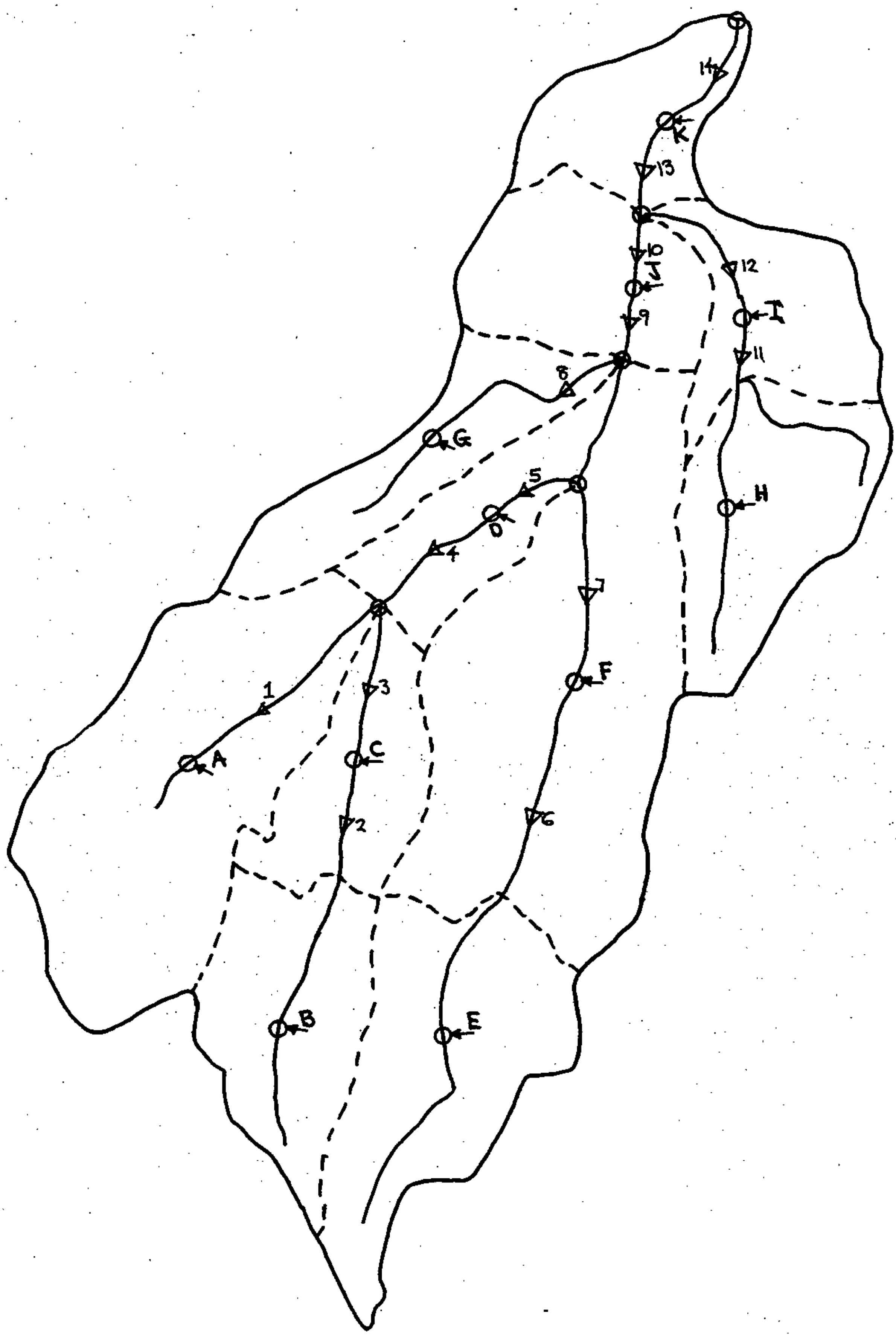

Figure B.5 RORB Model for Moonee Creek

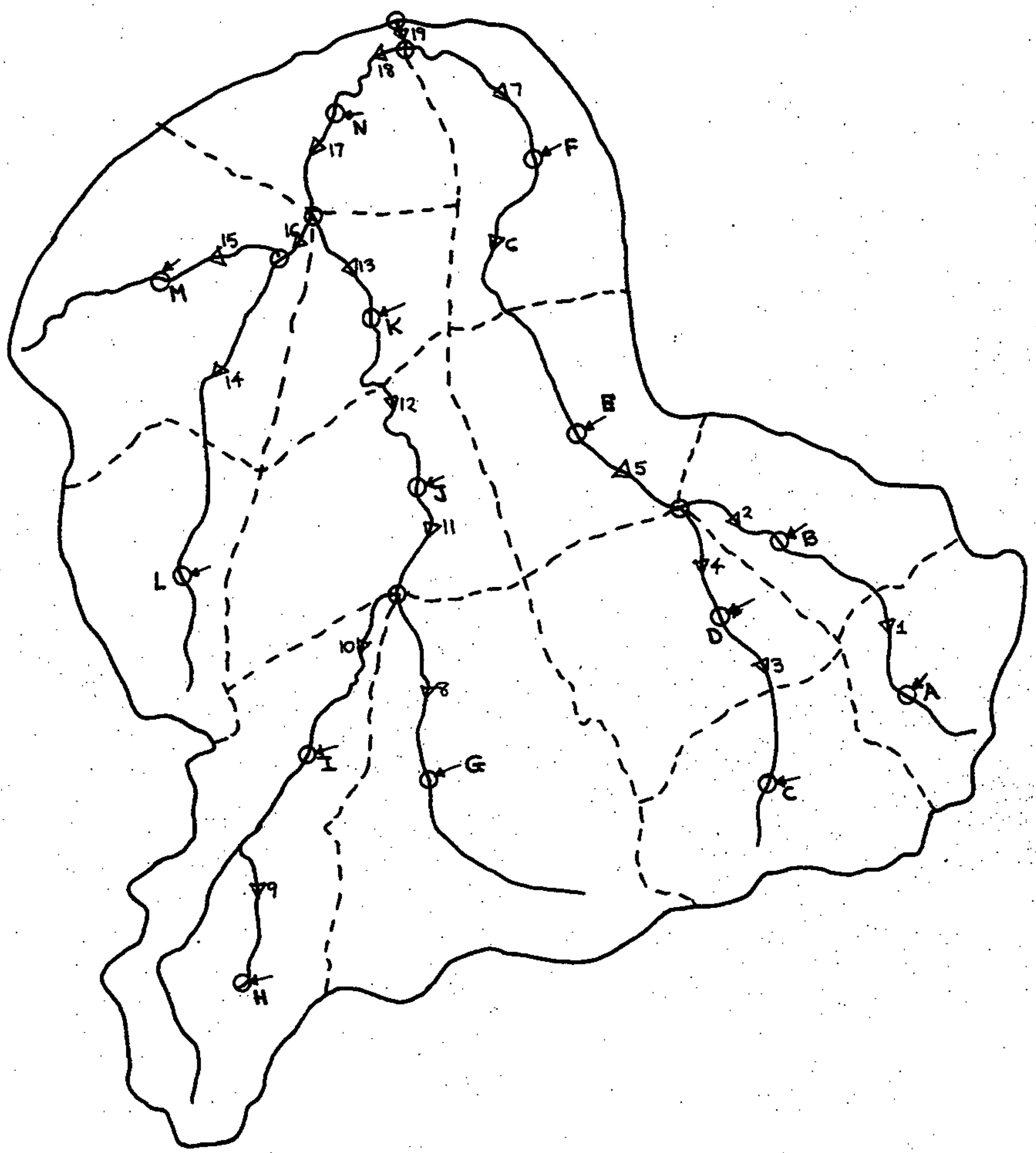

Figure B.6 RORB Model for Wanalta Creek

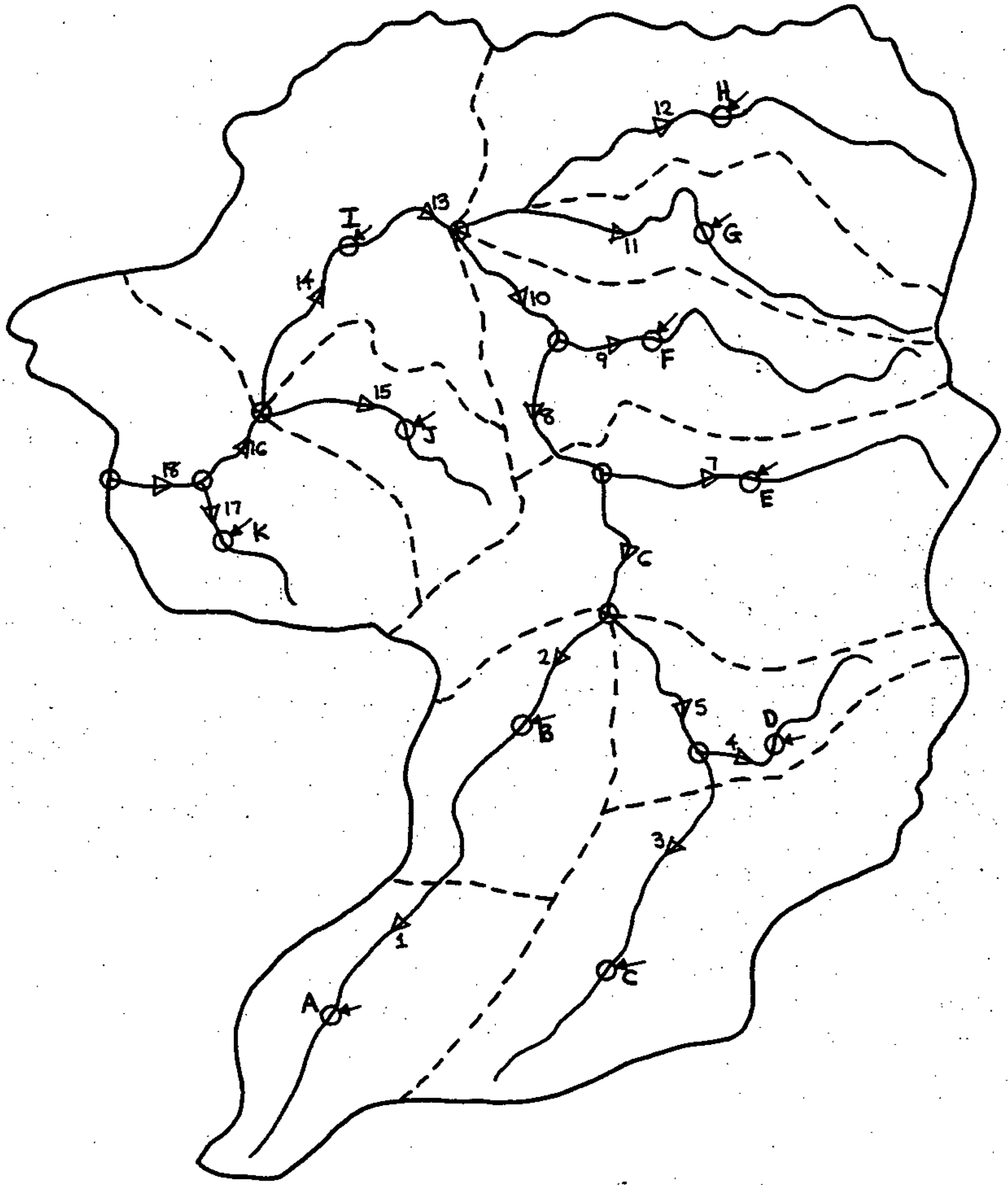

Figure B.7 RORB Model for Tarwin River

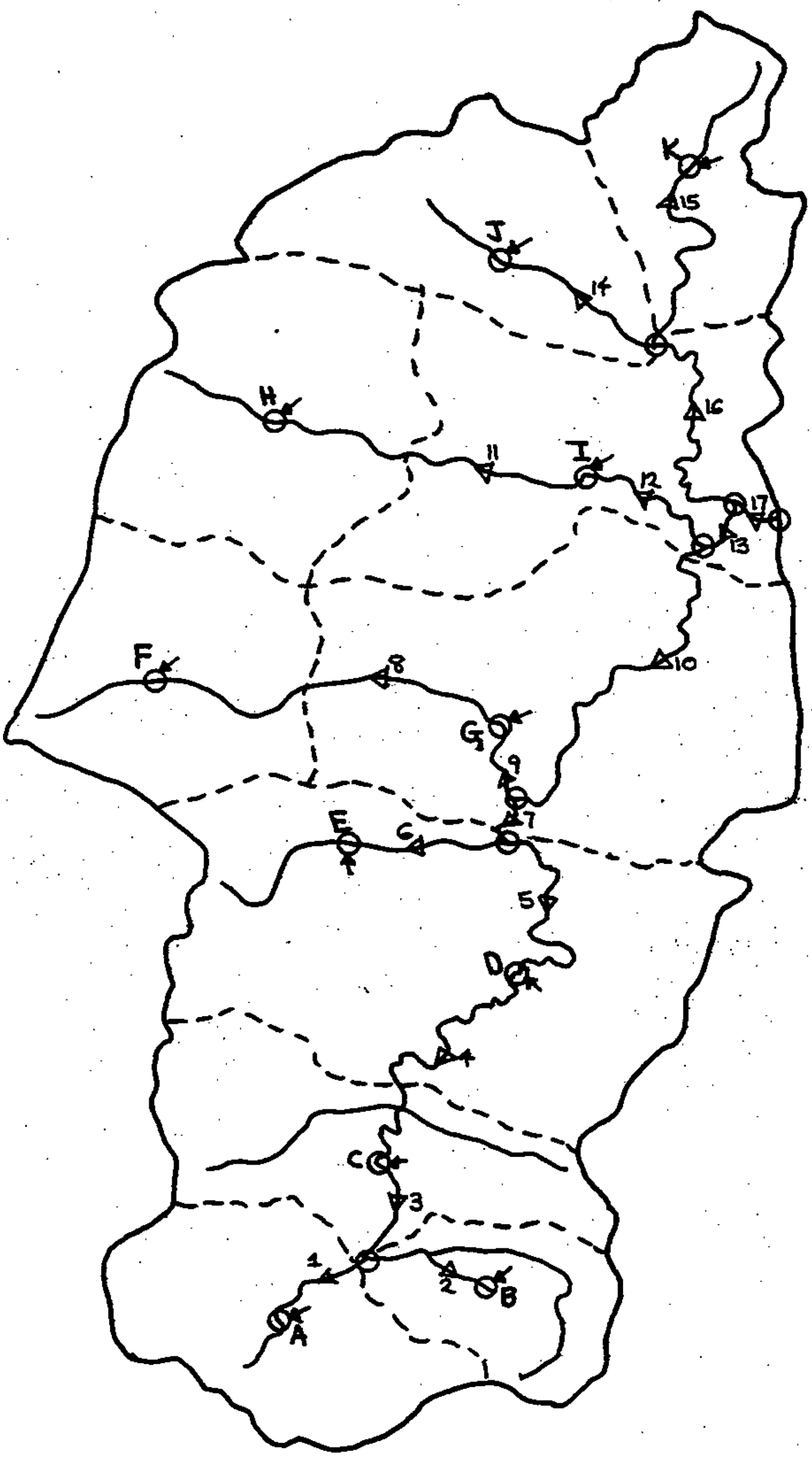

Figure B.8 RORB Model for Lerderderg River

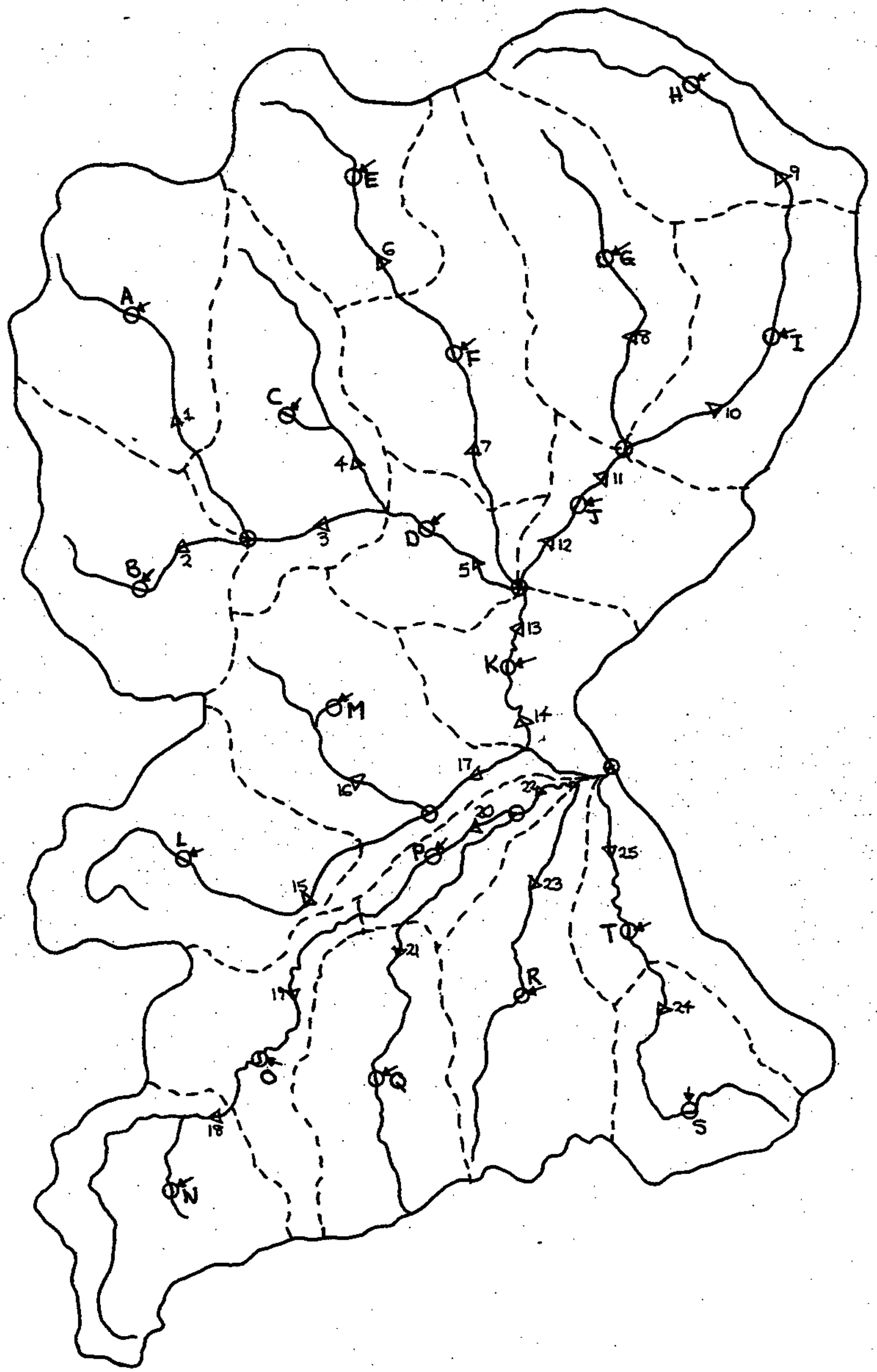

Figure B.9 RORB Model for Moe River

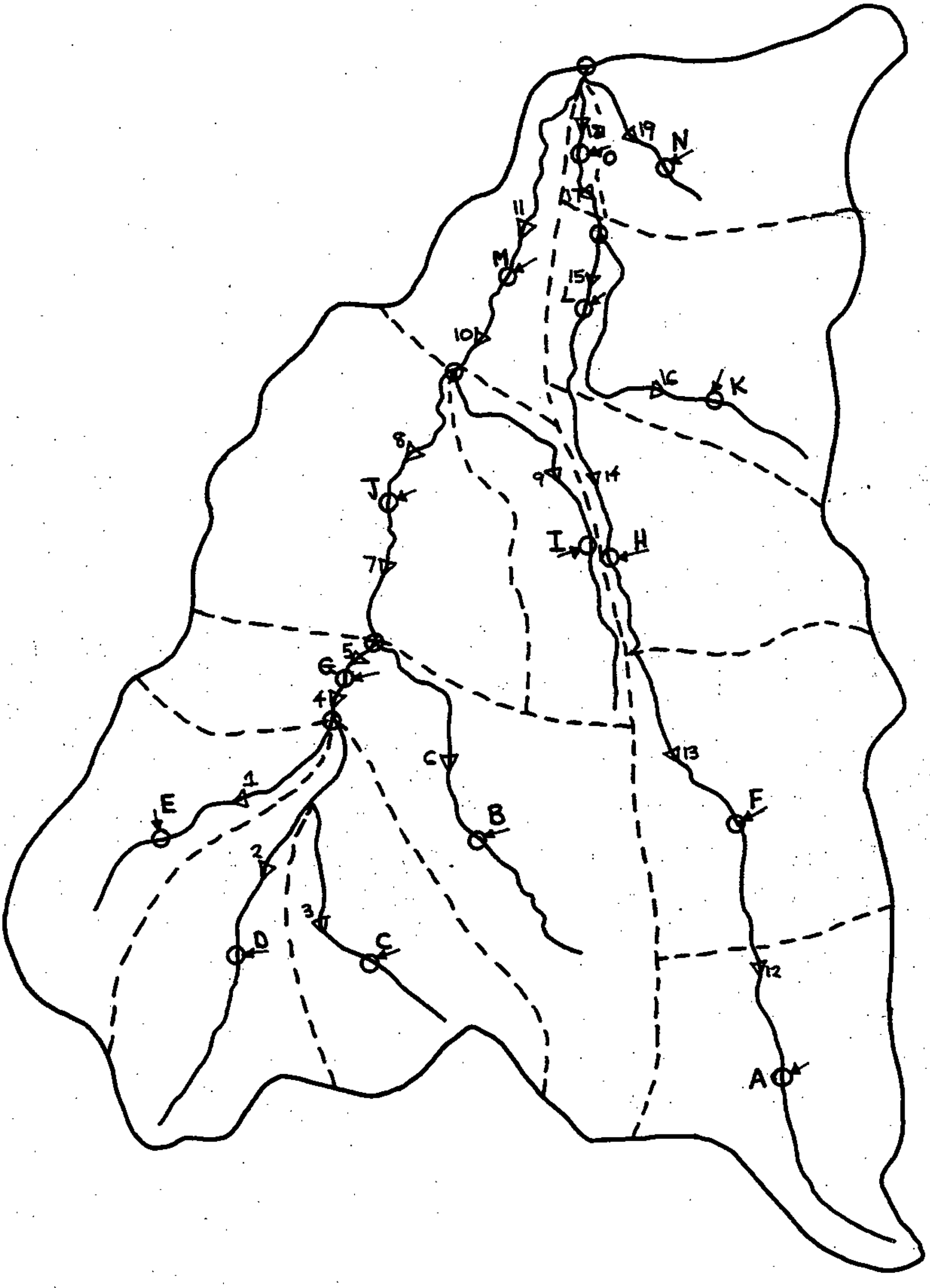

Figure B.10 RORB Model for Avon River

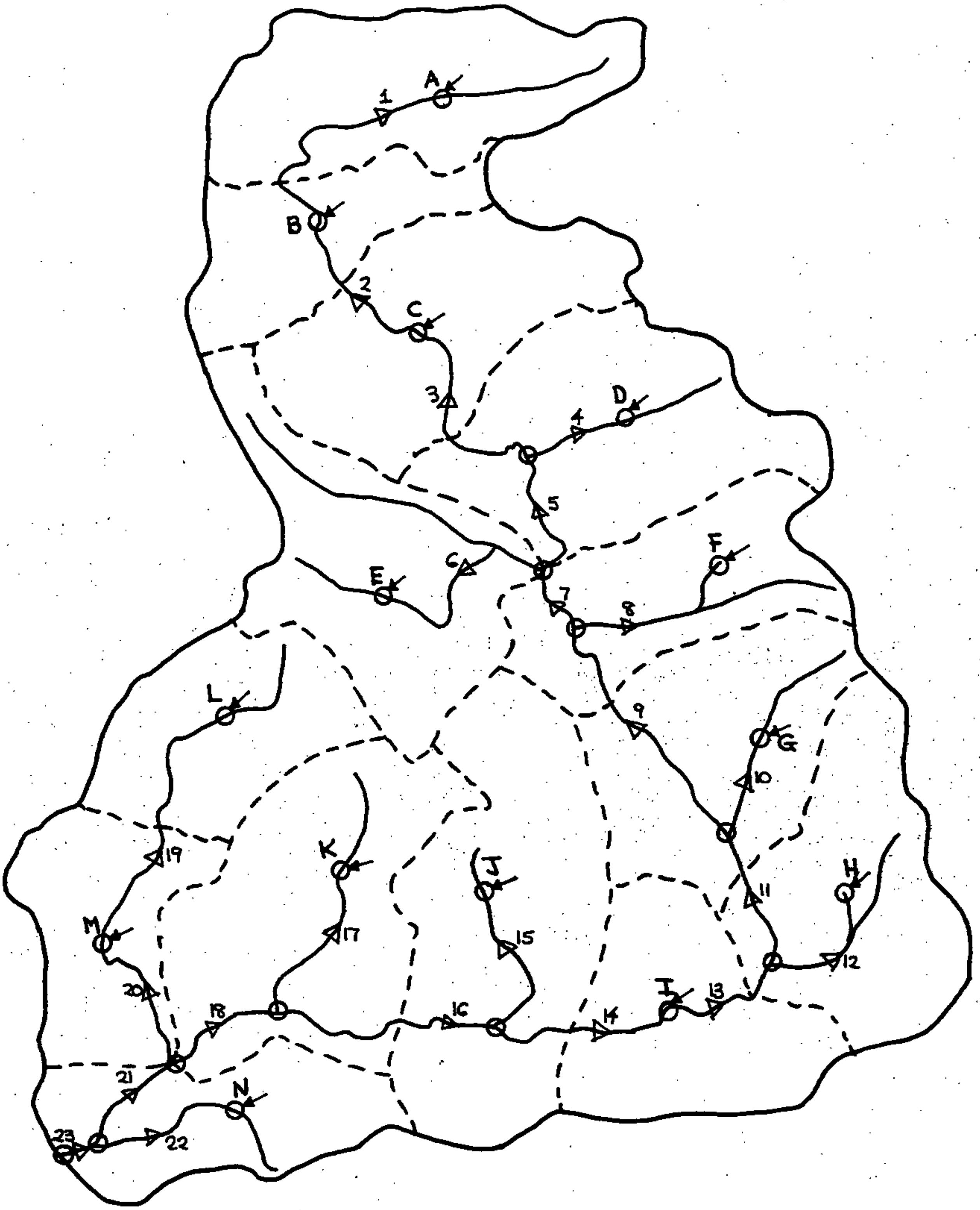

Figure B.11 RORB Model for Seven Creeks

RORB CATCHMENT FILES

Goodman Creek Catchment File

```
Goodman Creek at G.S 231219
1,              all reaches natural
1,1.4,-99,      sub-area A
3,              store h/g
1,1.5,-99,      sub-area B
4,              add h/g from B to stored h/g
5,1.7,-99,      route h/g through storage 3 to C
3,              store h/g
1,0.4,-99,      sub-area C
4,              add h/g from C to stored h/g
5,2.0,-99,      route h/g through storage 5
3,              store h/g
1,2.0,-99,      sub-area D
2,1.8,-99,      sub-area E
4,              add h/g from E to stored h/g
3,              store h/g
1,2.5,-99,      sub-area f
2,1.5,-99,      sub-area G
4,              add h/g from G to stored h/g
5,1.7,-99,      route h/g through storage 10 to H
2,1.6,-99,      sub-area H
3,              store h/g
1,2.5,-99,      sub-area I
4,              add h/g from I to stored h/g
5,1.5,-99,      route h/g through storage 13 to J
2,2.6,-99 ,     sub-area J
7
CATCHMENT OUTFLOW
0,              End of control vector
C Sub-area areas (sq.km)
2.5,2.8,3.8,3.6,2.6,3.9,3.1,2.7,4.7,2.7,-99
0,-99,          all areas pervious
```

Ford River Catchment File

```
Ford River at G.S 235229
1,                all reaches natural
1,1.4,-99,        sub-area F
2,1.8,-99,        sub-area G
3,                store h/g
1,4.0,-99,        sub-area H
2,2.1,-99,        sub-area I
4,                add h/g from I to stored h/g from G
5,0.5,-99,        route h/g through storage 11 to J
3,                store h/g
1,0.4,-99,        sub-area J
4,                add h/g from J to stored h/g from J
5,1.9,-99,        route h/g through storage 13
3,                store h/g
1,2.2,-99,        sub-area A
2,3.0,-99,        sub-area B
3,                store h/g
1,0.7,-99,        sub-area C
4,                add h/g from C to stored h/g
5,1.6,-99,        route h/g through storage 4
4,                add
3,                store h/g
1,2.2,-99,        sub-area D
2,1.3,-99,        sub-area E
4,                add running h/g to stored h/g
5,1.5,-99,        route h/g through storage 14
2,1.8,-99,        sub-area K
3,                store h/g
1,1.9,-99,        sub-area L
4,                add
3,                store h/g
1,1.7,-99,        sub-area M
4,                add h/g from M to stored h/g
5,1.0,-99,        route h/g through storage 18 to N
2,1.1,-99,        sub-area N
7
Ford River at Glenaire
0,                End of control vector
C Sub-area areas (sq.km)
3.3,2.6,3.9,5.8,2.3,4.0,4.2,6.3,4.1,2.5,4.3,7.9,2.7,2.5,-99
0,-99,             all areas pervious
```

Orroral River Catchment File

```
Orroral River at Road Crossing
1,           Channel type flag - all reaches natural
1,2.78,-99,  sub area A and reach 1 (2.78km)
2,2.15,-99,  sub area B and reach 2 (2.15km)
3,           store h/g
1,3.59,-99,  sub area C and reach 3 (3.59km)
2,2.31,-99,  sub area D and reach 4 (2.31km)
4,           add to stored h/g
3,           store h/g
1,3.27,-99,  sub area E and reach 5 (3.27km)
2,2.24,-99,  sub area F and reach 6 (2.24km)
4,           add to stored h/g
5,1.03,-99,  route along reach 7 (1.03km)
2,2.18,-99,  sub area G and reach 8 (2.18km)
3,           store h/g
1,1.03,-99,  sub area H and reach 9 (1.03km)
4,           add to store h/g
5,2.04,-99,  route along reach 10 (2.04km)
3,           store h/g
1,1.32,-99,  sub area I and reach 11 (1.32km)
4,           add to store h/g
5,1.38,-99,  route along reach 12 (1.38km)
2,2.16,-99,  sub area J and reach 13 (2.16km)
2,2.52,-99,  sub area K and reach 14 (2.52km)
7,           print h/g at the catchment outlet
Orroral River at Crossing
0,           end of control vector
C  sub area extents in sq.km
9.38,6.38,9.71,8.54,8.25,6.86,6.54,6.5,7.61,9.54,10.74,-99
0,-99,       no impervious areas
```

Aire River Catchment File

```
Aire River at Wyelangta
1,            Channel type flag - all reaches natural
1,1.89,-99, sub area A and reach 1 (1.89km)
3,            store h/g
1,1.75,-99, sub area B and reach 2 (1.75km)
4,            add to stored h/g
5,1.47,-99, route along reach 3 (1.47km)
2,2.38,-99, sub area C and reach 4 (2.38km)
2,3.50,-99, sub area D and reach 5 (3.50km)
2,2.94,-99, sub area E and reach 6 (2.94km)
2,1.75,-99, sub area F and reach 7 (1.75km)
3,            store h/g
1,3.15,-99, sub area G and reach 8 (3.15km)
2,3.78,-99, sub area H and reach 9 (3.78km)
3,            store h/g
1,0.91,-99, sub area I and reach 10 (0.91km)
4,            add to stored h/g
5,1.19,-99, route along reach 11 (1.19km)
4,            add to stored h/g
5,2.17,-99, route along reach 12 (2.17km)
2,3.08,-99, sub area J and reach 13 (3.08km)
3,            store h/g
1,3.36,-99, sub area K and reach 14 (3.36km)
2,2.52,-99, sub area L and reach 15 (2.52km)
4,            add to stored h/g
5,1.12,-99, route along reach 16 (1.12km)
3,            store h/g
1,3.36,-99, sub area M and reach 17 (3.36km)
4,            add to stored h/g
3,            store h/g
1,0.05,-99, sub area N and very short reach of 0.05km
4,            add to stored h/g
5,1.26,-99, route along reach 18 (1.26km)
7 ,           print h/g at the catchment outlet
Aire River at Wyelangta
0,            end of control vector
C  sub area extents in sq.km
7.96,6.19,6.63,9.19,5.06,6.34,5.31,5.55,5.06,7.56,9.38
5.50,6.14,3.93,-99
0,-99,        no impervious areas
```

Moonee Creek Catchment File

```
MOONEE CREEK STATION 404208
1,                  ALL REACHES NATURAL
1,3.3,-99,          SUB-AREA A    REACH 1
3,                  STORE H/G
1,3.8,-99,          SUB-AREA B    REACH 2
2,2.0,-99,         +SUB-AREA C    REACH 3
4,                  ADD RUNNING H/G TO STORED H/G
5,2.0,-99,          ROUTE TO D    REACH 4
2,2.9,-99,         +SUB-AREA D    REACH 5
3,                  STORE H/G
1,5.4,-99,          SUB-AREA E    REACH 6
2,4.5,-99,         +SUB-AREA F    REACH 7
4,                  ADD RUNNING H/G TO STORED H/G
3,                  STORE H/G
1,3.0,-99,          SUB-AREA G    REACH 8
4,                  ADD RUNNING H/G TO STORED H/G
5,0.9,-99,          ROUTE TO J    REACH 9
2,0.8,-99,         +SUB-AREA J    REACH 10
3,                  STORE H/G
1,2.5,-99,          SUB-AREA H    REACH 11
2,2.1,-99,         +SUB-AREA I    REACH 12
4,                  ADD RUNNING H/G TO STORED H/G
5,1.3,-99,          ROUTE TO K    REACH 13
2,1.5,-99,         +SUB-AREA K    REACH 14
7
CATCHMENT OUTFLOW
0
15.5,7.2,5.4,4.7,10.2,18.6,5.9,7.3,5.2,6.4,4.7,-99
0,-99
```

Wanalta Creek Catchment File

```
Wanalta creek @ Wanalta 405229
1,              All reaches natural
1,3.05,-99,     sub-area A reach 1
2,1.45,-99,     sub-area B reach 2
3,              store h/g
1,2.4,-99,      sub-area C reach 3
2,1.45,-99,     sub-area D reach 4
4,              add to stored h/g
5,1.7,-99,      route via reach 5
2,4.3,-99,      sub-area E reach 6
2,2.5,-99,      sub-area F reach 7
3,              store h/g
1,2.45,-99,     sub-area G reach 8
3,              store h/g
1,3.4,-99,      sub-area H reach 9
2,2.4,-99,      sub-area I reach 10
4,              add to stored h/g
5,1.4,-99,      route via reach 11
2,2.35,-99,     sub-area J reach 12
2,1.6,-99,      sub-area K reach 13
3,              store h/g
1,4.35,-99,     sub-area L reach 14
3,              store h/g
1,1.65,-99,     sub-area M reach 15
4,              add to stored h/g
5,0.75,-99,     route via reach 16
4,              add to stored h/g
5,1.25,-99,     route via reach 17
2,1.5,-99,      sub-area N reach 18
4,              add to stored h/g
5,0.4,-99,      route via reach 19
7
Results at Outlet
0,              end of control vector
5.2,5.6,7.3,8.0,7.5,7.4,15.6,8.0,5.3,8.9,4.7,6.8,11.7,5.7,-99
0,-99,          all areas pervious
```

Tarwin River Catchment File

```
TARWIN RIVER AT DUMBALK(227226)
1,               ALL REACHES NATURAL
1,4.1,-99,       Sub-Area A Reach 1
2,2.0,-99,       Sub-Area B Reach 2
3,               Store hydrograph
1,3.0,-99,       Sub-Area C Reach 3
3,               Store hydrograph
1,1.2,-99,       Sub-Area D reach 4
4,               add
5,2.0,-99,       Route to  Reach 5
3,               Add Running hydrograph to stored hydrograph
5,1.5,-99,       route to Reach 6
1,1.8,-99,       Sub-Area E Reach 7
4,               Add running hydrograph to stored hydrograph
5,2.0,-99,       route to Reach 8
2,1.1,-99,       Sub-Area F Reach 9
4,               Add Running hydrograph to stored hydrograph
5,2.0,-99,       route to reach 10
3,               Store hydrograph
1,3.5,-99,       Sub-Area G Reach 11
3,               Store hydrograph
1,3.0,-99,       Sub-Area H Reach 12
4,               Add Running hydrograph to stored hydrograph
1,1.1,-99,       Route To I Reach 13
5,2.0,-99,       Route to Reach 14
3,               Store hydrograph
1,1.5,-99,       Route J Reach 15
4,               Add
5,1.0,-99,       Route to Reach 16
3,               Store
1,0.9,-99,       Route to K Reach 17
4,               Add
5,1.0,-99,       Route to reach 18
4,               add
7, PRINTING RESULTS AT OUTLET
TARWIN RIVER
0
10.1,8.5,16.7,6.4,19.2,10.8,7.2,14.5,15.1,5.2,13.3,-99
0,-99,           All areas pervious
```

Lerderderg River Catchment File

```
LERDERBERG RIVER STATION 231213
1,           ALL REACHES NATURAL
1,1.7,-99,   sub area A reach 1
3,           store h/g
1,1.8,-99,   sub area B reach 2
4,           add h/g
5,1.3,-99,   route via reach 3
2,4.3,-99,   sub area C reach 4
2,3.5,-99,   sub area D reach 5
3,           store h/g
1,2.1,-99,   sub area E reach 6
4,           add h/g
5,0.7,-99,   route via reach 7
3,           store h/g
1,5.4,-99,   sub area F reach 8
2,0.8,-99,   sub area G reach 9
5,4.9,-99,   route via reach 10
3,           store h/g
1,4.4,-99,   sub area H reach 11
2,2.0,-99,   sub area I reach 12
4,           add h/g
5,1.1,-99,   route via reach 13
1,2.3,-99,   sub area J reach 14
3,           store h/g
1,3.5,-99,   sub area K reach 15
4,           add h/g
5,3.1,-99,   route via reach 16
4,           add h/g
5,0.6,-99,   route via reach 17
7
Catchment outflow
0,           END OF CONTROL VECTOR
C SUB-AREA AREAS (SQ. KM)
12.0,7.1,12.9,18.3,9.4,12.6,25.8,16.1,17.6,12.2,9.2,-99
0,-99,       ALL AREAS PERVIOUS
```

Moe River Catchment File

```
Moe River at Darnum
1,           Channel type flag - all reaches natural
1,5.02,-99,  sub area A and reach 1 (5.02km)
3,           store h/g
1,2.38,-99,  sub area B and reach 2 (2.38km)
4,           add to stored h/g
5,3.43,-99,  route along reach 3 (3.43km)
3,           store h/g
1,3.43,-99,  sub area C and reach 4 (3.43km)
4,           add to stored h/g
2,2.20,-99,  sub area D and reach 5 (2.20km)
3,           store h/g
1,3.78,-99,  sub area E and reach 6 (3.78km)
2,4.58,-99,  sub area F and reach 7 (4.58km)
4,           add to stored h/g
3,           store h/g
1,3.96,-99,  sub area G and reach 8 (3.96km)
3,           store h/g
1,5.72,-99,  sub area H and reach 9 (5.72km)
2,3.52,-99,  sub area I and reach 10 (3.52km)
4,           add to store h/g
5,1.32,-99,  route along reach 11 (1.32km)
2,1.85,-99,  sub area J and reach 12 (1.85km)
4,           add to stored h/g
5,1.50,-99,  route along reach 13 (1.50km)
2,1.76,-99,  sub area K and reach 14 (1.76km)
3,           store h/g
1,5.02,-99,  sub area L and reach 15 (5.02km)
3,           store h/g
1,3.34,-99,  sub area M and reach 16 (3.34km)
4,           add to stored h/g
5,2.11,-99,  route along reach 17 (2.11km)
4,           add to stored h/g
5,1.85,-99,  route along reach 18 (1.85km)
3,           store h/g
1,3.17,-99,  sub area N and reach 19 (3.17km)
2,5.63,-99,  sub area O and reach 20 (5.63km)
2,1.58,-99,  sub area P and reach 21 (1.58km)
3,           store h/g
1,6.25,-99,  sub area Q and reach 22 (6.25km)
4,           add to stored h/g
5,2.29,-99,  route along reach 23 (2.29km)
4,           add to stored h/g
3,           store h/g
1,5.10,-99,  sub area R and reach 24 (5.10km)
4,           add to stored h/g
3,           store h/g
1,5.46,-99,  sub area S and reach 25 (5.46km)
2,3.26,-99,  sub area T and reach 26 (3.26km)
4,           add to stored h/g
7,           print h/g at the catchment outlet
Moe River at Darnum
0,           end of control vector
C  sub area extents in sq.km
13.64,12.20,13.50,6.27,10.13,13.64,13.23,11.44,11.99,7.79,7.37
13.44,14.40,12.33,8.41,4.20,11.37,13.23,7.79,7.65,-99
0,-99,       no impervious areas
```

Avon River Catchment File

```
AVON RIVER 415224A
1,      ALL REACHES NATURAL
1,4.62,-99,       Sub-Area E Reach 1
3,                Store hydrograph
1,4.97,-99,       Sub-Area D Reach 2
4,                add
3,                Add Running hydrograph to stored hydrograph
1,6.03,-99,       Sub-Area C Reach 3
4,                Add Running hydrograph to stored hydrograph
5,0.89,-99,       Route to G Reach 4
2,0.71,-99,       Sub-Area G Reach 5
3,                Store hydrograph
1,5.33,-99,       Sub-area B Reach 6
4,                Add Running hydrograph to stored hydrograph
5,3.20,-99,       Route to J Reach 7
2,3.55,-99,       Sub-Area J Reach 8
3,                Store hydrograph
1,5.33,-99,       Sub-Area I Reach 9
4,                Add Running hydrograph to stored hydrograph
5,2.13,-99,       Route to M Reach 10
2,4.62,-99,       Route Sub-Area M Reach 11
3,                Store hydrograph
1,4.97,-99,       Sub-Area A Reach 12
2,6.39,-99,       Sub-Area F Reach 13
2,5.68,-99,       Sub-Area H Reach 14
2,1.42,-99,       Sub-Area L Reach 15
3,                Store hydrograph
1,6.04,-99,       Sub-Area K Reach 16
4,                Add Running hydrograph to stored hydrograph
5,1.78,-99,       Route to O Reach 17
2,1.60,-99,       Sub-Area O Reach 18
4,                Add Hydrograph
3,                store
1,2.84,-99,       Sub-Area N Reach 19
4,                add
8,3,-99
7
PRINTING RESULTS AT OUTLET
0
18.52,18.68,18.35,6.75,30.73,32.15,11.80,11.41,23.56,26.89,19.15,2.84,20.47
,1.25,18.25,-99
0,-99,             All areas pervious
```

Seven Cks Catchment File

```
SEVEN CREEKS STATION 405237
1,           ALL REACHES NATURAL
1,5.5,-99,   sub area A reach 1
2,3.5,-99,   sub area B reach 2
2,4.5,-99,   sub area C reach 3
3,           store
1,2.3,-99,   sub area D reach 4
4,           add to previously stored h/g
5,2.8,-99,   route through reach 5
3,           store
1,5.0,-99,   sub area E reach 6
4,           add to previously stored h/g
5,1.6,-99,   route through reach 7
3,           store
1,3.7,-99,   sub area F reach 8
4,           add to previously stored h/g
5,6.2,-99,   route through reach 9
3,           store
1,2.3,-99,   sub area G reach 10
4,           add to previously stored h/g
5,2.9,-99,   route through reach 11
3,           store
1,2.8,-99,   sub area H reach 12
4,           add to previously stored h/g
5,2.5,-99,   route through reach 13
2,4.3,-99,   sub area I reach 14
3,           store
1,3.6,-99,   sub area J reach 15
4,           add to previously stored h/g
5,5.2,-99,   route through reach 16
3,           store
1,3.9,-99,   sub area K reach 17
4,           add to previously stored h/g
5,2.7,-99,   route through reach 18
3,           store
1,6.0,-99,   sub area L reach 19
2,2.2,-99,   sub area M reach 20
4,           add to previously stored h/g
5,2.7,-99,   route through reach 21
3,           store
1,3.3,-99,   sub area N reach 22
4,           add to previously stored h/g
5,0.9,-99,   route through reach 23 to catchment outlet
C
7,           PRINTING RESULTS AT OUTLET
Outlet
0,           END OF CONTROL VECTOR
C
C SUB-AREA AREAS (SQ. KM)
25.4,14.4,25.7,28.4,27.9,20.6,24.6,25.0,21.6,35.0,30.8,18.2,16.4,18.2,-99
0,-99,       ALL AREAS PERVIOUS
```

SUMMARY OF RORB CALIBRATIONS

All RORB models were calibrated with m=0.8

Table B.1 Summary of RORB Calibration for Goodman Creek

	Initial Loss/Continuing Loss			Initial Loss/Proportional Loss		
Date	IL (mm)	CL (mm/h)	k_c	IL (mm)	RC	k_c
*07/11/71**	23	2.50	8.65	*35*	*0.37*	*3.00*
06/02/73+	*0*	*14.74*	*6.2*	*10*	*0.17*	*1.45*
14/10/76	20	0.73	7.75	23	0.54	3.15
03/07/78	1	1.72	6.75	4	0.61	5.65
06/08/78	15	2.30	10.55	15	0.47	6.10
19/11/78	15	3.26	7.75	23	0.55	4.20
mean	14.8	2.1	8.3	16.3	0.54	4.8

* Event 7/11/71 was excluded from the mean for the proportional loss because the fit was poor.
+ Event 6/2/73 was excluded from the mean for both loss models because the fit was poor

Table B.2 Summary of RORB Calibration for Ford River

	Initial Loss/Continuing Loss			Initial Loss/Proportional Loss		
Date	IL (mm)	CL (mm/h)	k_c	IL (mm)	RC	k_c
22/09/76	7	2.27	13.2	18	0.36	7.6
16/10/76	18	0.79	11.3	23	0.77	9.6
*03/06/78**	*40*	*0.45*	*11.3*	*40*	*0.90*	*10.6*
03/07/78	5	2.79	11.6	23	0.29	4.5
28/06/80	0	4.10	15.8	35	0.25	7.4
22/03/83	40	3.38	12.0	64	0.33	5.3
18/09/84	70	3.04	12.9	115	0.48	8.2
04/10/87	60	2.66	17.0	60	0.37	13.1
mean	28.6	2.72	13.4	48.3	0.41	7.9

* Event 3/6/78 was excluded from the mean because the fit was poor.

Table B.3 Summary of RORB Calibration for Orroral River

	Initial Loss/Continuing Loss			Initial Loss/Proportional Loss		
Date	IL (mm)	CL (mm/h)	k_c	IL (mm)	RC	k_c
09/02/71	18	3.10	18.0	16	0.31	16.6
*25/08/74**	*0*	*3.54*	*39.2*	26	0.27	18.1
12/07/75	7	3.42	19.8	11	0.17	13.5
17/05/78	0	4.79	16.8	26	0.15	13.0
25/08/83	0	3.83	17.2	12	0.17	13.0
25/01/84	15	5.89	16.5	26	0.26	13.1
mean	8	4.21	17.6	20	0.22	14.5

* Event 25/8/74 was excluded from the mean for the continuing loss model because the fit was poor.

Table B.4 Summary of RORB Calibration for Aire River

	Initial Loss/Continuing Loss			Initial Loss/Proportional Loss		
Date	IL (mm)	CL (mm/h)	k_c	IL (mm)	RC	k_c
02/01/87	34	1.79	17.2	42	0.53	14.0
31/01/90	0	4.17	23.9	58	0.28	9.5
08/02/90	*30*	*0.84*	*10.4*	*15*	*0.53*	*7.7*
11/06/91	0	0.24	16.5	0	0.73	13.5
14/12/91	62	2.99	16.4	77	0.47	12.2
mean	24	2.3	18.5	44	0.50	12.3

* Event 8/2/90 was excluded from the mean because the fit was poor.

Table B.5 Summary of RORB Calibration for Moonee Creek

	Initial Loss/Continuing Loss			Initial Loss/Proportional Loss		
Date	IL (mm)	CL (mm/h)	k_c	IL (mm)	RC	k_c
15/05/74	9.0	3.24	29.0	25	0.29	20
*18/07/74**	*0*	*4.14*	*8.5*	*10*	*0.14*	*5.2*
18/09/75	0	7.02	17.0	0	0.12	11.0
04/0781	0	3.03	21.5	0	0.24	12.5
*21/07/81**	*15*	*1.93*	*13.0*	*15*	*0.26*	*7.5*
06/06/88	11	2.43	22.5	17	0.17	13.5
04/10/93	0	5.70	32.5	0	0.23	31
mean	4.0	4.3	24.5	8.4	0.21	17.6

* Events 18/7/74 and 21/7/81 were excluded from the mean because the fit was poor.

Table B.6 Summary of RORB Calibration for Wanalta Creek

	Initial Loss/Continuing Loss			Initial Loss/Proportional Loss		
Date	IL (mm)	CL (mm/h)	k_c	IL (mm)	RC	k_c
13/05/74	8	2.27	18.9	8	0.40	11.3
03/10/74	19	1.84	13.7	19	0.62	13.2
23/10/75	15	1.66	24.0	15	0.43	15.4
26/08/79	15	2.77	14.3	15	0.33	13.3
07/09/83	5	0.56	20.5	5	0.78	18.1
13/01/84	26	4.32	14.6	26	0.21	12.0
03/07/91	10	1.7	14.3	10	0.37	12.7
mean	14	2.16	17.2	14	0.45	13.7

Table B.7 Summary of RORB Calibration for Tarwin River

	Initial Loss/Continuing Loss			Initial Loss/Proportional Loss		
Date	IL (mm)	CL (mm/h)	k_c	IL (mm)	RC	k_c
08/07/74	15	1.12	15	10	0.49	10
24/08/75	10	0.89	22	10	0.55	16
07/08/76	15	1.60	19	40	0.49	15.5
28/07/77	20	0.68	20	20	0.56	10
*04/06/78**	*15*	*0.05*	*10*	15	0.97	10
14/06/78	20	0.17	22	20	0.76	16.5
mean	16	0.89	19.6	19.2	0.64	13.0

* Event 4/6/78 was excluded from the mean for the continuing loss because the fit was poor.

Table B.8 Summary of RORB Calibration for Lerderderg River

	Initial Loss/Continuing Loss			Initial Loss/Proportional Loss		
Date	IL (mm)	CL (mm/h)	k_c	IL (mm)	RC	k_c
12/05/74	26	0.97	14.1	26	0.75	12.1
20/09/76	16	1.15	17.0	23	0.56	11.0
07/08/78	11	1.56	25.7	11	0.44	14.4
01/10/79	22	1.18	20.0	22	0.55	13.4
mean	18.8	1.2	19.2	20.5	0.58	12.7

Table B.9 Summary of RORB Calibration for Moe River

	Initial Loss/Continuing Loss			Initial Loss/Proportional Loss		
Date	IL (mm)	CL (mm/h)	k_c	IL (mm)	RC	k_c
10/08/75	0	1.52	21.1	3.5	0.32	17.0
17/09/75	3	1.48	17.8	7	0.31	10.1
27/06/80	2	4.01	22.7	41	0.24	15.8
20/08/81	12	1.20	15.1	8	0.42	16.0
08/09/83	3	3.33	18.1	3	0.21	14.6
12/09/83	0	1.07	26.6	5	0.46	24.7
mean	3.3	2.10	20.2	11.3	0.33	16.3

Table B.10 Summary of RORB Calibration for Avon River

	Initial Loss/Continuing Loss			Initial Loss/Proportional Loss		
Date	IL (mm)	CL (mm/h)	k_c	IL (mm)	RC	k_c
*06/02/73**	*15*	*5.57*	*13.8*	*15*	*0.26*	*4.0*
19/10/73	40	2.78	15.0	40	0.48	12.0
15/05/74	40	0.86	18.5	40	0.66	18.0
05/10/74	7.5	2.34	17.0	7	0.43	15.0
24/10/75	10	0.81	14.4	10	0.65	12.0
28/09/79	20	0.82	16.0	20	0.71	13.5
04/08/81	14.5	0.33	14.5	14.5	0.83	14.3
mean	22.0	1.32	15.9	21.9	0.63	14.1

* Event 6/2/73 was excluded from the mean because the fit was poor.

Table B.11 Summary of RORB Calibration for Seven Creeks

	Initial Loss/Continuing Loss			Initial Loss/Proportional Loss		
Date	IL (mm)	CL (mm/h)	k_c	IL (mm)	RC	k_c
*13/05/74**	*38*	*0.79*	*10.1*	*38*	*0.71*	*7.9*
15/09/75+	*15*	*5.55*	*20.1*	15	0.63	15.5
20/07/81	25	1.27	15.6	25	0.53	12.9
03/10/84	35	1.90	15.2	35	0.61	13.7
22/07/86	18	1.78	14.2	30	0.57	10.6
03/10/93	10	0.88	22.9	10	0.77	22.0
mean	22	1.46	17.0	23	0.62	14.9

* Event 13/5/74 was excluded from the mean for both models because the fit was poor.

+ Event 15/9/75 was excluded from the mean for the continuing loss because the fit was poor

Appendix C
Results using Australian Rainfall and Runoff Methodology

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Table C.1 Comparison of AR&R Peak Flows with FFA for AEP of 1 in 10

Catchment	Area (km^2)	FFA (m^3/s)	Peak Flow (m^3/s)	Difference (%)	Crit. Dur. (h)
Goodman Ck	32	69	59	-14	36
Ford R	56	57	102	79	36
Orroral R	90	50	99	98	48
Aire R	90	130	129	-1	36
Moonee Ck	91	29	67	131	72
Wanalta Ck	108	49	42	-14	18
Tarwin R	127	95	144	52	36
Lerderderg R	153	120	173	44	36
Moe R	214	44	194	341	36
Avon R	259	108	124	15	18
Seven Cks	332	257	468	82	48

Table C.2 Comparison of AR&R Peak Flows with FFA for AEP of 1 in 50

Catchment	Area (km^2)	FFA (m^3/s)	Peak Flow (m^3/s)	Difference (%)	Crit. Dur. (h)
Goodman Ck	32	126	98	-22	9
Ford River	56	116	147	27	36
Orroral River	90	99	173	75	24
Aire River	90	263	195	-26	36
Moonee Creek	91	55	100	82	18
Wanalta	108	111	94	-15	18
Tarwin River	127	161	236	47	36
Lerderderg	153	173	280	62	36
Moe R	214	59	369	525	36
Avon River	259	178	302	70	18
Seven Cks	332	549	697	27	72

Appendix D
New Areal Reduction Factors

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Table D.1 Areal Reduction Factors for an ARI of 2 years
(after Siriwardena and Weinmann, 1996)

area	Areal Reduction Factor for Duration (h)							
(km²)	1	2	3	6	12	18	24	48
1	0.95	0.96	0.97	0.97	0.98	0.99	1.00	1.00
5	0.91	0.92	0.93	0.94	0.96	0.96	0.98	1.00
10	0.89	0.90	0.91	0.93	0.94	0.95	0.96	0.99
50	0.82	0.85	0.86	0.88	0.90	0.92	0.93	0.97
100	0.79	0.82	0.83	0.86	0.88	0.90	0.92	0.95
500	0.69	0.73	0.75	0.79	0.83	0.85	0.88	0.92
1000	0.64	0.69	0.71	0.76	0.80	0.83	0.86	0.91
5000	0.49	0.56	0.59	0.66	0.72	0.76	0.80	0.87
10000	0.41	0.49	0.53	0.61	0.68	0.73	0.77	0.85

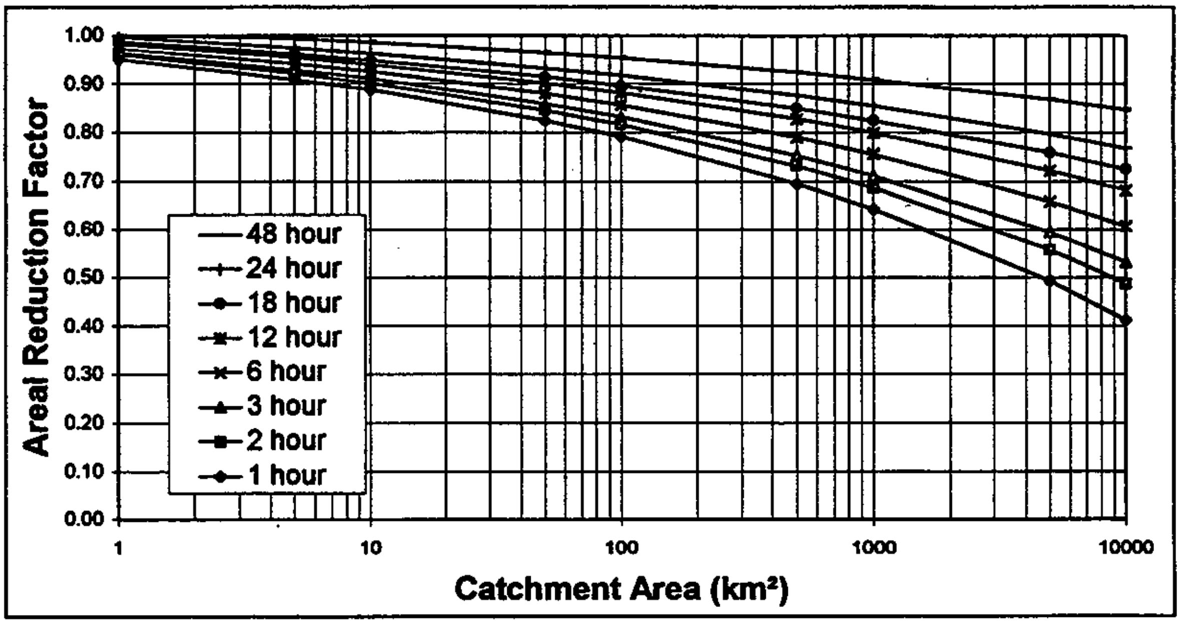

Figure D.1 Areal Reduction Factors for an ARI of 2 years
(after Siriwardena and Weinmann, 1996)

Table D.2 Effect of ARFs on Design Flood Peak Calculated with AR&R Losses and Temporal Patterns

Catchment	Area (km^2)	AEP of 1 in 10 peak flow (m^3/s) ARR ARFs	AEP of 1 in 10 peak flow (m^3/s) New ARFs	AEP of 1 in 10 Difference (%)	AEP of 1 in 50 peak flow (m^3/s) ARR ARFs	AEP of 1 in 50 peak flow (m^3/s) New ARFs	AEP of 1 in 50 Difference (%)
Goodman Ck	32	59	59	0	98	86	12
Ford River	56	102	102	0	147	139	5
Orroral River	90	99	96	3	173	161	7
Aire River	90	129	121	6	195	183	6
Moonee Ck	91	67	65	3	100	91	9
Wanalta	108	42	39	7	94	77	18
Tarwin River	127	144	135	6	236	220	7
Lerderderg	153	173	159	8	280	261	7
Moe River	214	194	179	8	369	336	9
Avon River	259	124	105	15	302	246	19
Seven Cks	332	468	447	4	697	660	5

Appendix D

Appendix E
Effect of New Design Losses

List of Tables

Table E.1 Peak Flow calculated using New IL/CL for an AEP of 1 in 10 (new ARFs)

			unfiltered			fully filtered		
Catchment	Area (km^2)	FFA (m^3/s)	peak flow (m^3/s)	difference (%)	Crit. Dur. (h)	peak flow (m^3/s)	difference (%)	Crit. Dur. (h)
Goodman Ck	32	69	64	-7	9	52	-25	9
Ford River	56	57	51	-11	36	36	-37	12
Orroral River	90	50	49	-2	24	31	-38	3
Aire River	90	130	55	-58	48	31	-76	12
Moonee Ck	91	29	32	10	24	28	-3	3
Wanalta	108	49	39	-20	3	39	-20	3
Tarwin River	127	95	94	-1	48	61	-36	12
Lerderderg	153	120	87	-28	12	72	-40	12
Moe River	214	44	95	116	36	78	77	12
Avon River	259	108	130	20	18	116	7	4.5
Seven Cks	332	257	306	19	24	189	-26	3

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