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DESIGN FLOOD ESTIMATION FOR SMALL CATCHMENTS IN NEW SOUTH WALES

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SYNOPSIS

A design procedure is described for flood estimation for rural catchments smaller than 250km^2 in eastern New South Wales using flood data from 284 catchments and the statistical interpretation of the Rational Method. While flood estimates for small rural catchments provide the primary design data for structures involving an overall average expenditure of about $180 million for the whole of Australia as at 1979, currently used procedures are generally based on judgment and experience. However, sufficient streamflow records are now available in many regions of Australia for the derivation of design procedures and data that will more accurately reflect actual flood characteristics.

The project described in this report had two aims:-

(a) to develop a practical design method for eastern New South Wales based on all available streamflow data; and

(b) to develop a methodology that could also be applied in other regions in Australia.

The approach using the statistical interpretation of the Rational Method was chosen as the derivation of data exactly matches the way in which the method is used in design, the approach is an efficient form of regional flood frequency analysis, and application of the method is simple and familiar to most designers.

The flood magnitudes of various return periods were obtained by frequency analysis of maximum monthly floods recorded on 284 catchments in eastern New South Wales and adjacent areas in Victoria and Queensland. Most catchments were smaller than 250km^2 and had at least ten years of records. The flood data were carefully checked to ensure the greatest possible accuracy of the data base. Rainfall intensities were obtained from the generalised procedures in the 1977 "Australian Rainfall and Runoff", which must also be used in application of the method. Critical rainfall duration was determined by a formula $t_c(h) = 0.76A^{0.38}$, derived from data from ninety-six catchments. Runoff coefficients for each return period were then derived as the ratio of flood magnitude to rainfall intensity. Ten-year coefficients C(10) were mapped for eastern New South Wales and contours of design values drawn. No significant relationships with other catchment characteristics could be found, and the scatter of the derived values from the design values given by the C(10) contours were shown to be consistent with the sampling errors inherent in the basic data. Average frequency factors were derived for each of three regions to enable evaluation of design runoff coefficients for return periods up to 100 years.

Use of the derived procedure was shown to provide much more accurate flood estimates than the method in "Australian Rainfall and Runoff", and should lead to average annual savings of approximately $12 million per year in structures on small catchments in eastern New South Wales.

For arid and semi-arid western New South Wales, insufficient streamflow data are available for the development of firm design relationships. However, an approximate design method was developed based on estimates of bankfull discharge at sixty-eight sites.

ACKNOWLEDGEMENTS

The project was carried out as Australian Water Resources Council Research Project 78/104. The financial assistance of the Department of National Development and Energy is gratefully acknowledged.

Release of the senior author by the Metropolitan Water Sewerage and Drainage Board, Sydney to work on the project for two years is also gratefully acknowledged.

Streamflow data used in the project were supplied by the Water Resources Commission of NSW, the Metropolitan Water Sewerage and Drainage Board, the NSW Soil Conservation Service, the State Rivers and Water Supply Commission of Victoria, and the Queensland Water Resources Commission. In particular, the project would not have been possible without the cooperation of the Water Resources Commission of NSW in supplying a large volume of data in a form suitable for analysis, for scheduling data processing to suit the requirements of the project, and for helpful comments and advice on many aspects of the streamflow data. The Bureau of Meteorology provided some of the rainfall data and also advice on various aspects of these data. Streamflow data from the catchments operated by The University of New South Wales were also used in the project, and the assistance of authorities that have helped to finance this catchment network is acknowledged, in particular the Forestry Commission of NSW, the Water Research Foundation of Australia, and the Rural Credits Development Fund of the Reserve Bank.

The Department of Main Roads, NSW, is thanked for the information on the capital costs and flow capacities of a selection of culverts and small bridges used in the economic analysis in Chapter 9.

Helpful discussions on the project were provided by the Reference Panel for the project and by Dr. S.N. Webb. Mr. B. Klaassen, who was secretary of the Reference Panel for much of the project, provided helpful comments on the draft report. The assistance is also acknowledged of various members of the Department of National Development and Energy, and of Mr. D.G. Doran and other colleagues in the School of Civil Engineering of The University of New South Wales.

Runoff coefficient data for the Canberra region discussed in Appendix O and incorporated into the design maps after the completion of the project were derived by Mr. J.F. Neal of the Department of Housing and Construction, Canberra, in an M.Eng.Sc. project at the University of New South Wales.

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1. BACKGROUND AND APPROACH

1.1 SCOPE

This report describes the development of a procedure for estimation of design floods for small ungauged rural catchments. Design data are derived for New South Wales, but the methodology could also be applied to other regions. The approach used is based on the statistical interpretation of the Rational Method of flood estimation. To define this study area more clearly the meaning and relevance of the terms 'design flood estimation', 'small catchment' and 'rural catchment' are discussed below.

1.1.1 Design Flood Estimation

This is the estimation of the magnitude of the peak flow rate at a site on a particular stream, which is expected to be equalled or exceeded an average number of times in a given time interval. The important word to note is 'average' in that it implies two fundamental assumptions when using design flood estimates to size a water-carrying structure.

These are:-

(a) the average frequency of occurrence will be the same in the future as in the period of observed flows; and

(b) the use of estimated flood magnitudes for average frequencies of occurrence will be a useful design criterion for achieving optimum economic and/or socio-political objectives in the sizing of particular hydraulic structures.

The necessity for these assumptions results from the random nature of the occurrence of extreme flood events. The actual occurrence of floods is highly irregular, and it is quite possible that two flood events with an average return period of say 50 years could occur in any one year, although the probability of this happening is low. The true average frequencies of occurrence of the statistical population of floods at a site will only be realised over a long period of time. If spatial correlation is absent or low, the frequency of floods actually occurring in a limited period of time over a number of catchments is more likely to approach the average or expected frequency than for a single catchment. In design for small catchments, the economic life of structures is limited and the problems of the random actual occurrence of floods cannot be avoided. Design based on average frequencies of occurrence estimated from observed data, as outlined in the above assumptions, is generally recognised as being the best practical approach.

1.1.2. Small catchments

Flood estimation methods such as the Rational Method are generally considered to be only applicable to small catchments. However, the real criterion is not catchment size, but the need to keep small the effects of factors not explicitly considered in the methods. The two main factors of this type are the volume of temporary storage of water on the catchment during the flood, and the variation of rainfall intensity in time and, to a lesser extent, with area. The variation of flood peaks with these factors is minimised when the catchment area is small, but they are also influenced by the slope of the catchment, the hydraulic roughness of the streams, the pattern of the stream network and other minor factors. The real criterion would lead to a small characteristic response time of floods on the catchment. However, an examination of response times in this project has shown that not all small catchments have low flood storage, and that some larger catchments have low storage.

Despite the effects of these other characteristics, catchment size has the greatest influence on flood response and has been generally accepted as the primary practical criterion. Something of a tradition has developed that small catchments are those with areas less than 25 km^2. The general limit of catchment size has been extended to 250 km^2 in this study for two reasons. Firstly, the statistical form of the Rational Method and other procedures used in this study take some account of the average effects of storage and non-uniformity of rainfall, although in a rather empirical fashion, and examination of the results has indicated no bias resulting from use of the larger catchments. Secondly, this was necessary to obtain suitable data from a sufficient number of catchments. A number of catchments larger than 250 km^2 have had to be used in the study in locations where no other suitable gauged catchment was available to indicate flood characteristics. However, it is recommended that the results of the study should be applied to catchments less than 250 km^2 in area.

1.1.3 Rural catchments

Catchments in virgin condition are rare, and the great majority have some form of man-introduced characteristics such as:-

- roads and tracks
- storages on the main streams
- small storages on the catchment (e.g. farm dams)
- urban areas
- industries (e.g. mining, electrical power generation)
- clearing and grazing
- agriculture
- forestry

Except where there are major water storages or where urban development covers an appreciable area of the catchment, the available evidence indicates that the effects of these introduced characteristics on major flood events is small (e.g. Baron, Pilgrim and Cordery 1980). Also clearing, grazing and agriculture, and even small storages and forestry, can be considered as typical of current rural conditions in a region. Rural catchments are therefore considered here to be all of those catchments where there are no large storages, appreciable areas of urban development, or other atypical land-use practices.

1.2 CURRENT PRACTICE AND EXPENDITURE

Pilgrim and Cordery (1980) present the results of a survey of the design flood estimation practices of local government authorities throughout Australia. Of the 216 rural respondents, the majority were from New South Wales and Victoria. The method used in the great majority (84%) of cases was the Rational Method, while those authorities not using this method base their design on observed or estimated flood levels or on judgment. Of the manuals of practice describing the use of some form of the Rational Method, the State main roads authorities' manuals were used by the largest number of respondents, while about half of that number used "Australian Rainfall and Runoff" (Institution of Engineers, Australia 1977, subsequently referred to as ARR). For New South Wales, both ARR and the manual of the New South Wales Department of Main Roads (1976) base design flood estimates on the general Rational Method formula:-

$$Q(Y) = F.C(Y).I(t_c,Y).A \qquad (1.1)$$

where
Q = peak flood flow rate (m^3/s)
Y = return period (years)
F = factor to adjust for the units used and is 0.278 for $Q(m^3/s)$, $I(mm/hr)$, $A(km^2)$
C = runoff coefficient (dimensionless)
A = area of catchment (km^2)
I = average rainfall intensity for a given return period and duration equal to the time of concentration 't_c': (mm/hr)

Wide variations can occur in the estimated values of the parameters and in the resulting flood estimates when using current design procedures with the Rational Method, or with other flood estimation procedures. Pilgrim (1978) and Cordery and Pilgrim (1980) illustrate these problems with examples of variations of fourfold and greater in runoff coefficients estimated for the one set of conditions by different manuals, and even greater variations in estimates of time of concentration using different formulae (see also French, Pilgrim and Laurenson 1974). Different experienced designers using the one manual were found to derive a wide range of flood estimates in a given situation, the maximum estimate being ten times larger than the lowest estimate. Added to these inconsistencies, the procedures used in practice for estimating runoff coefficients and times of concentration are generally based on judgment and not on observed data.

The likely errors involved in use of current design manuals is also illustrated by the comparison of estimates using ARR and observed flood data in this study (Section 9.1). Differences of 200% are common and can be as high as 1000%. Floods estimated from the various manuals tend to underestimate observed floods near the coast and to overestimate floods at inland locations in New South Wales.

The examples of Polin (1978) and Polin and Cordery (1979, 1980) and the data in this study (Section 9.2), indicate the considerable economic effects of the above errors and inconsistencies. This is of major importance on an overall national basis, as the average annual expenditure on works sized by design flood estimates on small rural catchments in Australia is about $180 million (Cordery and Pilgrim 1980). This is double the average expenditure on all major structures on larger catchments, and suggests that a large amount of public money is being spent inefficiently.

It is apparent that the present approach to the estimation of design floods on small rural catchments using a variety of handbook procedures is inadequate. There is a great need for the derivation of design values from observed flood data, specified in a manner that will lead to objective and consistent estimates by different designers.

1.3 AIM OF THE PROJECT

In view of the above considerations, the twofold aim of the project was:-

a) To derive a method of design flood estimation for small rural catchments in New South Wales for practical use, based on observed flood data; and

b) To develop a methodology for design flood estimation for small rural catchments that could be utilised in other regions of Australia.

Two major constraints were identified for the practical development of these objectives. The first was imposed by the availability of observed data. This applied particularly to flood data which were basic to the project. As it was desired to make the most efficient use of available data, and a large volume of design rainfall data was already published in ARR, it was decided to use an approach based on both rainfall and runoff data, such as the Rational Method, rather than a conventional regional flood frequency method based on runoff data alone.

The second constraint was that the method should be simple to apply and suited to the needs of the ordinary designer. Preferably, it should be similar in form to the familiar Rational Method to encourage its adoption in practice. It was also necessary to use only those forms of rainfall and catchment data that would be available to and easily extracted by the ordinary designer.

1.4 APPROACH TO DEVELOPMENT OF DESIGN METHOD

1.4.1 Statistical interpretation of the Rational Method

The basic approach adopted in this project was to use the statistical or probabilistic interpretation of the Rational Method, as used in a previous study by French et al. (1974). For a given catchment and return period of interest, the runoff coefficient C(Y) is found as the ratio of the flood peak of that return period derived from observed data to the design rainfall intensity of the same return period and the design duration. It is calculated by a rearrangement of equation (1.1):-

$$C(Y) = Q(Y)/(F.A.I(t,Y)) \qquad (1.2)$$

Runoff coefficients derived in this manner exactly correspond with the way in which they and the Rational Method itself are used in design practice. As noted earlier, the basic philosophy adopted in the project was to develop a method based on observed data that suited the needs of the designer and that would be relatively simple to apply in order to encourage its adoption in design practice.

The runoff coefficients derived by the above approach are not of a deterministic nature and should not be used for estimating the flood peak resulting from a specific rainfall. Rather, they represent the ratios of the flood and rainfall frequency curves at a given return period. Although the design method is in the form of the Rational Method, it is conceptually a type of regional flood frequency procedure with one of the independent predictor variables being average rainfall intensity for the particular location, design return period and critical duration.

The design duration of rainfall could be considered as the conventional time of concentration or simply as a standard response time of the catchment. In the statistical interpretation of the Rational Method, the physical meaning of the duration of the rainfall is not of fundamental importance, as long as it can be determined directly in the design situation and leads to consistent design values of the runoff coefficient. Five variations of the basic approach have been developed and tested in this project, using five different procedures for evaluating the design rainfall duration. Two other regional frequency procedures rather different to the statistical Rational Method have also been developed and tested. The recommended design method has been selected on the basis of the comparison of these seven procedures.

1.4.2 Reliability of flood magnitudes from observed data

The runoff coefficients derived using equation (1.2) are only as reliable as the input data used to calculate them, and hence are dependent on the reliability of both the flood frequency curves derived from observed data and the frequency curves of design rainfall. Of these, the reliability of the flood frequency curve is of primary importance in that its reproduction is the objective of the designer in using a design flood estimation method. Consequently, a deliberate decision was made at the start of the project that about half of the available time should be spent on obtaining flood data and flood frequency curves of the highest possible accuracy. In terms of practical results, refinement of analysis in deriving the design procedure could not compensate for deficiencies in the data.

1.4.3 Rainfall intensities

Equation (1.2) shows that the derived runoff coefficients are directly dependent on the rainfall intensities used in their derivation. If the designer is to reproduce the observed flood values, it is essential for the runoff coefficients to be derived by means of the same rainfall data as the designer will use in calculating the floods. In practice, local rainfall data are rarely available, particularly for small catchment design, and generalised data are used. For Australia, this is generally the data in ARR, as shown by the survey reported by Pilgrim and Cordery (1980). The rainfall data in ARR have therefore been used in deriving the design data in this report. However, several different types of rainfall data are given in that publication, and this problem is discussed further in Section 5.2.

There are several reasons for inconsistency in estimates of average rainfall intensity to be used for derivation of runoff coefficients at a given location, and these will affect the reliability and accuracy of the derived runoff coefficients. These are:

- response time of a given small catchment quite different to those of surrounding catchments;
- strong local rainfall gradients, so that the generalised rainfall data give a poor estimate of the rainfall on some small catchments;
- flood runoff produced on only part of the catchment so that the appropriate rainfall duration may be much shorter than that estimated by a general formula.

For the development of a design procedure for general use, it is unlikely that these inaccuracies can be explicitly allowed for. It is also unlikely that a designer would have detailed data available on these aspects, or that he would be able to use such data in ordinary design even if they were available. The effects of these inaccuracies on the developed design methods are discussed in general terms in Sections 5.5 and 8.1 and Appendix J.

1.4.4 Bankfull discharges in western New South Wales

While the design procedures outlined above could be developed for eastern New South Wales, this was not possible for the semi-arid and arid western half of the State where only short flow records are available from a few small catchments. Although the need for a design procedure is not as great in this region as in eastern New South Wales, there is still a need for design data. Accordingly, an approximate procedure based on estimated bankfull discharges was developed to indicate the magnitudes of flood events of a characteristic return period. This procedure is described in Chapter 11.

1.5 CATCHMENTS USED IN THE STUDY

Observed flood data were obtained for 290 suitable gauged catchments from the authorities listed in Table 1.1

Table 1.1: Authorities Operating Gauging Stations used in the Study.

Region	Authority	Number of Gauging Stations
Eastern New South Wales	Water Resources Commission of N.S.W.	215
	Metropolitan Water, Sewerage and Drainage Board,	7
	Snowy Mountains Hydro-Electric Authority	12
	University of New South Wales	13
	Soil Conservation Service of New South Wales	2
Near N.S.W. Border	QLD - Queensland Water Resources Commission	16
	VIC - State Rivers and Water Supply Commission	19
Arid Zone	NT - Dept. of Transport and Works	3
	University of New South Wales	1
	Water Resources Commission of N.S.W.	2
Total		290

The largest portion of these data was obtained from the Water Resources Commission of New South Wales. This authority greatly assisted the project by supplying the processed data in a form suitable for the analysis, and by its cooperation in scheduling of data processing to meet the needs of the project.

Of the total of 290 gauged catchments, 284 were used in developing the design procedure for eastern New South Wales, which is defined as the area between the coast and the line joining the towns of Mungindi, Nyngan, Condobolin, Narrandera and Tocumwal. All of the available catchments were used in developing the method described in this report to provide the best possible data base, rather than omitting some to provide an independent check on a procedure derived from a smaller data base. Most of the catchments (249) are actually located within the above area, but Table 1.1 indicates that sixteen are in Queensland close to the border of New South Wales, and nineteen in Victoria, although some of these are rather remote from the border. The remaining six gauged catchments lie in the arid zone, three being located in western New South Wales while three are near Alice Springs in the Northern Territory.

Fifty five of the 290 catchments were added late in the study and most of these fifty five have areas greater than 250 km^2. They were added to provide at least approximate data in regions where there was a deficiency of information from smaller catchments. The data for these additional catchments were extracted mainly from the report series 'Survey of Thirty Two River Valleys' (NSW Water Conservation and Irrigation Commission, various dates), and were not subjected to as rigorous analysis as those from the other catchments.

The distribution of catchment sizes for the 284 catchments in or adjacent to eastern New South Wales is presented in Figure 1.1. Less than 10% have areas less than 10 km^2. Figure 1.1 also shows that 91 catchments have areas greater than the desired upper limit of 250 km^2. As noted above, these catchments were used in regions where little or no other data were available to indicate general flood runoff potential.

Figure 1.2 presents the distribution of lengths of record of the 284 gauging stations, and shows that the great majority of stations have record lengths between 5 and 30 years. The greatest number is in the 10 to 15 years range. For this project a general lower limit of 10 years was adopted. However, where no other data were available, shorter record lengths (7 to 10 years) were accepted as providing indicators of flood magnitudes for the lower return periods up to 10 years.

A list of the 290 gauging stations used in the study is given in Appendix A, together with various characteristics of the catchments. Detailed information on each of the catchments, their physical and flood characteristics, and their analysed data, are given in the supplementary report to this project (McDermott and Pilgrim 1980). This report is of limited availability, but copies are held by libraries of most water-related authorities in Australia.

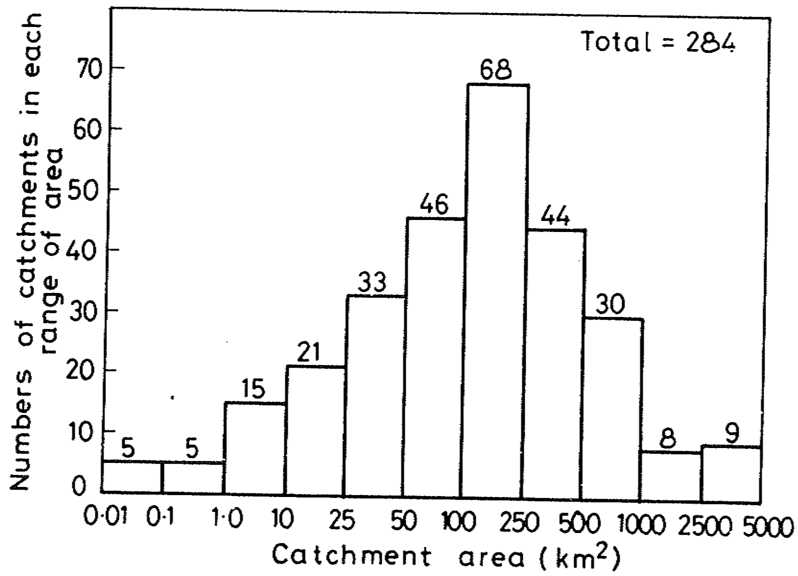

Fig. 1.1 Distribution of catchment sizes for gauged catchments used in study - Eastern NSW and adjacent regions.

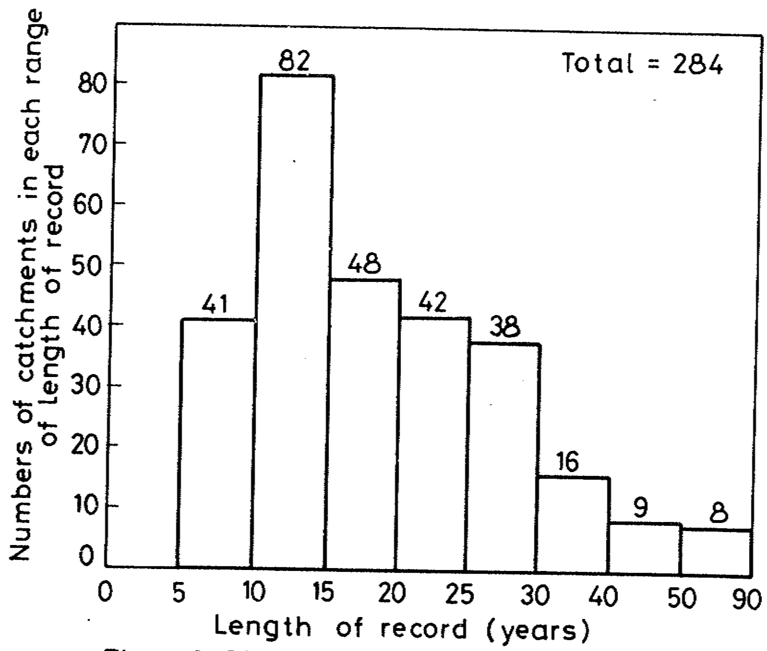

Fig. 1.2 Distribution of lengths of record for gauged catchments - Eastern NSW and adjacent regions.

2. FLOOD FLOW FREQUENCY

The primary importance of the accuracy of flood frequency estimates on the reliability of derived runoff coefficients, as discussed in Section 1.4.2, requires the careful consideration of decisions relating to flood data extraction, treatment and processing. With this in view the following aspects were investigated:-

- Use of the partial series as opposed to the annual series of flood peaks
- The time period or interval between events to adopt to ensure flood event independence where the partial series is used
- Methods for treating missing data
- A suitable probability distribution for fitting the partial series of flood peaks
- The number of events to use when fitting a probability distribution to data from a partial series
- The use of flood records from a daily read station.

Based on these investigations a procedure was formulated for the calculation of a flood frequency curve which was then applied to each gauged catchment. The reliability and suitability of the flood data were then re-examined with a view to defining a subjective reliability index for the flood frequency curve from each catchment.

2.1 TREATMENT OF DATA.

2.1.1 The importance of good data

Errors in the observed flood data obtained from the authorities listed in Table 1.1 or in the computed flood frequency curves would be directly reflected in the design method developed from these data. As a result of this primary importance of data quality, about half of the time available in the project was spent on careful investigation of the recorded flood data, as noted in Section 1.4.2.

Flood data are obtained from two types of stations; continuous water level recording stations and daily read (at 9 am.) stations. The latter of these two types is much less suitable than the former, in that some of the large peaks might have been missed, especially where the catchment is small and has a fast response time. These stations would be completely unsuitable if it were not for intermediate observations made by the gauge readers, which often provide the maximum water levels reached by the largest events.

Two forms of data were available from the authorities listed in Table 1.1. For the majority of the gauged catchments, data were provided in the form of summary printouts which gave the highest instantaneous peak flow rate recorded in each month of record, or in some cases the highest stage reached if the rating data were insufficient or unprocessed. Flood data for the remainder of the stations were available in the form of the original water level recorder charts or the manually-read records. Wherever possible anomalous data in printouts were checked using the original charts and any unprocessed recorder charts were computed.

2.1.2 Adoption of the partial series

From the continuous record of flow rates, two series of flood events can be extracted for use in flood frequency analysis. These are the annual series, consisting of the maximum instantaneous discharges for each year of record, and the partial series, consisting of all independent peak flows above some particular base magnitude regardless of the number of such floods occurring each year. The partial series was selected as appropriate for analysis in this project, based on three main considerations:-

a) The partial series gives better estimates of flood magnitudes for flows of return periods of 10 years and less. The theoretical difference is presented in Figure 2.1 but observed data differences can be considerably larger than this theoretical relationship indicates (Beard 1974).

b) The rainfall frequency data in ARR which have been used in the derivation of runoff coefficients for this project are effectively from partial series (Pierrehumbert 1972, 1974).

c) In practice, damage and inconvenience caused by surcharging of structures can occur more frequently than once per year.

2.1.3 Use of maximum monthly flows

To extract the partial series of flood events from a station record, the establishment of criteria for independence of events is required. The independence of one event from adjoining events preceding or following it is related to the response and recession times of floods on the catchment, and hence is related to the characteristics of the individual catchment. It also depends on the duration of the rainfall events causing the runoff.

The actual criteria for independence are thus complex and difficult to define in a manner which is unambiguous and covers all possible contingencies. Even by computer, it is also a long and laborious procedure to apply the criteria to a long streamflow record to extract the independent partial series events. Suitable criteria and their appication to small catchments have been discussed by Potter and Pilgrim (1971).

As a practical compromise based on the data forms available, the maximum peak flow rate occurring in each calendar month of record was extracted for each station. This means that where a number of large independent events occurred in the one month, only the largest one would be considered for the partial series. It was noted however, that monthly maximum peak flow rates that were high enough to warrant extraction were generally rare events, and as such were rarely accompanied by another large independent event in the same month. The largest flood in each month was also usually independent of floods in adjoining months. Computer printouts of daily flows were generally scanned to check this, and occasional events were discarded where it was apparent that events in adjoining months were not independent. In such cases, the next highest event in the particular month was adopted. A point of practical significance in support of the use of monthly maximum peaks is that in the event of damage caused by structure surcharge, there would most likely be a period of one month or more between the time of surcharge and the time of repair of any damage. Thus any other smaller flood occurring in this period would probably have a relatively small effect.

2.1.4 Use of the log Pearson Type III distribution

The events in each partial series were first ranked in order of magnitude and exceedance probabilities were than assigned to them by means of the California formula recommended in ARR:-

$$\text{Exceedance Probability} \quad P = m/(N+1) \tag{2.1}$$

where m = rank (highest = 1)
N = record length in years

In an N-year record the highest peak thus has a return period of (N+1) years. These values of flow rate and exceedance probability can then be plotted on appropriate axes. For the partial series however, no appropriate probability distribution and hence probability scale have been established. In general the partial series has been considered to fit an exponential distribution (Linsley, Kohler and Paulhus 1958, Todorovic 1978).

To reduce inconsistencies involved in fitting a flood frequency curve by eye, it was decided to standardise the fitting process by using an appropriate frequency distribution. The purpose in using this distribution was not to force an assumed parent distribution onto the data but to obtain a curve fitted objectively to the data. Various investigators (Benson 1968, U.S. Water Resources Council 1977, Cameron, Ward and Irish 1975) have found the log-Pearson type III distribution (subsequently referred to as LP3) to be more flexible and hence a more accurate curve-fitting technique than other recognised distributions.

Although normally used for the annual series, the LP3 distribution was here selected as a reliable and objective curve-fitting technique for the partial series. There are two methods for fitting the LP3 distribution, the maximum likelihood method and the method of moments. Due to the wider use of the method of moments (ARR 1977) and the availability of a computer program this method was chosen. The program was based on that developed by the U.S. Geological Survey (Thomas 1968). More detailed discussion of the mathematical formulation of the LP3 distribution is given by Matalas and Wallis (1973) and Boyd (1978). For practical purposes the means of application of the LP3 distribution to calculate flood magnitudes of a given return period has been arranged in the form below (ARR):-

$$X(Y) = M + K(Y).SDEV \tag{2.2}$$

If the logarithms (base 10) are used as inputs, then,

$X(Y)$ = the logarithm of flood event magnitude of return period Y years,

M = mean of the logarithms of the floods in the data series,

$SDEV$ = standard deviation of the logarithms of the floods in the data series,

$K(Y)$ = a multiplying factor which is a function of the skew coefficient (G) of the logarithms of the data series and the return period (Y) - see Harter (1969) and ARR.

The return periods for which flood estimates were required were 1, 2, 5, 10, 20 and 50 years, with consideration given to the 100-year return period where appropriate. The maximum return period for which a flood was estimated from the fitted LP3 frequency curves depended on the length of record. For record lengths of 14 years or less and 15 to 24 years, the maximum return periods were 10 and 20 years respectively. For record lengths greater than 25 years, the once in 50 year flood magnitude was calculated and the once in 100 year flood magnitude was estimated as a rough indication of the population value.

The Pearson type III distribution becomes the normal distribution when the skew coefficient equals zero. The distribution of the skew coefficients of the logarithms of the highest N peak flows for the 290 catchments is shown in Figure 2.2 The coefficients were positive for the majority of gauging station records and covered a wide range of values so that the LP3 distribution was more appropriate than the normal and Gumbel distributions which have zero and constant skew coefficients respectively.

Use of regional skew coefficients was not attempted due to skew coefficient dependence on record length 'N' (Kirby 1974). In addition Boyd (1978), using flood data from New South Wales stations, found large variability in skew coefficients and hence any possible regional tendencies could not be ascertained. For these reasons the skew coefficient as calculated from the observed flood data was used for the derivation of the flood frequency curve for each station.

2.1.5 Number of floods in each partial series

When fitting an LP3 distribution to an annual flood series, the number of events in the series is fixed at the number of years of record (N), and the mean, standard deviation and skew coefficient are defined unambiguously. With the partial series however, the number of events in the series is only determined by the base flow above which events are selected. The statistical parameters and hence the fitted distribution thus depend on this base flow and the number of events included in the series. An extensive investigation was carried out to determine the optimum number of events to be used in fitting the distribution. The investigation and its results are described in detail in Appendix B. It was found that when the number of events exceeded N, the number of years of record, the fitted curve diverged from the plotted positions of the highest floods, which are the values of greatest interest. It was therefore concluded that the highest N floods should be used in fitting the LP3 distribution to determine the flood frequency curve.

2.1.6 Treatment of missing data

Streamflow data frequently contain gaps during which records are missing for a variety of reasons. With the partial series, the manner in which these periods of missing data should be treated is unclear. A further extensive study of this problem was carried out, and is described in detail in Appendix C. The results are summarised below.

Seven different methods were investigated for treatment of missing data. These were tested by their ability to approximate the original series when several events were eliminated in a random manner from both recorded and synthetic data series. One of the methods tested was that recommended in ARR (page 108), where a whole year is eliminated from consideration if it is likely that a significant flood occurred in any period of missing data in that year. It is of interest that this method was significantly worse than any of the other procedures tested.

On the basis of the investigation, the policies recommended for the treatment of missing data at any particular station fall into the following three categories:-

Case 1: Where a nearby station record exists covering the missing period, and a good relation between the flood peaks on the two catchments can be obtained;

Policy - Use this relation and the nearby station record to fill in the missing events of interest.

Case 2: Where a nearby station record exists covering the missing period, and the relation between the flood peaks on the two catchments is such that only the occurrence of an event can be predicted but not its magnitude;

Policy - For record lengths less than 20 years, ignore the missing data and include the missing period in the overall period of record. For record lengths greater than 20 years, subtract an amount from each year with missing data proportional to the ratio of the number of peaks missed to the total number of ranked peaks in the year.

Case 3: Where no nearby station record exists covering the missing period, or where no relation between flood peaks on the catchment exists;

Policy - Ignore the missing data and include the missing period in the overall period of record.

These recommendations are dependent on the assumption adopted in the investigation that the occurrence of periods of missing data is of a random nature and is unrelated to the occurrence of flood events.

2.1.7 <u>Record length not a whole number of years</u>.

The length of record (N) for most stations was often not a whole number of years but a whole number plus a number of months, such that:-

$$N = N' + \frac{n}{12} \qquad (2.3)$$

where N = record length in years (real number)

N' = whole years of record (integer)

n = additional months of record (less than twelve)

From the results of the investigation of the treatment of missing data, it was decided that where there were six or more additional months of record ($n \geq 6$), the record length N should be taken as being equal to ($N' + 1$) and the missing (12 - n) months should be ignored, as long as N' was greater than or equal to ten years. Otherwise the record length was considered to equal the real number value given by equation (2.3). For these stations there was then the problem of how many whole data points (N or N') should be used in fitting the LP3 distribution to the partial series. A concept similar to that used by Jennings and Benson (1969) was used to construct the flood frequency curves for these stations. The procedure followed is set out below:-

(i) Calculate the flood frequency curve by fitting the LP3 distribution to the top $(N' + 1)$ flood events.

(ii) For particular flood magnitudes, calculate adjusted probabilities as

$$P = P' x \left[\frac{N' + 1}{N}\right] \tag{2.4}$$

where

P = adjusted exceedance probability

P' = exceedance probability calculated from (i).

(iii) Re-calculate return period appropriate to each flood magnitude;

$$Y = 1/P \tag{2.5}$$

where Y = return period in years, and
P is derived from equation (2.4)

2.1.8 Once in one year flood

The once in one year return period flood was adopted as the lower limit of interest to designers. On log-normal probability paper, however, the one year return period is not defined. As the flood magnitude of this return period was calculated mainly for lower limit definition and not for direct use in design, and to avoid replotting of the lower section of the flood frequency curve on semi-log paper, an approximate procedure was used. The approach utilizes the results derived by Shane and Lynn (1969) based on the compound Poisson distribution, that:-

$$Q(Y) = Q_{Nth} + (\bar{Q} - Q_{Nth}) \ln(Y) \tag{2.6}$$

where $Q(Y)$ = flood magnitude of return period Y years,

Q_{Nth} = the magnitude of the Nth ranking flood event,

$\bar{Q}$ = the mean of the highest N flood event magnitudes.

Substituting for Y equal to one year and Y equal to two years and rearranging gave the relationship below for calculation of the once in one year flood magnitude:-

$$Q_1 \doteq Q_2 \times \left[\frac{Q_{Nth}}{0.31\, Q_{Nth} + 0.69\bar{Q}}\right] \tag{2.7}$$

where Q_1 & Q_2 are the one and two year return period flood magnitudes respectively and Q_{Nth} and $\bar{Q}$ are as above for equation (2.6).

2.2 DATA PROCESSING AND EVALUATION

Before processing could begin the extensive data base of observed flows had to be obtained from the sources listed in Section 1.5. The efficient realisation of this primary step was achieved as a result of the help and cooperation extended by these authorities, in particular the New South Wales Water Resources Commission.

2.2.1 Data processing

Of the total 6260 years of station records processed, 3908 years were from continuously recording instruments and 2352 years were daily read records. For the continuous records 3657 years were in the form of printout summary listings and 251 years were in unprocessed chart form. Originally it was intended to reject all daily read records. However, this would have entailed a large reduction in the data base, and it was decided to consider the rejection criteria more closely.

A daily read record lists gauge heights taken at 9 am. each morning at a particular stream gauging site. After processing several daily read station records, study of these records revealed the following general points:-

a) For catchments larger than about 25 km^2, the highest stage reached by the largest events (larger than the once in one year event magnitude) have in the majority of cases been recorded in the form of intermediate readings.

b) For catchments smaller than 25 km^2, the rapidity of hydrograph rises and recessions makes the recording of intermediate heights unreliable and hence provides a basis for rejection of station record.

c) Where a large peak has occurred but no intermediate record of maximum stage reached is available, then this month should effectively be treated as missing data, and the policies recommended in Section 2.1.6 followed.

With the large number of intermediate flood stages recorded by gauge readers, it was decided to include stations with daily read records, treating flood periods with only 9 am. readings as missing records and examining the records from the smaller catchments carefully. With the inclusion of daily read stations there were three types of data used; continuously recorded, daily read and a mixture of the two. The methods used to process each of these types are described below.

a) Continuous records

For each station, the period of record was defined and the maximum monthly flows larger than a base magnitude were extracted, the base being selected so that more peaks than the number of years of record (N) were obtained. Missing periods were noted along with anomalous values, such as where the printout summary listed the average flow for the day as equal to the maximum peak flow rate reached by the event. The occurrence of this latter anomaly was relatively common and would have required the checking of a large number of original recorder charts. An average ratio of flood peak to daily average flow was calculated for such stations and used as a guide to judge which of these anomalous events would be likely to be high enough to rank in the partial series.

The original charts for only these events were then checked for the true peak, and in this manner the amount of chart checking was greatly reduced. The number of missing periods requiring checking was also reduced by using nearby station records (where available) to indicate whether peaks had actually occurred in the periods, but not their magnitudes. All of the relevant anomalies and missing periods were then checked using the original recorder charts and data filled in where appropriate. The remaining unexplained anomalies and gaps in the record were then treated as missing data as in Section 2.1.6. Peak flows were also extracted from the original charts for periods where the records had not been processed by the gauging authorities. The resultant partial series of flows was then ranked and the LP3 distribution fitted to obtain the flood frequency curve for the station.

b) Daily read records

For each station, the period of record was defined and all maximum monthly instantaneous peaks above an appropriate base magnitude were extracted. If the highest N peaks were all from intermediate readings then these data were ranked and the LP3 distribution fitted to obtain the flood frequency curve. If the proportion of intermediate readings in the highest N peaks was greater than 85%, the record length was taken as N years and these intermediate readings were taken as the data points and fitted with the LP3 distribution. The resulting exceedance probabilities corresponding to the particular event magnitudes were then reduced by the procedure of Jennings and Benson (1969), similar to that described in Section 2.1.7, to obtain the flood frequency curve. Where the proportion of intermediate readings in the N highest peaks was less than 85%, the record length was reduced by a proportional amount (as in the proportional reduction method of Appendix C) of the year in which the non-intermediate value occurred. That is, for each year in which one or more of these values occurred, the effective record length was considered to include only that proportion of the year equal to the proportion of the total large floods for the year for which intermediate readings were available.

c) Mixture of daily read and continuously recorded data

For each station, the lengths of daily read record (N_D) and continuous record (N_C) were determined. When N_D was larger than N_C, each type of record was treated separately and processed as in (a) and (b) above to obtain the ranked highest N_e peaks, where N_e is the sum of the two record lengths. Where N_C was larger than N_D, the highest N_C floods were first extracted from the continuous record. All floods larger than the smallest of these N_C values were then extracted from the daily read record, their number being denoted as n. The floods under consideration were then taken as the N_C floods from the continuous record and the n floods from the daily read record. The procedure then depended on whether n was greater or less than N_D. If n was greater, the total effective record length was adopted as $N_e = (N_C + N_D)$, and the N_e highest floods were analysed. Where n was less than N_D, the total effective record length was taken as $N_e = (N_C + n)$, and all $(N_C + n)$ floods were analysed. However, if intermediate readings were available for every flood in the daily read period, the record length was taken as $(N_C + N_D)$, as all floods in the entire period of record then would have been directly recorded.

2.2.2 Compilation of processed data.

Details of the data extracted for all of the catchments used in this project have been compiled in a companion volume to this report, 'Flood Data and Catchment Characteristics', (McDermott and Pilgrim 1980). For each catchment, this gives information on the flow record and lists the extracted floods forming the partial series, the statistical parameters of the flood series used to fit the LP3 distribution, flood magnitudes of various return periods estimated from the fitted distribution, and various physical characteristics of the catchment.

2.2.3 Evaluation of reliability of the data

Some degree of error is inevitably involved in all flow data, and the errors in flood magnitudes on small catchments may be large. Primary causes of these errors are listed below, together with indicators used in this study to gauge their likely importance.

(i) Sampling error; that is the sample of flood flows is not representative of the population of flood flows.
Indicator - record length.

(ii) Rating curve extension error; for example where the extension of the rating curve required for the larger floods is over more than one log-cycle.
Indicator - amount of extension required.

(iii) Missing events; errors are likely where a high percentage of any type of record is missing, even with continiously recorded data. Where only daily readings are available, it is likely that some events will be missed, especially if intermediate readings have not been recorded.
Indicator - amounts of missing record and daily read records.

In addition to these errors in the flood data, inclusion of some of the catchments in the study may be of questionable validity as a result of:-

(iv) Presence of man-introduced features which could appreciably affect flood peaks, as discussed in Section 1.1.3.
Indicators - the presence of such features and their relative extents.

(v) Size of catchment; areas greater than 250 km^2 were considered to be undesirable if data were available from smaller catchments.
Indicator - size of catchment.

However, the fact that the above indicators are present does not necessarily mean that the data from a catchment actually contain large errors or are unsuitable. It is not possible to determine the true magnitude of the errors, and the best that can be achieved is a rather subjective assessment of reliabiity. It was therefore decided that no station record should be rejected outright as unsuitable, but that a reliability index should be devised based on the above indicators. The assigned value of the index could then be used in examining the relation of derived runoff coefficients to catchment location and characteristics, and in later investigation of possible reasons for outlier values (Appendix K).

The procedure used to assign a reliability index to the flood frequency curve for each station is described in Appendix D. The index was based on scores relative to a catchment of maximum reliability which was assumed to have a 50 year record of flow and no other error indicators. The reliability index values were arranged in five class intervals, which are shown for each catchment in Appendix A and on Map 1 at the back of this report.

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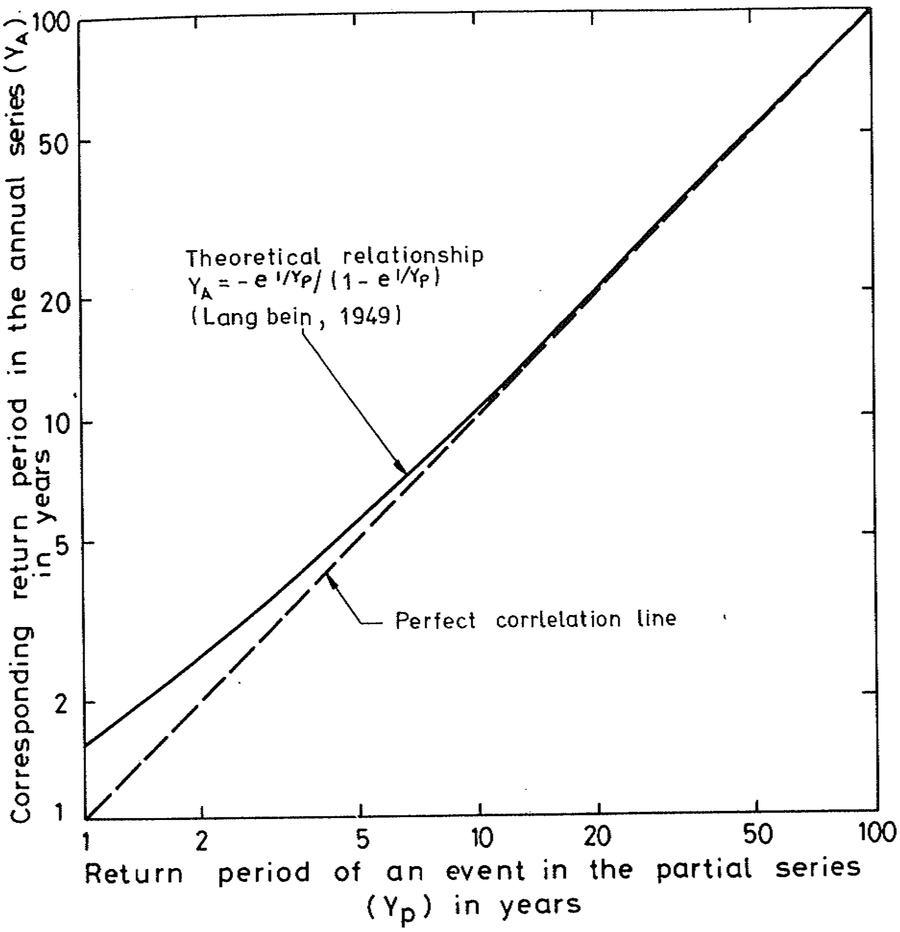


Figure 2.1 Theoretical relationship between statistics of annual and partial series.

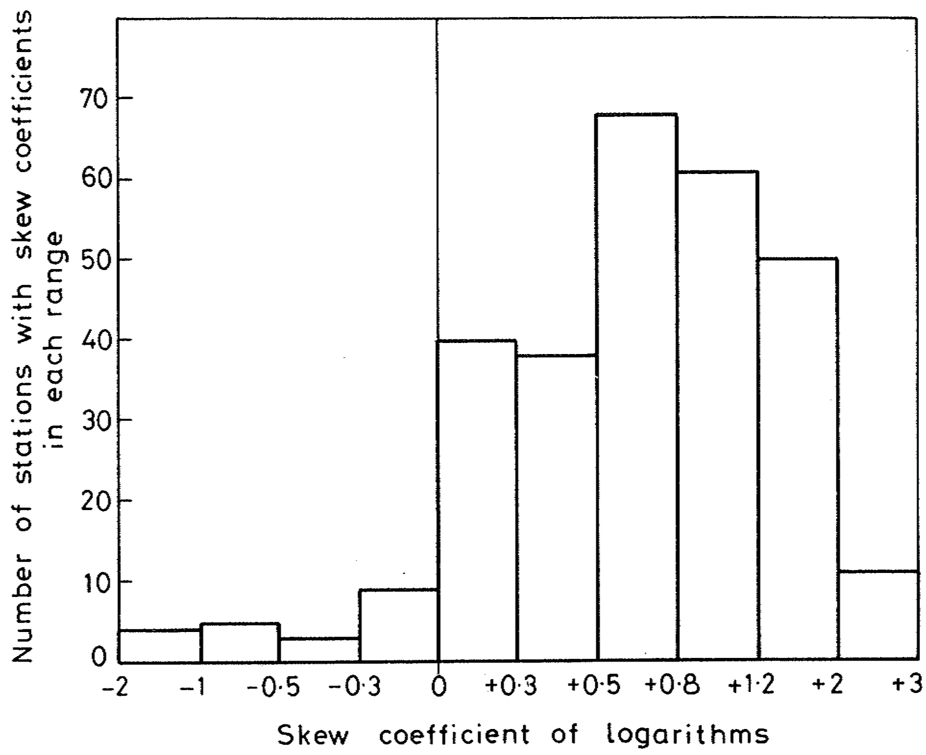


Figure 2.2 Distribution of the skew coefficients of the logarithms to base 10 of the highest N floods for each of the 290 catchments.

3. CATCHMENT CHARACTERISTICS

3.1 IMPORTANCE OF CATCHMENT CHARACTERISTICS

The physical characteristics of a catchment have an obvious effect on the floods occurring on it. Not only do the characteristics affect the floods, but the flood regime of the catchment is largely instrumental in shaping its form and characteristics, so that there is a fundamental interrelationship between the two. While the relation is obvious in the deterministic approach of considering floods resulting from particular rainfall events, it also applies to the probabilistic approach adopted here of relating floods and rainfalls of the same frequencies.

Measures of catchment characteristics are required in the following three aspects of a design flood estimation procedure in the form of the Rational Method:

- area is used directly in the formula;
- estimation of the duration of the design rainfall, whether it is considered as the time of concentration or simply as a characteristic response time parameter;
- evaluation of the runoff coefficient for a catchment, or its deviation from some regional average value.

In view of their importance to the development of a design procedure, considerable time was spent on the extraction of measures of catchment characteristics, and on investigating the relationship of the varicus characteristics with one another.

3.2 DESCRIPTION OF CATCHMENT CHARACTERISTICS

The measures considered, and for which values were extracted, were limited to those that could be readily obtained from available maps. Not only was this necessary because of the limited time available for the project, but also because any measures used should be able to be easily derived by the designer. The characteristics extracted can be divided into natural and introduced groupings. The former were used directly in developing design relationsnips, while the latter were only used in assigning relative reliabilities to the flood data and the derived runoff coefficients.

3.2.1 Natural characteristics

(a) Area (A km^2) - the total plan area contributing to flow at a site.

(b) Stream length (L km) - the length of the main stream measured around the channel sinuosities from the site to the catchment divide. The main stream above a confluence is generally that draining the larger area.

(c) Stream slope (S m/km) - two measures of stream slope were extracted, as illustrated in Figure 3.1:

- Average Slope (S_a), the total elevation differnce from source to outlet site of the main stream divided by L; and
- Equal Area Slope (S_e), the slope of a line drawn through the outlet site and intersecting the longitudinal profile of the main stream such that the area enclosed above the profile is equal to that below the profile.

The latter slope should be more representative of the general stream slope and more closely related to travel times of floods, and was thus adopted as the primary measure of stream slope in this study, although both measures were used in investigations.

(d) Perimeter (P, km) - the length of the smoothed catchment boundary, following only general changes in direction and not the small-scale sinuosities.

(e) Median annual rainfall (MAR,mm).

(f) Soil type.

(g) Vegetation cover.

3.2.2 Introduced characteristics

(a) Storages on main streams.

(b) Small storages on remainder of catchment.

(c) Percentage of urbanisation.

(d) Cultivation and grazing.

(e) Industry.

3.3 DATA SOURCES

Soil and vegetation cover types were extracted from 'Atlas of Australian Resources' (Department of National Resources 1977) and are only general classifications. Median annual rainfalls for the New South Wales catchments were obtained from the report series 'Survey of Thirty Two River Valleys' (NSW Water Conservation and Irrigation Commission, various dates), while those for adjoining areas in other states were obtained from 'Atlas of Australian Resources'. Other characteristics were extracted from State Mapping Authority maps. Some additional information on introduced characteristics was provided by personnel of the N.S.W. Water Resources Commission.

The available State Mapping Authority map scales were such that no one scale (except 1:250 000) covered the whole of N.S.W. As at 1980, the coverage available is:-

1:25 000 & 1:31 680-From the NSW coast to the line joining Tenterfield, Boggabri, Bathurst, Goublurn, Cobargo, Bombala. Also, a finger from Ulladulla to Wagga Wagga.

1:50 000(& 1:63 360)-From the 1:25000 western boundary to the line joining Collarenebri, Gilgandra, Lake Cargelligo, Euston.

1:100000 - The remainder of the State.

The maps are available in either printed form on request, in provisional form, or not yet completed. Of the 1:25 000 & 1:31 680 maps, the vast majority are available in final printed form. Of the 1:50 000 maps about half are available in final printed form, while for the 1:100000 maps about one fifth are available in final printed form. Most of the remaining maps are available in provisional form excepting the section of the State near the Queensland border. For the few very small catchments used in this study, the authorities involved were able to provide more detailed coverage (scales ranging from 1:4000 to 1:12 000).

3.4 DATA EXTRACTION

Latitude and longitude co-ordinates were obtained for each station from Gauging Stations Annual Returns of the N.S.W. Water Resources Commission and from 'Stream Gauging Information, Australia, 1974', (Department of National Resources 1976), and were checked on the largest scale map available. Catchment areas quoted in the latter document were adopted after checking of a large sample of catchments by planimetering of areas from the largest scale map available. Stream lengths were measured from the maps and profiles plotted for each catchment for determination of the two slope measures. Other catchment characteristics listed in Section 3.2 were determined from the relevant maps, and tracings made of catchment shapes and mainstream positions for general reference.

Values of various characteristics of each of the 290 catchments used in the study are listed in the supplementary report to the project 'Flood Data and Catchment Characteristics' (McDermott and Pilgrim 1980).

3.5 PROBLEMS WITH CATCHMENT DATA

3.5.1 Inconsistency in detail of data

The variety of map scales and production dates meant that the extraction of introduced characteristics was not always reliable and consistent. For example less detailed map scales (1:100 000 & 1:250 000) do not show cultivated areas while older maps might not show storages which have been constructed since the map was produced. As a result, not all of the introduced characteristics of relevance may have been extracted for each catchment.

Detailed local data on soil type, vegetation cover and median annual rainfall were not available for each particular catchment, so that the types and values extracted from general maps represent gross classifications only. Different survey methods were used to produce the older maps (land survey) compared with newer maps (photogrammetry). The latter is considered to be more accurate over large areas. The effects of any resultant differences on the results of this project, however, are considered to be negligible.

3.5.2 Definition of flat catchments

In areas of low slope, such as western New South Wales and with small scale maps, (1:100 000), there is often a definition problem regarding catchment boundaries and longitudinal profiles. For example there might only be one or even no contour crossing the total length of the main stream. For these stations, slopes were calculated by using the average length between the two most appropriate contours in the area.

3.5.3 Waterfalls

Waterfalls occur in the stream profiles of three of the catchments. As the number was small, the calculated slopes were not adjusted, but the catchments were noted for later checking of the consistency of their derived runoff coefficients. In design, it seems preferable to vertically transfer the segment of the stream and catchment above the waterfall to give a continuous profile, as the waterfall does not affect the flood travel time.

3.5.4 Different map scales

The derived values of stream length and slope for a catchment will depend on the map scale used. This results from the increasing sinuosity of the stream shown on maps of increasing scales, leading to a systematic inconsistency. Large scale maps will give increased lengths and hence decreased slopes, with resulting increases in travel time and decrease in design rainfall intensity. This would in turn lead to differences or inconsistencies in derived values of runoff coefficients and in estimated floods in design. The maximum increase in stream length observed in this study was 80 per cent for a 1: 25 000 map compared with a 1: 250 000 map.

The effect of map scale on measured elevation differences was also examined. Differences in the vertical distance from stream source to outlet site were found to be less than the contour interval of the smaller scale map, and to result from lack of definition of elevation. The elevation differences are thus random rather than systematic as with stream length, and are not amendable to adjustment for scale effects.

3.6 ADJUSTMENT TO STANDARD MAP SCALE

To reduce or eliminate the systematic inconsistencies discussed in Section 3.5.4 above, adjustment factors were derived to convert stream lengths and slopes to values corresponding to a standard map scale. This was chosen as 1:25 000, as it is the largest scale with a wide coverage of maps, and there will be an increasing availability of maps of this scale in the future. From maps of different scales, sets of measurements of stream length were made for 268 catchments. The scales available were 1:25000, 1:50 000, 1:63 360, 1:100 000 and 1:250 000. Plotting of the data showed the relationship between the lengths from the maps of different scales to be of the form:-

$$L_{sc1} = MF \times L_{sc2} \tag{3.1}$$

where L_{sc1} = stream length measured from map of larger scale 1:SC1

L_{sc2} = stream length measured from map of smaller scale 1:SC2

and MF = multiplying factor.

The multiplying factor MF was found, from regression, to be related to the scale ratio SR:-

$$MF = 1.031 + 0.170 \log_{10} (SR) \tag{3.2}$$

where $SR = SC2/SC1$

The correlation coefficient (r) obtained for this equation was 0.40, with a standard error of estimate of 0.10. Although these statistics indicate that the relation is significant at less than the 0.1% probability level, considerable scatter was evident in the data. A more logical and realistic formula very similar to equation (3.2) would incorporate a constant term of 1.0 so that the multiplying factor MF would be equal to 1 when the scale ratio SR was equal to 1 (i.e. taken from the same map). A best fit equation of this type adopted for use in this project was:-

$$MF = 1 + 0.21 \log_{10} (SR) \tag{3.3}$$

A trend of higher multiplying factors for the flatter catchments was evident, probably as a result of greater average sinuosity of streams on flat catchments. A multiple regression for MF including equal area slopes (S_e) gave the relation:-

$$MF = 1.072 + 0.175 \log_{10} SR - 0.049 \log_{10} S_e \qquad (3.4)$$

Where a measure of stream slope is available, Table 3.1 lists values of the multiplying factor MF to convert stream lengths from maps of different scales to the value corresponding to the standard scale of 1:25 000, as calculated from equation (3.4)

Table 3.1 Multiplying Factors for Calculation of Standardised (1:25 000) Stream Length, using Equal Area Stream Slopes.

Map Scale Used to Measure L	Multiplying Factor for 1:25 000 length calculation, for stream slopes S_e (m/km) as below.					
	1	3	7	10	50	100
1:250 000	1.25	1.22	1.21	1.20	1.16	1.15
1:100 000	1.18	1.15	1.14	1.13	1.09	1.08
1:63 360	1.14	1.12	1.10	1.09	1.06	1.04
1:50 000	1.12	1.10	1.08	1.08	1.04	1.03

Where no measure of stream slope is available, values of the multiplying factor calculated from equation (3.3) are listed in Table 3.2.

Table 3.2 Multiplying Factors for Calculation of Standardised (1:25 000) Stream Length - Neglecting Stream Slope

Map Scale Used to Measure L	Multiplying Factors for 1:25 000 Length Standardisation
1:250 000	1.21
1:100 000	1.13
1:63 360	1.10
1:50 000	1.08

Although consideration of slope in equation (3.4) and Table 3.1 reduces the scatter, the coefficient of multiple determination of this relation is still only 0.43. However, the correlation is significant at the 0.1% probability level.

All stream lengths in this project have been adjusted to the standard 1:25 000 scale by means of equation (3.4) together with (3.1). In application of any derived design relations involving stream lengths, measured values should also be adjusted by equation (3.1) and preferably (3.4), or alternatively (3.3) if slope is not available. Measured slopes should also be adjusted by the reciprocal of MF. In using equation (3.4) to adjust the measured length and slope, MF depends on the slope S_e so that ideally, an iterative solution would be required. However, this would not be justified in practice, and a single application of the multiplying factor would be satisfactory.

3.7 RELATIONSHIPS BETWEEN CATCHMENT CHARACTERISTICS

Previous studies from around the world, and particularly in the United States, have shown that a high correlation exists between stream length and catchment area, and to lesser degree between stream slope and area or stream length. These relations are of interest in themselves, but they also indicate that regressions for design rainfall duration or any other dependent variable using more than one of these characteristics as 'independent' variables are not really valid. As a large amount of catchment characteristic data had been extracted during the project, its availability allowed the derivation of relationships between characteristics for the catchments in New South Wales and adjacent areas. The 429 catchments for which the characteristics were extracted were divided into the general geographical regions of Coastal, Dividing Range (including 63 catchments in the Snowy region), Western Slopes and Western Plains. Regressions were calculated for stream length on catchment area, stream slope on length, and slope on area. The derived relations are shown in Figures 3.2, 3.3 and 3.4, and details of the relations and correlations are given in Table 3.3

The strongest and best-known relationship of catchment characteristics is between area and length. Hack (1957), Gray (1961) and Leopold, Wolman and Miller (1964) all derived relations in which mainstream length is a function of area to a power of approximately 0.6. Using a stratified sample of 250 catchments with areas of 0.3 km^2 to 8 x 10^6 km^2 selected from several thousand sets of data, Mueller (1973) derived a similar relation with an exponent of 0.55. Meynink (1975) obtained an exponent 0.58 for 76 small Australian catchments. Shreve (1974) claimed that the relation tended to vary a little with area, and that the exponent tended to decrease from 0.6 to 0.5 as catchment area increased. Shreve and also Werner and Smart (1973) were able to verify the form of the relation between mainstream length and area from channel network theory.

The regional relationships derived here and shown in Figure 3.2 are very similar to one another and to the published relations cited above, with exponents of 0.57 - 0.58 for most regions and 0.54 for the Western Plains. The correlation coefficients are also high (0.97 - 0.99) as found for the published relations. This indicates that the stable relationship between area and length found in other parts of the world applies equally to different types of regions in Australia.

In previous studies, it has also been shown that over a given region, stream slope is inversely related to stream length, and hence also to catchment area, but that the relation varies from one region to another. Gray (1961) derived relationships for three different regions in the U.S.A. Flint (1974) found that slopes of individual stream segments within a catchment were related to variables which themselves are related to area, although Onesti and Miller (1974) found that this type of relation did not apply to very small areas with stream orders below four. In this case, geology and others factors controlled slope. However, it seems that within a fairly homogeneous region, slope of individual stream segments is related to drainage area. As overland slope has been found to be correlated with stream slope (Strahler 1964), this is also related to area.

The regional relationships derived here and shown in Figures 3.3 and 3.4 confirm the results cited above. Relationships were derived for each region, and although the correlation coefficients (0.66 - 0.84) were not as high as for the regression of length on area, they are all significant at the 0.1% probability level. Different relations apply to the different regions, and as expected, predicted slopes decrease from the mountainous regions to the plains. This dependence of slope on region supports the concept utilised later in this study that runoff coefficients can be related to the location of the catchment.

Table 3.3 Derived Relationships of Catchment Characteristics - 429 Catchments.

Key: A - catchment area (km^2)
L - mainstream length (km), standardised for 1:25 000 map scale
S_e - equal area slope of main stream (m/km), standardised for 1:25 000 map scale
r - correlation coefficient.

Relationships from regression analysis

Region	Number of Catchments	Length vs Area $L=k\,A^n$			Slope vs Length $S_e = kL^n$			Slope vs Area $S_e = kA^n$		
		k	n	r	k	n	r	k	n	r
Coastal	140	1.77	.58	.97	62	-.63	.66	41	-.35	.63
Dividing Range	120	1.70	.57	.99	94	-.68	.73	66	-.40	.73
Western Slopes	95	1.74	.57	.97	83	-.72	.68	66	-.44	.71
Western Plains	74	1.80	.54	.98	19	-.56	.84	14	-.31	.84
Combined (mostly NSW)	429	1.78	.57	.98	45	-.56	.68	34	-.32	.67
Snowy	63	1.75	.57	.98	138	-.64	.81	-	-	-

(Note: The 63 Snowy Catchments have also been included in the 'Dividing Range' region in the upper part of the table).

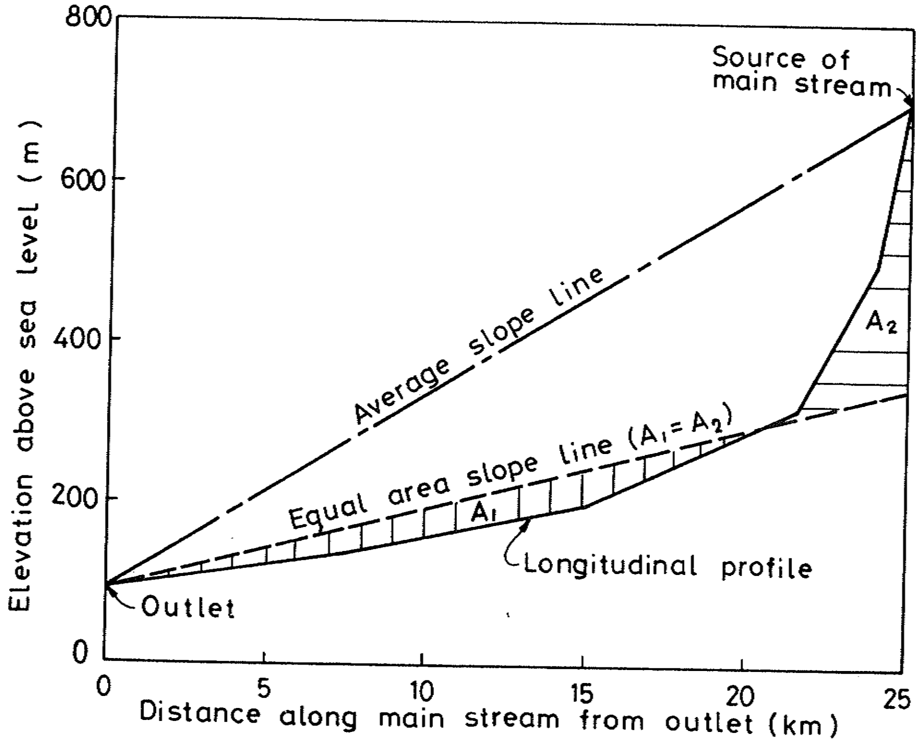
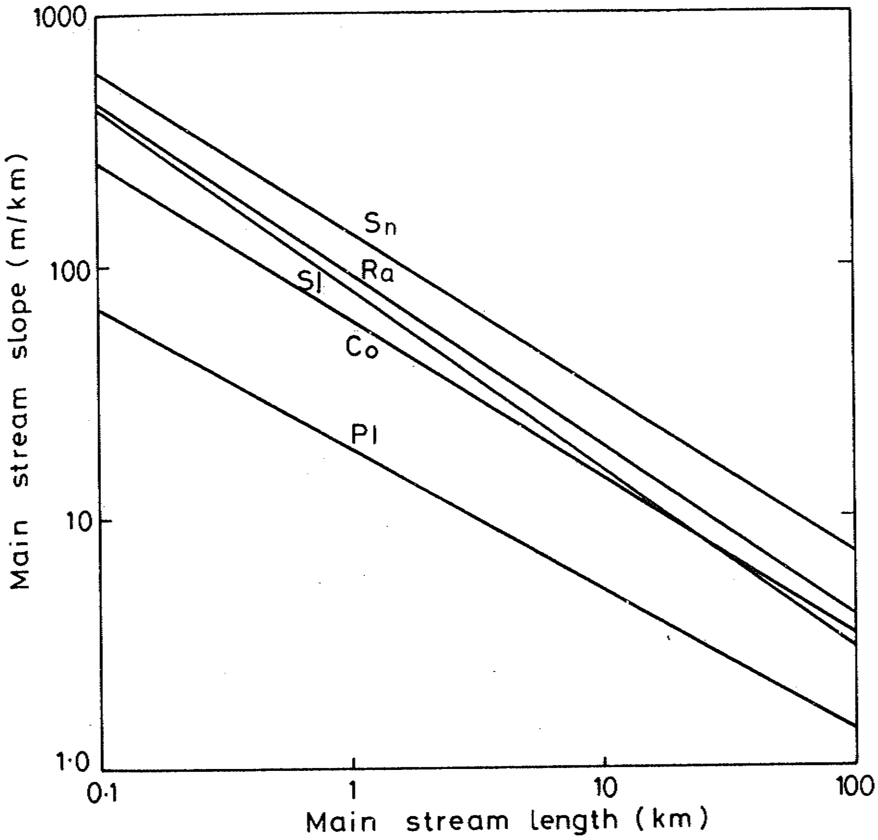


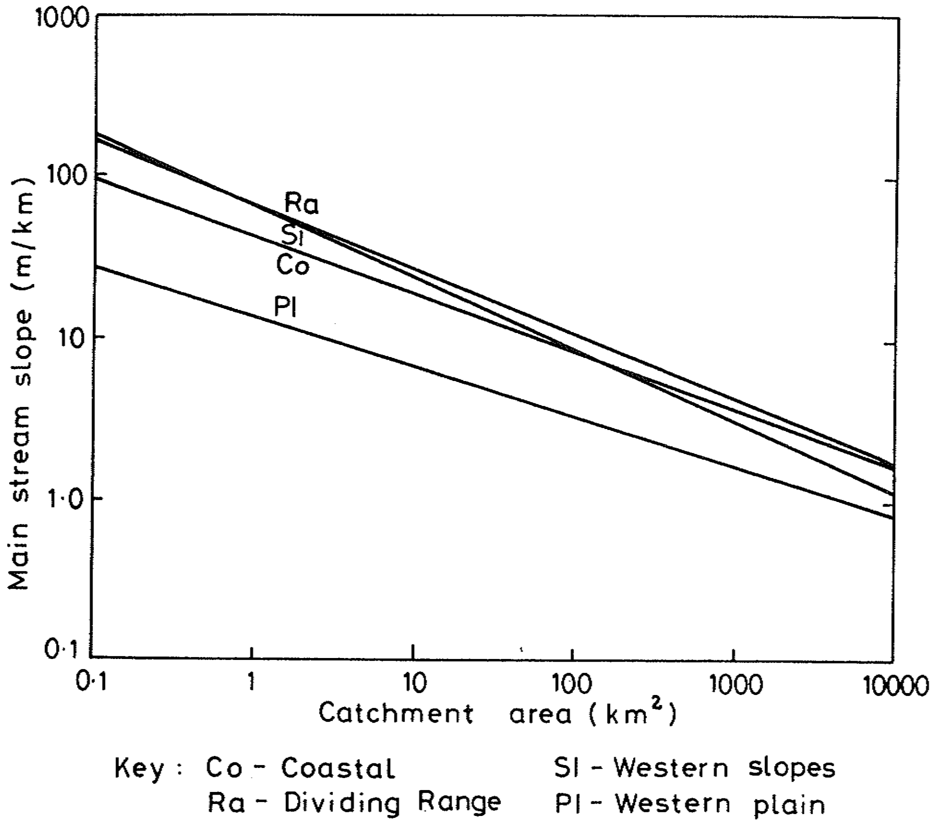
Figure 3.1 Longitudinal profile of main stream showing average slope S_a and equal area slope S_e



Note: The 63 Snowy catchments have also been included in the 'Dividing Range' region.

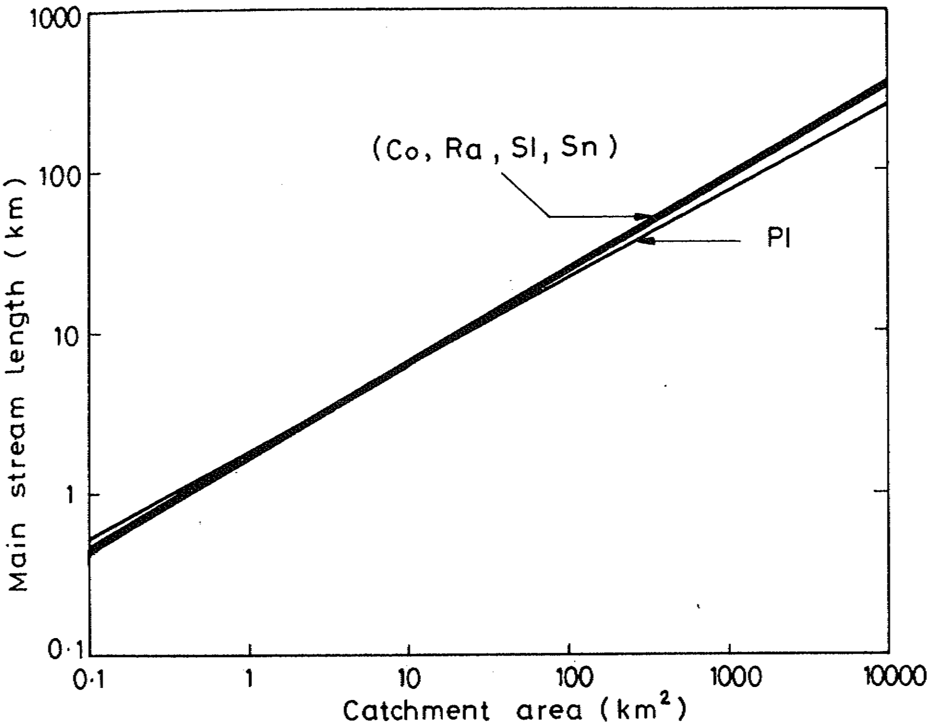
Figure 3.2 Regional relationships between length of main stream and catchment area (map scale 1:25000).

Key: Co - Coastal, Sl - Western slopes, Ra - Dividing range, Pl - Western plains, Sn - Snowy



Note: The 63 Snowy catchments have also been included in the 'Dividing Range' region.

Figure 3.3 Regional relationships between equal area slope and length of main stream (map scale 1:25000)



Key : Co- Coastal , Sl - Western slopes
Ra- Dividing range, Pl- Western plains
Sn - Snowy

Note: The 63 Snowy catchments have been included in the 'Dividing Range' region.

Figure 3.4 Regional relationships between equal area slope of main stream and catchment area (map scale 1:25000).

4. DESIGN DURATION OF RAINFALL

4.1 TIME OF CONCENTRATION AND CRITICAL DURATION

One of the assumptions made in the Rational Method is that the design duration of the rainfall is equal to the time of concentration of the catchment. Traditionally, time of concentration has been considered to be the maximum time taken by water to travel from the catchment boundary to the catchment outlet. This time measure is assumed to be the 'critical duration', meaning that the use of any other duration would result in a lower flood estimate (see, for example, Cordery and Meynink 1976). This implies that the use of a shorter duration, although giving a higher design intensity of rainfall, does not allow the whole catchment area to contribute simultaneously to flow at the outlet, while the use of a longer duration allows the whole area to contribute but with a lower intensity. In design practice, the time of concentration must be estimated by means of an empirical formula or set of recommended values. The Bransby Williams (1922) formula is recommended by ARR, on the basis of the results of French et al. (1974), and is discussed further in Section 4.3.

As noted in Section 1.4.1, an alternative concept is to regard the duration of rainfall as being simply a characteristic response time of the catchment. No assumption regarding the time of travel of water from the most remote point or any other location is necessarily involved. This approach has been used in a design flood procedure by Reich and Hiemstra (1965). The only requirement of such a measure would be that it would lead to consistent derived values of the runoff coefficient in the development of a design method and consistent flood estimates in application of the method in design. The original intention in this project was to adopt this characteristic response time approach. However, it became necessary to develop a formula from values of time of concentration estimated from observed floods, as described later in this chapter.

A point which has caused confusion in practice is whether the time of concentration formulae used in design give the total rainfall duration, or whether they only give channel flow time to which an overland flow time must then be added. In a survey of the design practice for flood estimation of Australian local government authorities, Pilgrim and Cordery (1980) found that the 216 respondents were almost equally divided as to whether an overland flow time should be added. In this study, all formulae for time of concentration or response time are considered to give the total rainfall duration, and no overland flow time has been or should be added.

4.2 CATCHMENT CHARACTERISTICS FOR USE IN FORMULAE

The common characteristics that have been used in formulae for time of concentration are catchment area (A), length of main stream (L) from the catchment divide to the outlet, and stream slope (S). Other catchment characteristics have been used in some formulae, but it is difficult to obtain satisfactory practical measures of them. As discussed later in Section 4.4.2 and Appendix F, several characteristics in addition to A, L and S were used in this study. These were a catchment shape factor, a stream slope nonuniformity index, a vegetation cover roughness index, median annual rainfall, and soil type.

Two other characteristics that would be expected to affect average velocities and times of travel are the form of the channel network and the hydraulic roughness of the channels. Suitable simple and unique measures of the form of the channel network are not available, and most that have been proposed are related to area (Onesti and Miller 1978). Mannings n would directly affect the time of travel, but its value is very difficult to estimate for natural streams. Cordery (1968) used n in design relations for synthesis of unit hydrographs for eastern New South Wales, but omitted it in a later version of the procedure (Cordery and Webb 1974). The vegetation cover roughness index noted above was used to give some indication of resistance to flow, but in general it was not possible to consider the channel network and channel roughness in the study as simple and unambiguous measures are not available.

Stream slope presents a practical problem in that it can be evaluated in several different ways, as discussed in Section 3.2.1. The simplest is the average slope S_a defined as the difference in elevation between the source of the main stream and the outlet, divided by the stream length L. This was the type of slope used in the Bransby Williams (1922) formula, although this was not stated in ARR and has led to considerable confusion. Actually, Bransby Williams used the direct length from source to outlet rather than the stream length. Quite different stream profiles can have the same average slope S_a, and thus this does not seem to be a good measure for estimating travel times. The equivalent area slope S_e (Section 3.2.1) has been adopted in this study as a satisfactory compromise between a realistic measure of effective slope and ease of derivation. It has been widely used in practice.

4.3 ASSESSMENT OF EXISTING FORMULAE

French et al. (1974) tested eight formulae for estimating time of concentration in a previous study of the Rational Method in eastern New South Wales. This showed that all formulae were of low accuracy, and that many gave biased results. The Friend (Department of Main Roads, N.S.W. 1976) and Bransby Williams (1922)formulae gave the least biased results on the average. However, the former is complicated, and different designers can obtain widely different results. On the basis of these findings, ARR recommended the Bransby Williams formula for use in design. This is

$$t_c = \frac{0.975\ L}{A^{0.1}\ S^{0.2}} \qquad (4.1)$$

where t_c = time of concentration (hours)

L = stream length (km)

A = area (km^2)

S = stream slope (m/km)

As this formula had been found to be the best available in the previous investigation, it was selected for testing for suitability for estimating rainfall duration in this project. As the formula was to be regarded as a means of estimating a characteristic response time of each catchment, it was only necessary for the use of the formula to lead to consistent values of runoff coefficient. Reproduction of measures of time of concentration, as used in the study of French et al. (1974), would not have been necessary.

However, the use of values of S_e rather than S_a for the slope term in equation (4.1) led to large values of rainfall duration. This resulted in high values of the derived runoff coefficients, with many values over one and large scatter of the values. The highest value was 3.0. It was considered that these high values would lead to resistance in acceptance of the method by designers. Also, the values of runoff coefficients derived by the approach outlined in Section 1.4.1 showed considerable inconsistencies and deviations from average trends. A sensitivity study was carried out on the effects of variations in duration on design rainfall intensities and derived runoff coefficients and the results are described in Appendix E. A 50% overestimate of duration, which was a common magnitude of this variation, typically produced a difference of about 25% in the average rainfall intensity and hence over 30% in the reciprocal function, the derived runoff coefficient.

In view of these unsatisfactory aspects of the Bransby Williams formula, an investigation was carried out to develop alternative formulae for rainfall duration.

4.4 DERIVATION OF FORMULAE FOR DESIGN DURATION OF RAINFALL

4.4.1 Measures of critical response time

Although the variable used to estimate the critical rainfall duration was considered to be a characteristic response time of the catchment and not necessarily the time of travel from the most remote location, the logical approach to derivation of a formula for estimating the critical rainfall duration is to use values of time of concentration estimated from observed floods. In the few cases where time of concentration has been estimated from observed flood data, some characteristic of the hydrograph has generally been used. The most common measure is the typical minimum time of rise of the flood hydrograph. This was used in the previous study of French et al. (1974), who describe and illustrate the variation of rise time with duration of rainfall excess and flood magnitude, and the selection of a typical minimum value for use as the time of concentration. A second variable that has been suggested as an estimate of time of concentration is the base length (usually denoted as C) of the time area diagram in the Clark (1945) model for synthesis of unit hydrographs. This is also equal to the time from the end of rainfall excess to the point of contraflexure on the falling limb of the flood hydrograph. Where the maximum ordinate of the time-area diagram is at or near its end, the minimum time of rise of the hydrograph (for a very short rainfall excess) is also equal to C. This is the case with the flood synthesis method of Cordery and Webb (1974).

The catchment lag, generally defined as the time between centres of mass of rainfall excess and the resulting hydrograph of direct storm runoff, has also been suggested as a measure of the critical duration of rainfall (e.g. Hoyt and Langbein 1955, Linsley et al. 1958, Bell and Omkar 1969). Although all three measures are highly correlated (Bell and Omkar 1969), lag is somewhat longer than the other two measures, which were adopted for this study.

Values of time of concentration derived from observed flood data were available for 96 small rural catchments. All of these values were effectively minimum times of rise of the flood hydrograph. Suitable values for 53 catchments were available from published studies, 23 being time of rise values from French et al. (1974) and 30 were Clark C values reported in ARR and by Baron et al. (1980). The remaining 43 values were minimum times of rise extracted in this study. Details of the 96 values are given in Appendix F.

4.4.2 Derivation of formulae

Relationships were derived between catchment response times and combinations of catchment characteristics. Details are described in Appendix F. As noted in Section 4.2, the catchment characteristics used were area A, length of main stream L, equal area slope S_e of the main stream, a catchment shape factor, a stream slope nonuniformity index defined by S_a/S_e, a vegetation cover roughness index, and median annual rainfall MAR. As several of these variables are correlated with one another as discussed in Section 3.7, it is not valid to consider each as an independent variable in regression studies. Various combinations of area, length and slope were used as single variables in the development of relationships.

The 96 catchments for which time of concentration data were available were divided into the three regions of Coastal, Tablelands and Snowy Mountains. Regressions of catchment response time on various combinations of catchment characteristics revealed that differences in relations between the regions were very small, and could be accounted for by the correlations between characteristics described in Section 3.7. Data for the three regions were then pooled and regression relationships were derived for the values from all 96 catchments. The three relationships adopted for further study were:-

$$t_c = 0.76\ A^{0.38} \qquad (r = 0.92) \qquad (4.2)$$

$$t_c = 1.69 \left(\frac{L}{\sqrt{S_e}}\right)^{0.50} \qquad (r = 0.92) \qquad (4.3)$$

$$t_c = 3.01 \left(\frac{L}{\sqrt{S_e}}\right)^{0.40} \qquad (4.4)$$

where t_c = time of concentration or characteristic response time (hours).

As noted in Appendix F, equation 4.4 is the relation for the Clark C obtained by Baron et al. (1980). The regression relation derived in this study was slightly different, but (4.4) was adopted here to prevent proliferation of formulae giving virtually the same results.

The length and slope values for use with these formulae must be the adjusted values to correspond with the standard map scale of 1:25 000, as described in Section 3.6

Only the simple measures A, L and S_e are used in the above equations which represent the three relationships using these measures with the highest correlation coefficients. The investigation described in Appendix F showed that the other catchment characteristics accounted for very little additional scatter, and would be difficult to evaluate in practice. The equations involving only A, L and S_e were thus adopted, as it was desired to use only simple and unambiguous relationships in the design procedure.

4.4.3 Similarity with relations of previous investigators

The above relations are very similar to those derived by several previous investigators. The form of equation (4.2) is the same as those for critical rainfall duration reported by Linsley et al. (1958), Hoyt and Langbein (1955), Bell and Omkar (1969) and Alexander (1972). The exponent 0.38 is identical with that given by Linsley et al. (1958) while exponents of 0.4, 0.33 and 0.4 were found by the other three investigators. The coefficient of 0.76 was found in this present study to be a constant virtually unaffected by catchment slope, median annual rainfall, catchment shape, slope nonuniformity or regional divisions. In comparison the coefficients derived by each of the above investigators excepting Alexander were considered to be functions of channel storage characteristics or vegetation cover. In general the times estimated from the formulae for critical lag would be expected to be slightly larger than those estimated using equation (4.2) for minimum time of hydrograph rise. Equation (4.3) is similar in form to the Ramser-Kirpich (Kirpich 1940) and Bruce and Clark (1966) formulae. The third relation (equation 4.4) is that obtained by Baron et al. (1980) for the base length C in the Clark model for synthesis of unit hydrographs and is very similar to the regression derived in this study and to the relation for C derived by Cordery and Webb (1974).

4.4.4 Use of standard area and one hour duration

An alternative approach to the formulae for critical duration of rainfall derived above was developed from a procedure suggested by French et al. (1971). In this, a standard catchment area is adopted, a value of 25 km^2 being used here. The observed flood flows are adjusted to values corresponding to this area by means of a power relation between discharge and area with an exponent of 0.7, as suggested by Alexander (1972). This relation is

$$Q \propto A^{0.7} \qquad (4.5)$$

The observed flows are thus multiplied by $(25/A^{0.7})$ to convert them to values corresponding to the standard area of 25 km^2. As this standard or reference area is now the same for all catchments, a single rainfall duration can be used in equation (1.2) to calculate the runoff coefficient. Equation (4.2) indicates that this should be 2.58 hours. However, a standard one hour duration was used to simplify the procedure. If the ratio of the one hour to 2.58 hour rainfalls was constant over New South Wales, this would simply scale down the coefficient. In fact, the ratio of the rainfalls does vary to some extent, but the variation is largely location dependent and would have little effect on the usefulness of the derived runoff coefficients. The critical rainfall duration for this procedure is thus always one hour.

4.5 APPROACH TO SELECTION OF DESIGN FORMULA

Five methods have been detailed above for estimation of the critical duration of design rainfall. The Bransby Williams formula is described in Section 4.3, three formulae listed as equations (4.2), (4.3) and (4.4) were derived in Section 4.4.2, and a standard one hour duration with adjusted runoff coefficients has been suggested in Section 4.4.4.

Selection of the formula to be adopted for the design method was based on two criteria. The first was the goodness of fit of the observed typical minimum times of hydrograph rise, where appropriate, as described in this chapter. The second and more important was the consistency of runoff coefficients derived using each method. Assessment of this second criterion, and selection of the design approach, are described in Chapter 7.

5. RAINFALL FREQUENCY ANALYSIS

5.1 SOURCE OF DATA

The most recent and readily available rainfall data source used in design flood estimation for small catchments is the 1977 'Australian Rainfall and Runoff' (ARR). As described previously the runoff coefficients derived in this project are ratios of peak rate of flood flow to average rainfall intensity. Rainfall data from ARR were used exclusively as the rainfall data source in this project as these data are used in practice in the estimation of design floods. The derivation and application of the derived runoff coefficients are then consistent, and use of the coefficients with the rainfall data in ARR should reproduce the flood flows of selected frequencies based on observed data from which the coefficients were derived.

5.2 SELECTION OF DATA FORM

Two data forms are available for use in ARR. One is based on pluviograph or recording raingauge data and the other primarily on daily rainfall data. To avoid inconsistency in application it was necessary to choose one of these forms.

5.2.1 Pluviograph data

Pluviographs or rainfall recorders give a continuous record of cumulative rainfall depth and as such record the temporal distribution as well as the amount of rainfall. The pluviograph data in ARR are presented as intensity - frequency - duration relationships and are derived from the original station records. Relationships are presented in ARR for 23 pluviograph stations in eastern New South Wales. Record lengths of these stations range from 15 to 60 years with a mean of around 25 years. The locations of the stations are presented on Figure 5.1.

From each station record the Bureau of Meterology has extracted intensity - frequency data for a range of durations, representing maximum average intensities of bursts of rain within storms. These relationships are in the form of sixth order polynomials of duration, with one set of seven coefficients for each return period and each station. They cover durations between 6 minutes and 72 hours, for return periods ranging from 1 to 100 years.

The pluviograph relation for a given station may be used for sites around the station which satisfy the following criteria listed in ARR:-

1. Elevations within 200m
2. Terrains within a radius of 5 km of each site are similar
3. Annual rainfalls differ by less than 100 mm in areas where annual rainfall is less than 1000 mm or by less than 10% in wetter areas
4. The aspects of the general area within 5 km of each site are similar This means that locations on both sides of minor watersheds may be considered to be similar, but locations on opposite sides of major divides may not.
5. Where the areas under consideration are within 50 km of the coast, the distances of each site from the coast should differ by not more than 20 km.
6. The two sites should not be more than 150 km apart.

The map in Figure 5.1 shows the areas for which the data from a pluviograph may be considered representative, and those areas which are excluded on the basis of the criteria listed above. In delineating these areas, the critera have been interpreted liberally, especially criterion 2. Despite this, it is evident that large areas of the region of interest are not covered by the pluviograph data.

5.2.2 Generalised data

The generalised rainfall intensity - frequency - duration data based mainly on daily rainfalls are presented in ARR in the form of rainfall maps (four for each State) showing the 12 and 72 hour duration rainfall intensities for return periods of 2 and 50 years.

5.2.3 Comparison of rainfall data forms

The two ARR rainfall data forms described above were compared with regard to their advantages and disadvantages for application in a design flood estimation procedure. These are listed below:-

Pluviograph data

(a) Advantages:

(i) easy to use in graphical form, and for this reason are in common use by local government and main roads authorities in several states. However, use in the sixth order polynomial form is often difficult.

(ii) better definition of short duration rainfall intensities.

(b) Disadvantages:

(i) do not cover all of the area of interest in eastern New South Wales (see Figure 5.1),

(ii) the procedures are based on only 23 pluviograph stations for eastern New South Wales. Most of these stations do not have long records and the relationships thus have only a small data base,

(iii) discontinuities may occur at the boundaries of zones represented by adjacent pluviographs.

Generalised data

(a) Advantages:

(i) cover the whole of eastern New South Wales,

(ii) the procedures were derived from a large data base of approximately 1000 stations with record lengths of up to 140 years,

(iii) more detailed variations of rainfall intensity with location.

(b) Disadvantages:

(i) rainfall intensities for durations less than 24 hours were derived from generalised relationships found from pluviograph data using 12 hour rainfall intensities as a base. That is, the procedures assume the same relationsips between intensity and duration to hold for daily read station data as were found to hold from pluviograph data,

(ii) for short duration rainfall intensities, the division of New South Wales into rainfall zones causes rainfall intensity discontinuities to exist at zone boundaries.

5.2.4 Selection of generalised data

The pluviograph data have been widely used in several States because they can be presented as a simple set of graphs, and are thus easy to use in practice. However, the generalised data from ARR can also be presented in a relatively simple form, and simplified procedures are detailed in Section 5.4 below. The disadvantage of discontinuities at zone boundaries in generalised data is matched by those at boundaries of zones represented by adjacent pluviographs. In both cases a linear interpolation scheme can be used, as described in Section 8.4 and in application of the design method in Chapter 10. Use of the generalised data form from ARR was adopted for use in this project and the resulting design method, based on the following considerations:-

(i) The generalised procedures can be used for the whole of eastern New South Wales whereas the pluviograph only cover about one half of this region,

(ii) the generalised procedures are based on a far larger data base in both space and time,

(iii) for short durations the generalised procedures gave intensities comparable with those from the pluviograph data at the pluviograph station sites.

Verification of this last point was obtained by comparing rainfall intensities calculated using both pluviograph based and generalised procedures at a number of pluviograph station sites. A comparison table of results is presented in Appendix G and shows generally good agreement between intensities calculated from each data source. Variations in region, return period and duration had negligible effect, in general, on the magnitude of differences of intensities. Similarly, no bias was evident in these differences. In a very few cases, intensity differences of up to 50% were observed. However, it is possible that sampling error alone or combined with local orographic or exposure effects could cause such differences.

As differences do exist between rainfall intensities calculated by each procedure and these differences are random, it is recommended that the generalised procedures only should be used and not intermixed with the other procedure when using the design method for flood estimation developed in this report.

5.3 DATA PROCESSING IN DERIVING DESIGN METHOD

The procedure used for deriving rainfall intensities from ARR for development of the design flood method is described below. This was simplified for use in applying the method, as described in Section 5.4. The duration range for which ARR procedures can be used to calculate rainfall intensity - frequency data at a site is 6 minutes to 72 hours. To calculate rainfall intensities of various frequencies covering this duration range it was necessary to extract five rainfall parameters from ARR as below (with

ARR references in brackets):-

AFACT (A in ARR)	-	Zone Factor dependent on location (Table 2.3 and Figure 2.17)
I(12,2)	-	the 12 hour duration, 2 year return period rainfall intensity in mm/h. (NSW - Figure 2.18)
I(12,50)	-	the 12 hour duration, 50 year return period rainfall intensity in mm/h. (NSW - Figure 2.19)
I(72,2)	-	the 72 hour duration, 2 year return period rainfall intensity in mm/h. (NSW - Figure 2.20)
I(72,50)	-	the 72 hour duration, 50 year return period rainfall intensity in mm/h. (NSW - Figure 2.21)

All of these parameters depend only on location, and the appropriate location used for each catchment was the approximate catchment centroid. However, this location was significantly different from the gauging station location for only the larger gauged catchments, as a result of the scale of the maps from which parameters were extracted.

Calculation of the design duration (t_c) was the second step using one of the appropriate catchment response time formulae described in Chapter 4. The general procedure was then to calculate $I(t_c,2)$ and $I(t_c,50)$ (the 2 and 50 year return period, t_c hour duration rainfall intensities), and use these to calculate intensities for other return periods of interest using Figure 2.46 in ARR. A different procedure to calculate $I(t_c,2)$ and $I(t_c,50)$ was used for each of the duration ranges 6 minutes to 60 minutes, 1 hour to 12 hours and 12 hours to 72 hours as shown below in Table 5.1

Table 5.1 Summary of Rainfall Intensity Calculations

Rainfall Intensity	Duration Range		
	6 - 60 minutes	1 - 12 hours	12 - 72 hours
$I(t_c, 2)$	$MF_2 \times I(12, 2)$	$MF_1 \times I(12, 2)$	$f^*[I(12, 2), I(72, 2)]$
$I(t_c, 50)$	$MF_2 \times I(12, 50)$	$MF_1 \times I(12, 50)$	$f^*[I(12,50), I(72,50)]$

$$\text{where } MF_1 = AFACT\left[\frac{1.78}{t_c + .576} - .143\right] + 1 \qquad \text{(equation 2.7, ARR)}$$

$$MF_2 = (AFACT + 1) \times \left[0.309 + \frac{49.586}{t_c \times 60 + 11.767}\right] \qquad \text{(equation 2.7 \& 2.8, ARR)}$$

f^* = function given in Figure 2.47 in ARR

Using Figure 2.46 of ARR, these intensities were then used to determine intensities for return periods between 2 and 100 years. To calculate the once in one year rainfall intensity, equation 2.6 of ARR was used:-

$$I(t, 1) = 0.885 \times I(t, 2) \Big/ \left[1 + 0.1734 \times \ell n \left(\frac{1.13 + I(t, 50)}{I(t, 2)} \right) \right] \quad (5.1)$$

Reduction of point estimates for application to catchments of specific size was not used. The differences for catchments up to 250 km^2 are small, and neglect of the reduction simplifies the procedure. The effects are incorporated in the derived runoff coefficients.

The whole procedure can be programmed for a TI 59 programmable calculator, as described in Appendix H. However, the CDC Cyber 72 computer of the University of New South Wales was used for this project as a result of the large amount of data and the different design duration formulae to be tested.

5.4 CALCULATION OF RAINFALL INTENSITIES FOR PRACTICAL DESIGN

The previous section describes programmed procedures that were used in the project in the development of the design method. However, for general use in practical design, a more direct and simple approach is required. This is described below, and is a re-presentation of the generalised procedure using daily rainfall data from ARR, which can be used instead of the simplified version given here. The procedure below covers the practical range of durations up to 12 hours and return periods between 2 and 100 years for locations in eastern New South Wales. Figures 5.2 to 5.6 reproduce the relevant design figures from ARR and present equations 2.7 and 2.8 of that publication in graphical form for ease of use. The steps in the use of Figures 5.2 to 5.6 for calculation of a design rainfall intensity are as follows:-

Step 1 - Obtain latitude and longitude of approximate catchment centroid (to the nearest minute)

Step 2 - Extract:- zone and zone factor (AFACT) from Figure 5.2;
- 12 hour, 2 year rainfall intensity I(12, 2) mm/h from Figure 5.3;
- 12 hour, 50 year rainfall intensity I(12, 50) mm/h from Figure 5.4

Step 3 - For the duration of interest (t) read off the appropriate multiplying factor (MF) from either Figure 5.5(a) or Figure 5.5(b). (Note - for durations greater than 12 hours ARR Figure 2.47 can be used).

Step 4 - Calculate the 2 and 50 year return period rainfall intensities for duration (t) hours:-

$I(t, 2) = MF \times I(12, 2)$

& $I(t,50) = MF \times I(12, 50)$.

Step 5 - Select an appropriate scale for the intensity axis on Figure 5.6 and plot I(t, 2) and I(t, 50). Draw a straight line through these points and read off intensities for the return periods of interest between 2 and 100 years.

As noted in Section 5.3 above, point rainfalls were not reduced to catchemnt average values in derivation of the design procedure. Consequently, point rainfalls only should be used in application of the method, and no reduction made for the size of the catchment.

Example of Use

Given location of interest:

latitude = 32° 36' (32.6°)
Longitude = 150° 24' (150.4°)

and selected duration t = 4 hours

then:-

Step 2 - Zone - B

Zone Factor = 3.78
I(12,2) = 5.0 mm/h
I(12,50) = 9.8 mm/h

Step 3 - Figure 5.5(b) is appropriate, and for Zone B, with t = 4 hours, MF = 1.95

Step 4 - I(t, 2) = 1.95 x 5.0
= 9.8 mm/h

and I(t, 50) = 1.95 x 9.8
= 19.1 mm/h

Step 5 - Plotting these two intensities on Figure 5.6 and drawing the connecting line allows the following rainfall intensities to be read off:-

Return Period (Years)	Rainfall Intensity (mm/h)
2	9.8
5	12.6
10	14.2
20	16.6
50	19.1
100	21.3

5.5 EXAMINATION OF ERRORS IN THE APPLICATION OF RAINFALL DATA

There are two major sources of error relating to rainfall data in the derivation of runoff coefficients from observed flood data, and in the use of the derived runoff coefficients in estimating design floods. The first concerns the basic assumptions of the Rational Method that the rainfall intensity is uniform in time and space during the duration of the design rainfall, and that this critical duration is equal to the time of concentration or characteristic response time of the catchment. Both of these assumptions are more likely to be fulfilled by small rather than large catchments. However, with the statistical interpretation of the Rational Method where the runoff coefficient represents the ratio of flood peaks to average rainfall intensities of the same return period, deviations from both of these assumptions may not be important. This would be the case if in deriving runoff coefficients, the observed periods of record from which the rainfall and flood frequency curves were derived contained the same types of deviations from the assumed conditions as were likely to occur in the design case. The average or expected (in the statistical sense) conditions are under consideration here. Under these circumstances, use of the design rainfall data with the derived runoff coefficients should reproduce the flood frequency curve from observed floods. While these deviations are thus of much less importance with the statistical interpretation of the Rational Method than when it is viewed as a deterministic model, errors are more likely as catchment size increases. A general upper limit of 250 km^2 was adopted in this project, largely to provide a satisfactory data base. Any limit is arbitrary, and it was not possible in the time available in this project to fully examine the effect of catchment size on the assumptions regarding rainfall in the Rational Method. Lower reliabilities were assigned to runoff coefficients derived for large catchments in the reliability index described in Section 2.2.3 and Appendix D.

The second major source of error is in the rainfall values extracted from ARR. Errors from this source can be divided into four types:-

(i) Intensities given in ARR are not representative of intensities in the general area. This could result from the rainfall at a station used in developing the ARR data not being typical of the surrounding region. However, this type of error would be relatively unlikely as a result of the large data base of rainfall stations used in the derivation of the generalised procedure.

(ii) Local features of the particular catchment cause its rainfall to be appreciably different to that over the general area. This type of error would be more likely to occur on small than on large catchments where spatial averaging would occur. Where local high or low rainfalls are likely, the procedure for regions of strong rainfall gradients in Section 2.7 of ARR could be used in estimating design rainfalls. However, these values were not used in the derivation of runoff coefficients in this project.

(iii) Errors in the derivation of intensities for durations less than 24 hours from daily data. These intensities are based on relationships derived from pluviograph data for each of the rainfall zones. The smaller the rainfall duration and thus the greater the difference from 24 hours, the larger would the relative error be likely to be. Larger errors are thus likely with smaller catchments.

(iv) User errors resulting from the difficulty of precisely locating a catchment on a map where rainfall gradients are high (for example, the north coast of New South Wales). In such areas a location error of 1.5 mm on a map can cause a 25% difference in rainfall intensity.

These causes of error and their effects were used as guides in a later study of runoff coefficient outliers (Appendix J).

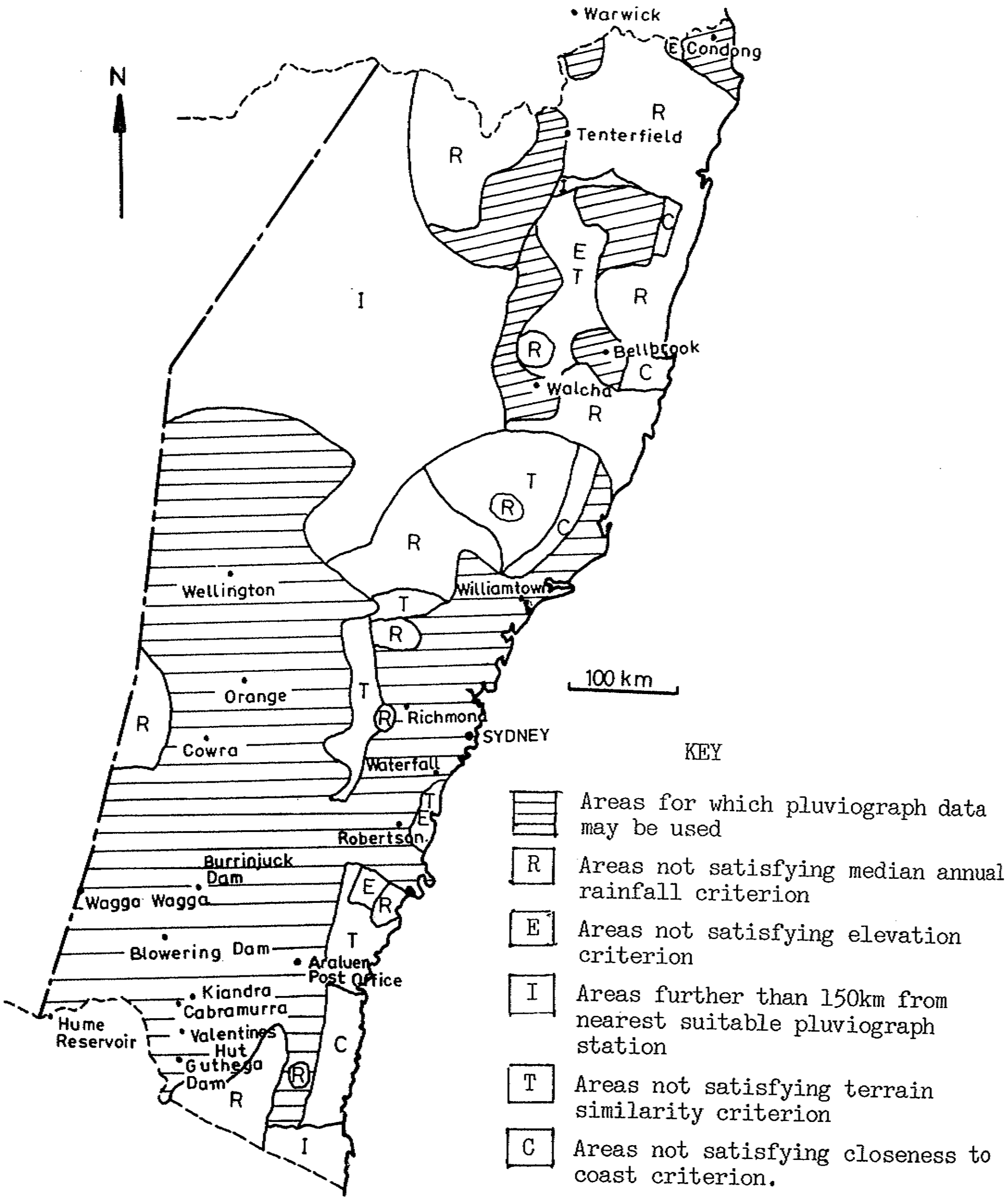


Figure 5.1 Locations of ARR pluviograph stations in eastern New South Wales, showing representative areas and areas that do not meet the criteria for transferability of data

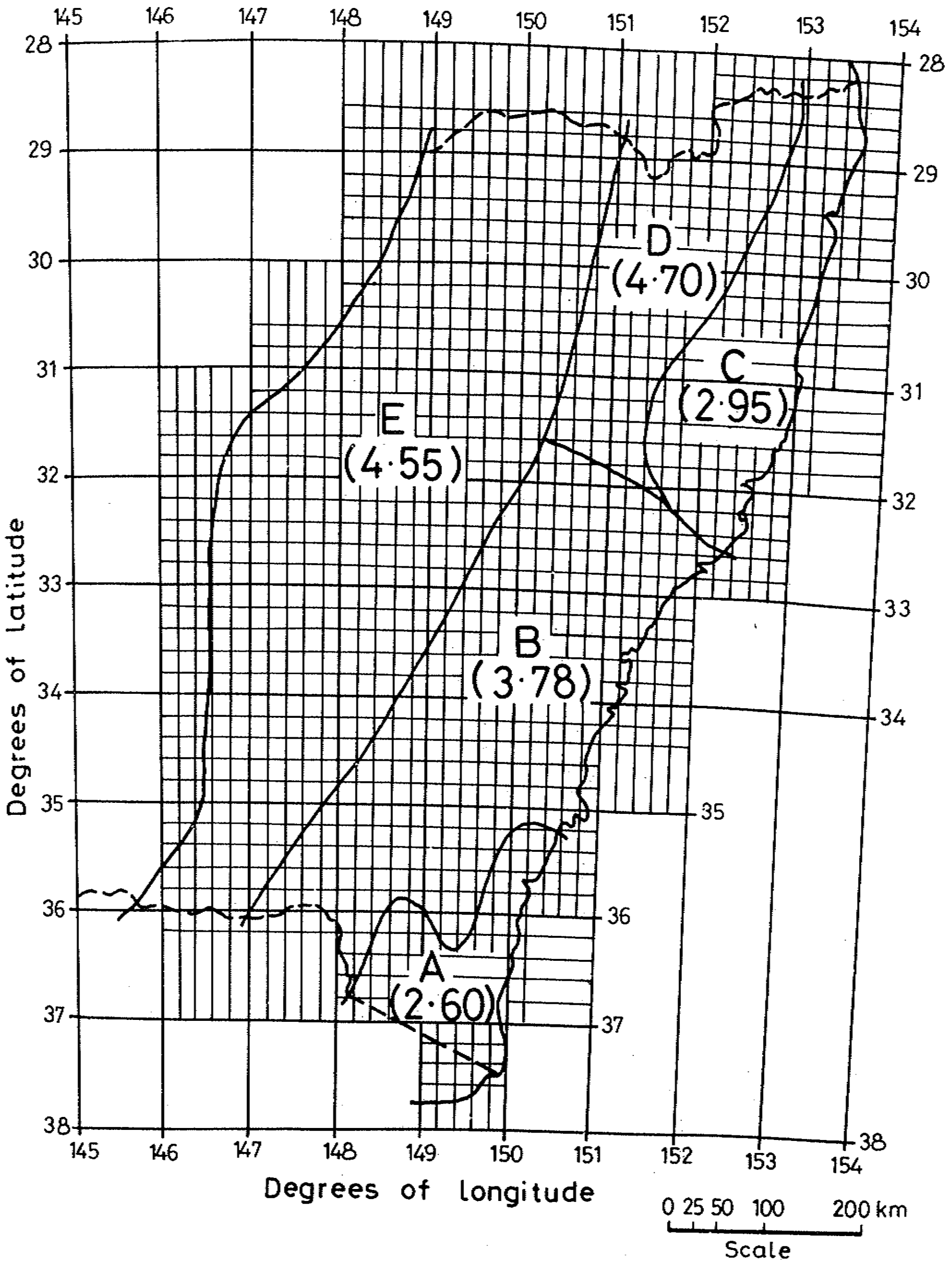


Figure 5.2 Rainfall zones from ARR, with factors AFACT shown in parentheses.

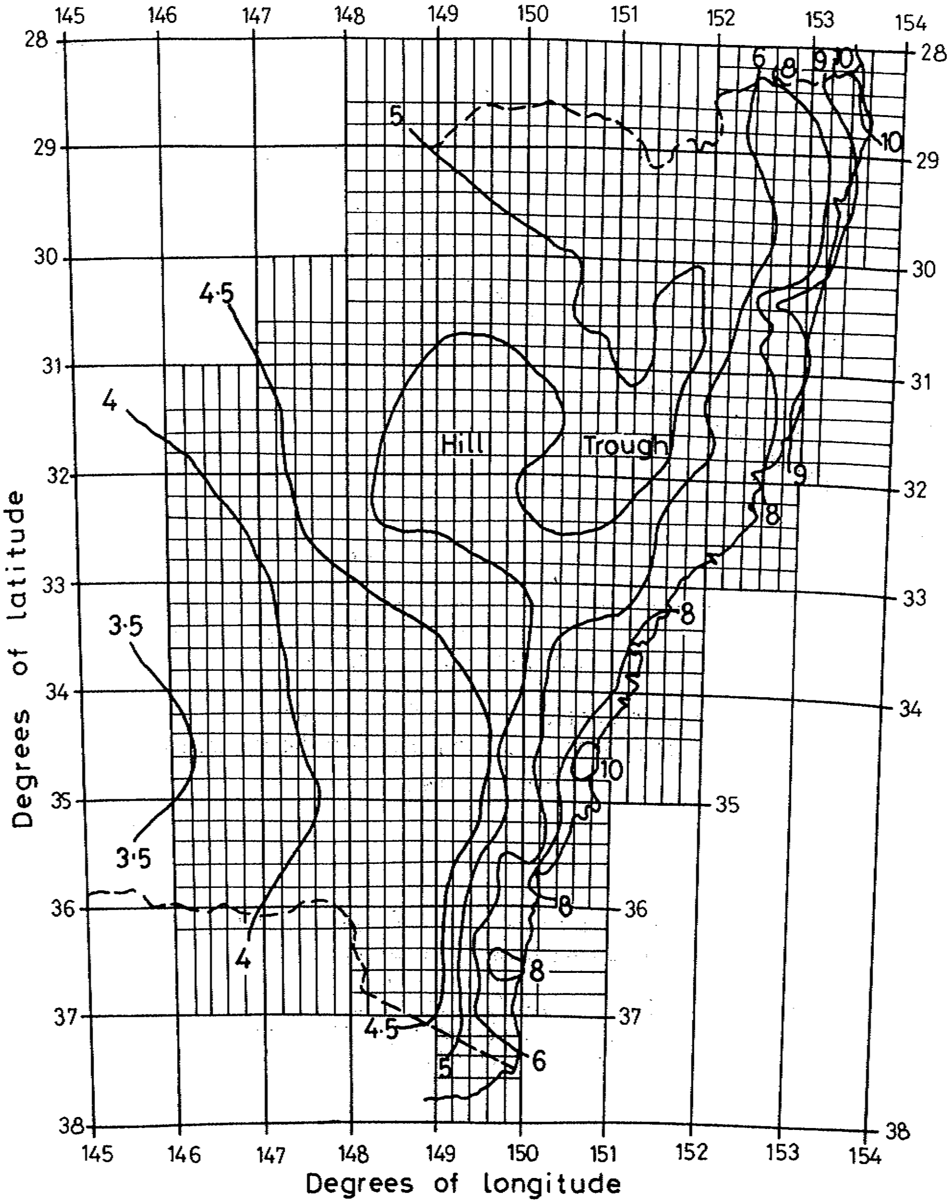


Figure 5.3 12 hour duration, 2 year return period rainfall intensities I(12,2) mm/h for eastern New South Wales from ARR.

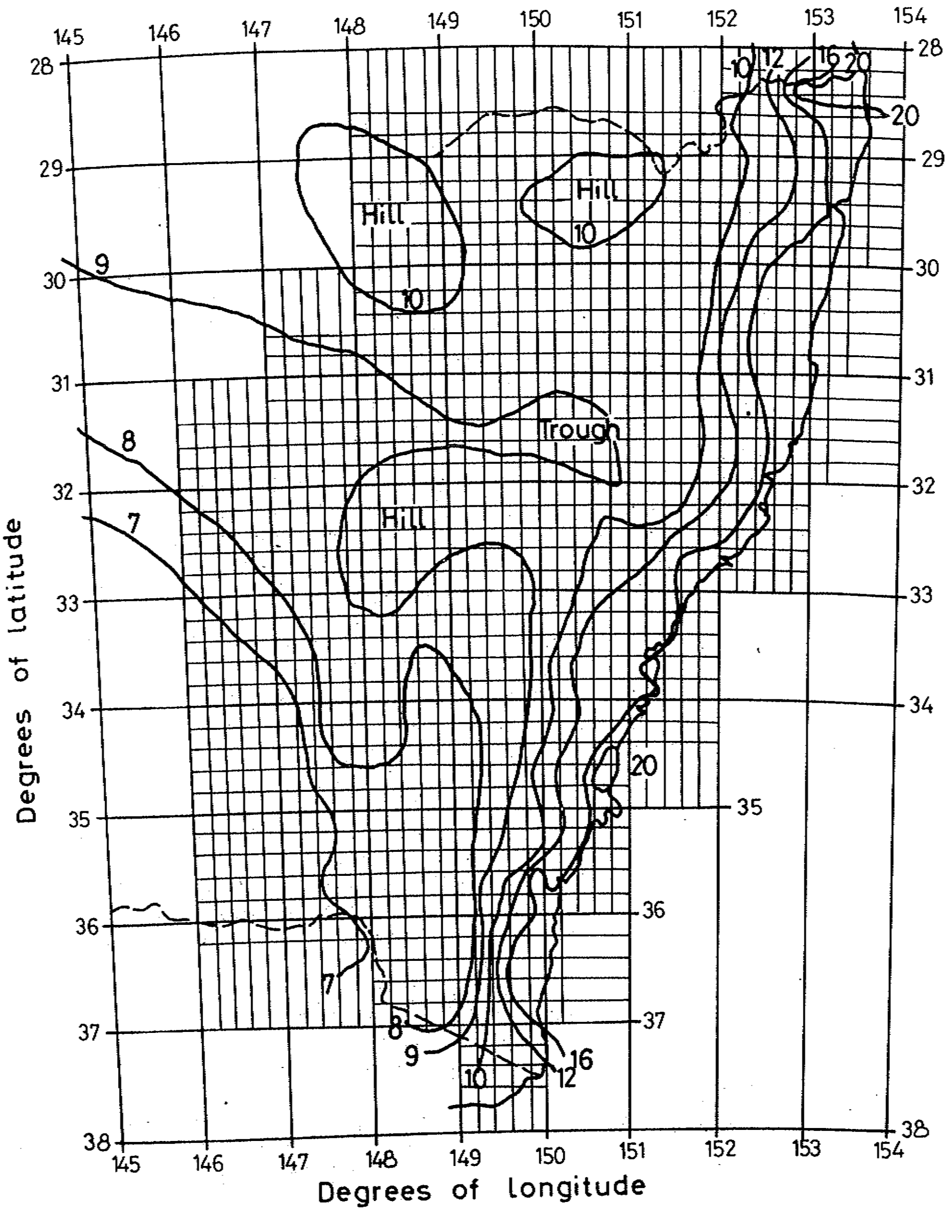


Figure 5.4 12 hour duration, 50 year return period rainfall intensities I(12,50) mm/h for eastern New South Wales from ARR.

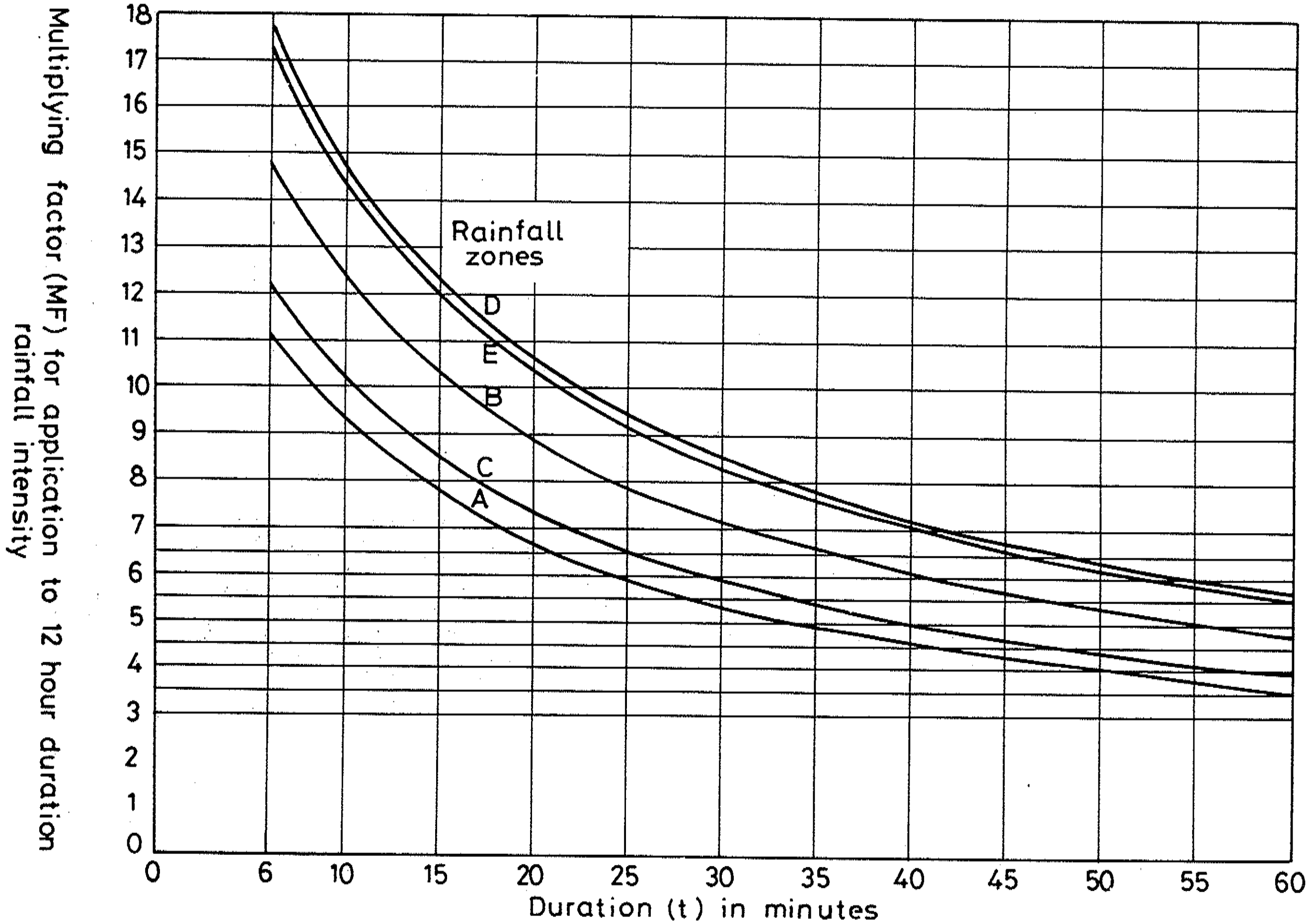


Figure 5.5.(a) Multiplying factors from ARR for application to 12 hour rainfall intensities to obtain intensities for durations of 6 - 60 minutes.

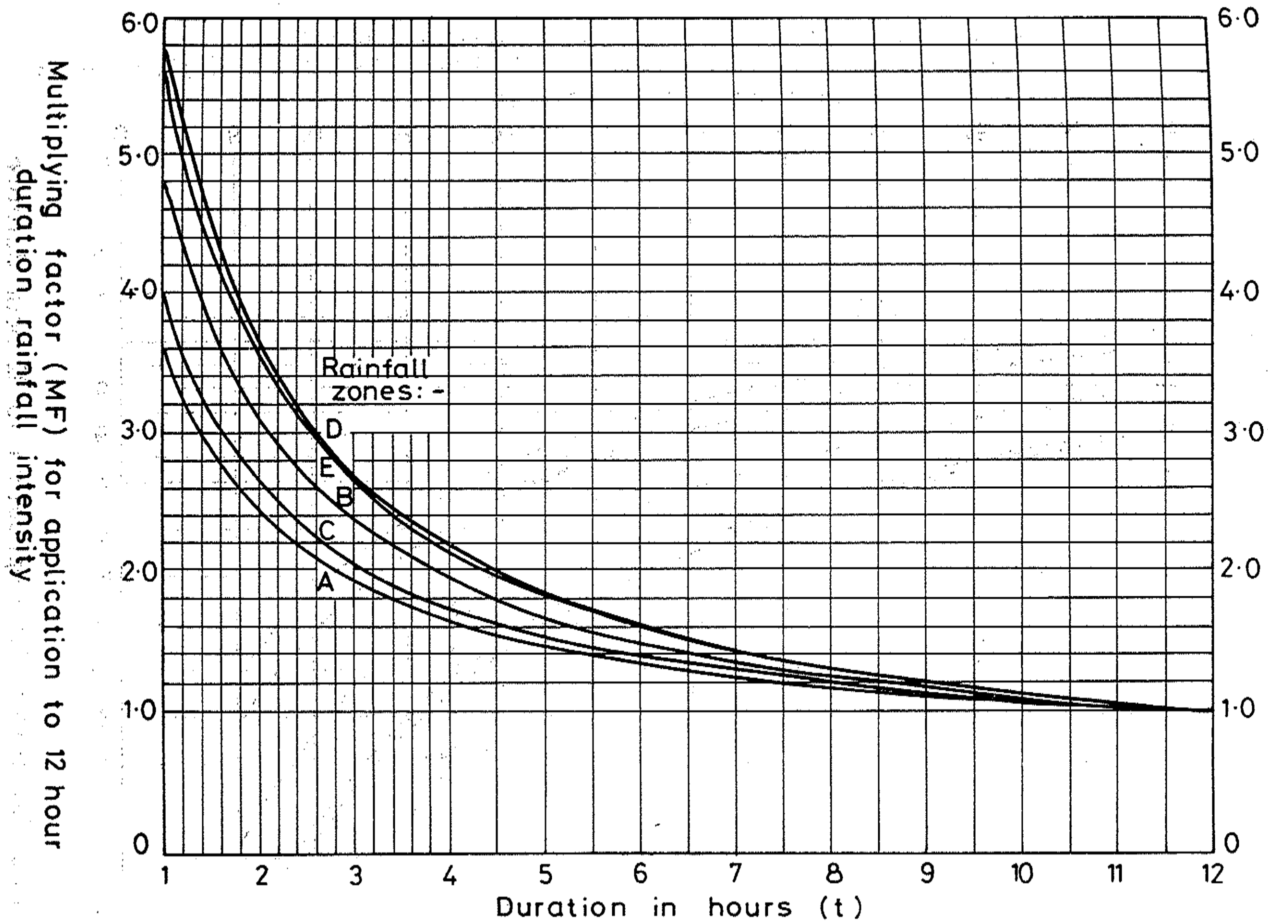


Figure 5.5(b) Multiplying factors from ARR for application to 12 hour rainfall intensities to obtain intensities for durations of 1 - 12 hours.

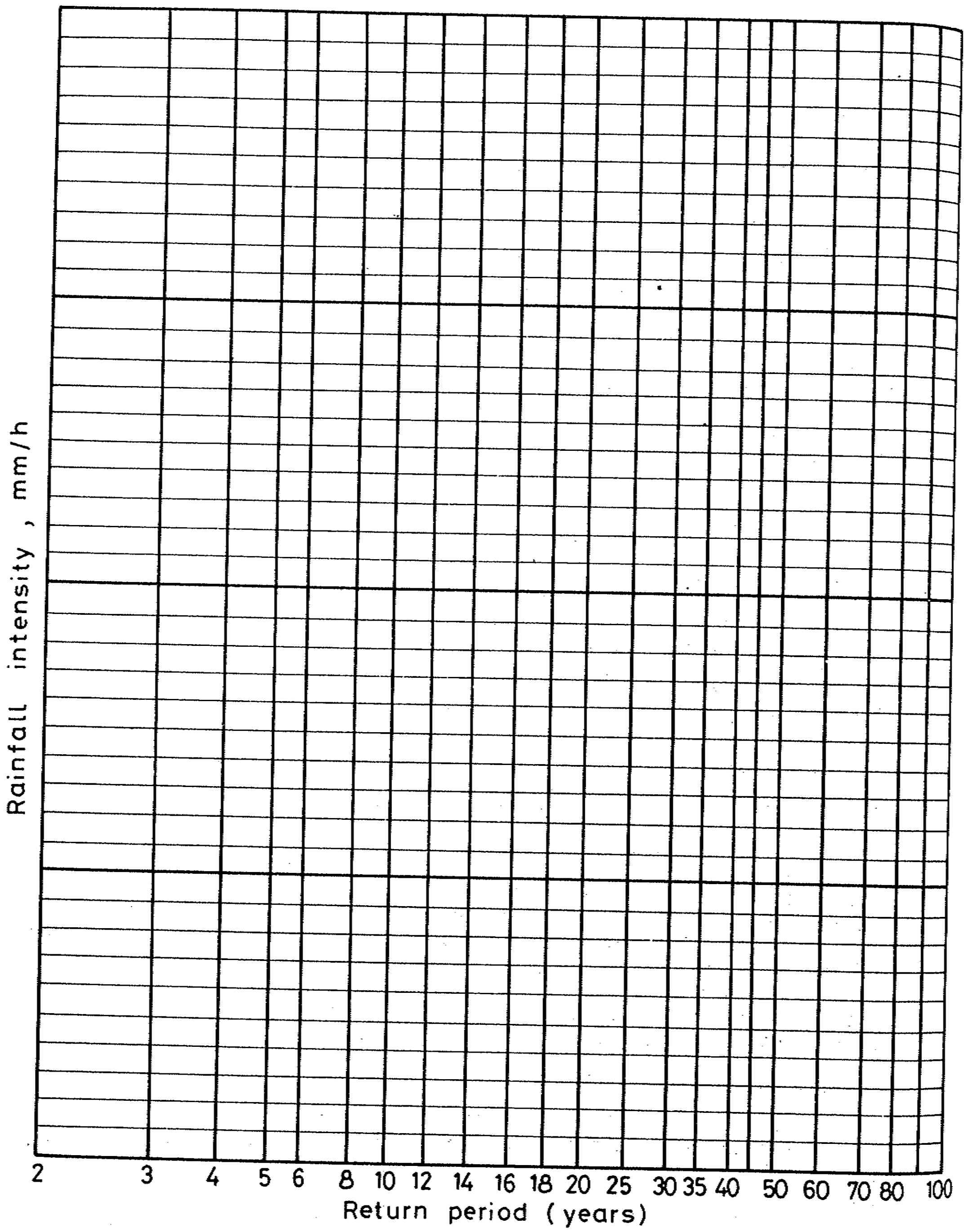


Figure 5.6 Interpolation diagram from ARR for determination of rainfall intensities for return periods of 2 - 100 years.

6. APPROACH USED FOR DERIVATION OF RUNOFF COEFFICIENTS

6.1 DESIGN METHOD ALTERNATIVES

In Chapter 4, five formulae for design rainfall duration were developed. Before selection of the final design procedure, runoff coefficients were derived for all of the catchments using each of the rainfall duration formulae, to provide a further basis for comparison. In addition to the five procedures using different formulae for estimating rainfall durations, two other methods using conceptually different approaches for determining design floods were also developed and tested.

In this chapter, the general approach used for derivation of the runoff coefficients for the first five methods and the corresponding parameters of the last two methods will be described. The comparison of the seven procedures on the basis of accuracy and consistency of the derived coefficients and also rainfall durations, and the selection of the adopted design procedure, are described in Chapter 7. More detailed development of the design runoff coefficients for the adopted procedure is described in Chapter 8.

The seven alternative design methods are summarised in Table 6.1. Methods 1 to 4 are direct applications of the statistical interpretation of the Rational Method, and runoff coefficients are calculated by equation (1.2) with the different formulae for design rainfall duration. A similar approach is used in method 5, but a standard rainfall duration of one hour is used and the flood flows estimated from the frequency analysis are transformed to values corresponding to a standard catchment area of 25 km^2 by multiplication by $(25/A)^{0.7}$, as discussed in Section 4.4.4. Method 6 is a more direct form of regional flood frequency procedure and uses a relative flow coefficient rather than a runoff coefficient. For a given location and return period in Method 7, the rainfall duration is determined as that value which would give a runoff coefficient of 0.8, and a flood conveyance factor is determined based on rearrangement of the Manning formula.

6.2 USE OF 10 YEAR RETURN PERIOD FOR BASIC DESIGN DATA

The flood frequency curve for each station was extended to a return period only a little longer than the period of record. Estimated floods for return periods above 10 years were thus not available for many of the catchments. That is, the highest return period for which the whole of the data base could be used was 10 years. Return periods less than 10 years are below those most commonly used in design. Accordingly, runoff coefficients and other parameters derived for a return period of 10 years were adopted as reference values for use in both development of the design procedures and for comparison of the seven alternative procedures.

6.3 MAPPING OF 10-YEAR COEFFICIENTS

Inspection of the 10-year coefficients derived for each of the seven procedures showed that the values were strongly related to location, and that mapping would account for much of the variation in the coefficients. This results from the fact that the effects of several factors influencing flood flows are reflected in location, such as soil and rock types, topography and relief, rainfall intensity, and possibly the most important, the average antecedent wetness of the catchment. The last factor depends on average annual rainfall, temporal distribution of this rainfall, potential evapotranspiration and drainage characteristics of the catchment. Mapping of the 10 year coefficients was therefore adopted as the means of presenting the basic design data, as used by French et al. (1974). Although this accounts for the overall or gross variation in the coefficients derived for the catchments, it was hoped that deviations of individual values from generalised contours or isopleths would be able to be related to characteristics such as soil type, slope and vegetation. This is discussed further in Chapter 8.

For each method, 10-year coefficients for the 284 catchments were plotted on base maps of eastern New South Wales and adjoining regions. Contours or isopleths of best fit were then drawn through these values. Reliability indices as described in Section 2.2.3 and Appendix D were noted against each catchment. These are shown on Map 1 at the back of this report. In drawing the average isopleths, greater weight was given to the stations with higher reliabilities. Patterns of elevation and relief, average annual rainfall and 12 hour rainfall intensities were also used in providing guidance in drawing the isopleths.

6.4 FREQUENCY FACTORS FOR OTHER RETURN PERIODS

For return periods other than 10 years, it would be possible to produce similar isopleth maps of coefficient values. However, the relative scarcity of stations for which high return period floods could be estimated with any certainty and the lack of areal coverage would make the drawing of maps for these high return periods difficult. In addition, inconsistencies between maps would be likely as a result of sampling errors of values at individual stations. In view of these difficulties, the frequency factor approach used by French et al. (1974) was also adopted here. For this, the ratio C(Y)/C(10) of the coefficient for each return period to the 10-year value is calculated for all of the catchments. For each return period, average values of the ratio are then determined for all catchments within regions or over the whole area of interest. Alternatively, the ratios could be related to catchment characteristics if significant relationships can be shown to exist. The results derived for the adopted design method are described in Section 8.3

Table 6.1 Summary of Seven Design Procedures and Derived Parameters

METHOD NUMBER	DERIVED PARAMETER	TIME PARAMETER (h) USED IN DERIVATION	SOURCE AND REMARKS
1	$C(Y) = \dfrac{Q(Y)}{0.278 \times A \times I(t,Y)}$	$t = 0.975 \dfrac{L}{A^{0.1} S_a^{0.2}}$	Bransby Williams (ARR) formula.
2	As above	$t = 0.76\ A^{0.38}$	From correlation studies (Appendix G)
3	As above	$t = 1.69 \left(\dfrac{L}{\sqrt{S_e}}\right)^{0.50}$	From correlation studies (Appendix G)
4	As above	$t = 3.01 \left(\dfrac{L}{S_e}\right)^{0.40}$	Both from correlation studies and Baron et al. (1980)
5	$C'(Y) = \dfrac{Q(Y) \times \left(\frac{25}{A}\right)^{0.7}}{0.278 \times 25 \times I(1,Y)}$	$t = 1$	From French et al. (1971) using Alexander's (1972) relation $Q \propto A^{0.7}$
6	$K(Y) = \dfrac{Q(Y)}{A^{0.7}}$	NA	Using Alexander's (1972) relation $Q = KA^{0.7}$
7	$CF(Y) = \dfrac{L}{D_y \times \sqrt{S_e}}$	D from solution of $I(D,Y) = \dfrac{Q(Y)}{0.278 \times 0.8 \times A}$ (ie. $C(Y) = 0.8$)	From rearrangement of Mannings Eqn: $CF = \dfrac{R^{2/3}}{n} = \dfrac{V}{\sqrt{S_e}} = \dfrac{L}{D\sqrt{S_e}}$; where D was calculated assuming a constant runoff coefficient of 0.80.

7. SELECTION OF DESIGN METHOD

7.1 BASIS FOR SELECTION

As noted in Sections 4.5 and 6.1, two criteria were adopted as the basis for selecting the design method from the seven procedures developed in the study. The more important of the two criteria was the accuracy and consistency of the runoff coefficients or other parameters derived using each procedure. The second criterion was the accuracy with which the rainfall duration formulae fitted the observed minimum times of hydrograph rise. This second criterion was only appropriate to the first four procedures incorporating a formula for time of concentration or characteristic response time.

7.2 INITIAL SELECTION

An initial examination of the seven procedures was carried out to select two methods for further detailed study. Ten year runoff coefficients or parameters were derived for each of the catchments for each of the procedures. These were then mapped and isopleths or contours drawn as described in Section 6.3 The accuracy and consistency of these contour maps were then assessed visually on the basis of:-

(a) goodness of fit of contour lines to individual derived values of runoff coefficients or other parameters;

(b) general smoothness of contour patterns.

For the second criterion, the goodness of fit to the observed minimum times of hydrograph rise was judged by the correlation coefficients of the derived formulae for critical duration of rainfall where they were available. No value was available for the Bransby Williams formula (method 1), but the plotted data in French et al. (1974) show that the correlation is poor. For method 4 where the relation of Baron et al. (1980) was adopted rather than that derived in this study, the correlation coefficient for the latter rather than that of Baron et al. (1980) was used for consistency.

The results of the comparison of the seven procedures are summarised in Table 7.1

Table 7.1 Comparison of Performance of Seven Design Procedures

Method Number (see Table 6.1)	Satisfaction of Criterion: Runoff coefficient or parameter (a)	Satisfaction of Criterion: Runoff coefficient or parameter (b)	Minimum time of hyd. rise - correlation coefficient
1	Poor	Fair	Low
2	Good	Good	0.92
3	Fair	Fair	0.92
4	Fair	Fair	0.81
5	Good	Good	-
6	Good	Fair	-
7	Poor	Poor	-

Although the assessment of the accuracy and consistency of the runoff coefficients and other parameters was subjective and based only on visual examination, the differences in the comparison were sufficiently obvious to eliminate the four methods 1, 3, 4 and 7 from further consideration. Of the three remaining procedures, method 6 was then excluded as the contours of the plotted 10-year parameter showed considerably more irregularity and direction changes than those of methods 2 and 5. This almost certainly results from the fact that variations of rainfall intensity with location are explicitly accounted for in methods 2 and 5, and their runoff coefficients do not incorporate these variations. However, the parameter values of method 6 have to incorporate the effects of variations of rainfall intensity as well as those of the other factors that affect flood magnitude.

As there was no means of subjectively choosing between the remaining methods 2 and 5, a more detailed and quantitative study was made of these two methods.

7.3 FINAL SELECTION OF METHOD 2

Before the detailed comparison of methods 2 and 5, the contour plots of the 10-year coefficients C(10) for both methods were refined. From the results of regression and correlation studies on both runoff coefficients (Tables I2 to I5 , Appendix I) and flood magnitudes (Table I6, Appendix I), these variables were found to be significantly related to rainfall intensity and mean annual rainfall, and to a lesser degree to catchment elevation and catchment relief. Underlay maps of each of these characteristics were then prepared and used as guides in refining the contour maps of runoff coefficients for methods 2 and 5. The resulting maps are shown as Maps 2 and 3 at the back of this report.

Values of C(10) were then read from the contour maps at each location for which a value had been derived. Deviation ratios for each site were then calculated as

Underestimates: $MF = C_D/C_P$ (7.1)

Overestimates : $DF = C_P/C_D$ (7.2)

where C_D = derived 10-year runoff coefficient for the location

C_P = value of 10-year runoff coefficient predicted from contour map

MF = ratio by which to multiply C_P to obtain C_D

DF = ratio by which to divide C_P to obtain C_D

The reciprocal ratios were used to keep all values greater than one.

Frequency distributions of the deviation ratios in different ranges calculated for 271 of the catchments are listed in Table 7.2. The catchments used comprise all of those in eastern New South Wales, those in Queensland (all of which were close to the border), and those in Victoria close to the border. The deviation ratios shown would also apply to observed and estimated 10-year peak discharges at the sites. For method 2, the contours shown on Map 2 at the back of the report have been amended in some areas from the contours used in the derivation of the deviation ratios listed in Table 7.2. However, these amendments (carried out after later detailed study) would make little difference to the distribution of the deviations shown in Table 7.2

Table 7.2 Distribution of C(10) Deviations - Methods 2 and 5

Direction of Deviation of C_P from C_D	Type of deviation ratio	Range of deviation ratio	Numbers of catchment values in various ranges	
			Method 2	Method 5
Under-estimate	MF $=C_D/C_P$	3-5	-	-
		2-3	3	3
		1.5-2	6	5
		1.3-1.5	14	12
		1.15-1.3	21	26
		1-1.15	88	82
Over-estimate	DF $=C_P/C_D$	1-1.15	88	84
		1.15-1.3	26	32
		1.3-1.5	17	12
		1.5-2	5	9
		2-3	3	5
		3-5	-	1
		Total	271	271

The tabulated deviations show that there is little difference between the performances of the two methods, although method 2 is slightly better. However, method 2 is easier to use than method 5 and more closely conforms with the traditional Rational Method. It thus has an advantage in terms of the aims and constraints of the project outlined in Section 1.3, that the method developed should be simple to apply, suited to the needs of the ordinary designer, and preferably be similar in form to the familiar Rational Method to encourage its adoption in practice. In view of these advantages, method 2 was adopted as the design procedure to be developed and presented as the final result of this study.

The runoff coefficients of various return periods derived for each of the 284 catchments by method 2 are listed in Appendix J.

8. DESIGN RUNOFF COEFFICIENTS

8.1 TEN-YEAR RUNOFF COEFFICIENTS

8.1.1 Mapped values

As discussed in Section 7.3, values of C(10) were mapped for eastern New South Wales on Map 2 at the back of this report for the adopted design procedure with rainfall durations calculated by equation (4.2). The derived C(10) values for all of the catchments are listed in Appendix J. As with the earlier values of French et al. (1974) derived from a much smaller data base, the values of the coefficients in Map 2 show a wide range from one or greater near the coast, grading rapidly to low values on the western slopes at the western limit of the region to which the method applies. This rapid variation in runoff coefficients contrasts with the much more uniform values in current design methods based on judgment and experience. The consequent error in these methods and their inability to reproduce variations in recorded flood data over eastern New South Wales have been discussed by Pilgrim (1978) and Cordery and Pilgrim (1980).

The reasons for the rapid decrease of runoff coefficient values away from the coast is not entirely clear, but the major cause is probably variation in the average antecedent wetness of the different regions and its effect on initial losses and hence runoff volumes. The contours on Map 2 are of similar configuration to the isopleths of average annual rainfall, and the higher rainfalls near the coast would lead to wetter average conditions and lower initial losses. Intense rainfalls of short duration tend to occur in long duration general storms near the coast of New South Wales, while they occur much more frequently in isolated thunderstorms in inland regions (Pilgrim 1966). Initial losses are likely to be satisfied in the former case and to result in much greater runoff. Higher potential evapotranspiration in the less humid inland regions would also be likely to result in lower average antecedent wetness and higher initial losses.

As noted in Section 7.3, the configuration of the isopleths of runoff coefficient values in Map 2 is also related to those of the rainfall intensity maps in ARR, and to a lesser extent to patterns of topography and relief.

For catchments smaller than 100 km^2 in the western part of eastern New South Wales, recommended design values of C(10) vary to some extent from those shown on Map 2. This is discussed in Section 8.2.

While the isopleths on Map 2 are based on 284 runoff coefficient values and have been drawn to take account of variations in average annual rainfall and runoff, rainfall intensities, topography and relief as discussed above, they should still be regarded as generalised values. It was not possible in this project to take account of variations in the above factors and in catchment characteristics at the local scale. It would therefore seem reasonable in a local area to adopt a more detailed variation of runoff coefficients based on small-scale variations of characteristics. However, the general trends of coefficients should follow those on Map 2 based on all available recorded data. It should also be noted that as discussed in subsequent sections of this Chapter, it was not possible to identify the effects of variations in catchment characteristics on the coefficient values derived from the recorded data. Despite this, local features such as rock outcrops, flat areas of alluvium, and sand dunes, might warrant modification of the mapped isopleths. The only disadvantage of such modifications is that they would introduce subjectivity into the design procedure whereas the use of the isopleths is objective.

Information on historical floods might also be used to modify locally the coefficient values from Map 2. However, this information should be used with great care, and generally little is known of the probabilities of the floods. Once again, the general trends of coefficients should follow the isopleths on Map 2, as these are based on a large amount of recorded data.

Map 2 indicates that for much of the south coast of New South Wales and for small regions on the north coast, the 10-year runoff coefficients are greater than one. The frequency factors derived below in Section 8.2 indicate that for a return period of 50 years, this would apply to most of the south coast and nearly half of the north coast. While runoff coefficients of greater than unity are not possible with the deterministic interpretation of the Rational Method, they are with the statistical interpretation. Several factors could account for values greater than one in regions with high rainfall and runoff:-

(i) The rainfall intensities in the intensity-frequency-duration data used may be too low. In this regard, it should be noted that the generalised data used were derived basically from daily data.

(ii) A further reason for low rainfalls may be that in areas of rugged topography, especially in forested regions, rain gauges are often located at low elevations in cleared valleys. Data from these gauges may appreciably underestimate average rainfall over the entire area.

(iii) The rainfall and flood data used for deriving the respective frequency curves were in general not observed over identical periods. There may thus be some consistent bias resulting from the sampling periods. The period since 1950 when most runoff data were observed has contained some very wet years.

(iv) The design rainfall durations used in determining the design intensities may have been longer than the durations of intense rainfall that caused the highest floods.

(v) Related to (iv), the design approach only utilises the mean rainfall intensity over the design duration, and ignores variations of intensity that would increase the flood peak.

(vi) Where runoff coefficients are close to unity, random errors in both the observed rainfall and runoff would be expected to cause up to half the derived coefficients to be greater than one.

It is not possible to give categorical reasons for the runoff coefficient values greater than unity. What is certain is that with the design procedures adopted in this report, coefficient values greater than one must be used on parts of the coastal region to reproduce the observed flood frequency data. This also highlights the fact that to use the method validly and to obtain the best possible reproduction of the observed floods, exactly the same procedures and data must be used in applying the design method as were used in deriving the runoff coefficients. For example, if improved rainfall intensity data were published for a given region, use of these would lead to less accurate flood estimates. Valid use of the new rainfall data would require rederivation of the runoff coefficients using these rainfalls.

The coefficient values greater than one also illustrate the fact that in the use of the Rational Method in design practice, the concept of the runoff coefficient as being the fraction of some actual rainfall that runs off is fallacious. In all design applications, it is actually the ratio of peak discharge to average rainfall intensity of the same return period.

8.1.2 Deviations from mapped values

Although location accounts for much of the variation in the C(10) values, the mapped contours still deviate appreciably from the derived values as discussed in Section 7.3. It would be expected that catchment characteristics such as area, slope and soil type would affect the C(10) values. Characteristics that are not location dependent should be related to the deviations from the contours, which represent trend or average values with location. If relationships could be developed between the C(10) deviations and catchment characteristics, their use would improve the accuracy of the design procedure. Study of the deviations is described in the following sections.

8.1.3 Outlier values

The extreme outlier values of C(10) deviations for all of the methods tested were always from the same catchments. A careful examination of all aspects of these catchments and the calculation of their runoff characteristics was carried out, and is described in Appendix K. The only general conclusion was that most of the catchments had rating curve extension problems. However, there was no apparent explanation for the large deviations for some catchments.

8.1.4 Expected random scatter of C(10) values

Before correlation studies were attempted, the random error in the C(10) values that would be expected to result from the scatter of the original flood data was investigated. The flood frequency studies were carried out on a mixed population of record lengths and periods and for a wide range of catchment sizes. The expected deviations resulting from sampling error would therefore be large. The 271 catchments used in the comparison to select the design method in Section 7.3 were also used for the investigation of random errors. These were all of the catchments in eastern New South Wales, those in Queensland (all of which were close to the border), and those in Victoria close to the border.

For the Log-Pearson Type III distribution that was used in fitting the flood frequency curves ARR suggests appropriate confidence intervals. These were derived from the non-central t distribution (e.g. Resnikoff 1957). Use was made of this distribution to enlarge the table given in ARR for other confidence intervals in addition to those given for 5% and 95% probabilities. Using these data, the number of stations within various ranges of record length, and the overall average value of 0.230 of the standard deviations of the logarithms of the flood events for the gauging stations, it was possible to determine the expected frequency distribution of the deviations of the estimated 10-year flood magnitudes from the true values for the 271 stations. This distribution was calculated in the form of the expected numbers of the 271 catchments for which the deviations fall into various class intervals. As errors in the flood frequency estimates are transferred directly into the derived runoff coefficients, these expected numbers of deviations also apply to the C(10) values. They are listed in Table 8.1 in terms of the deviation ratios defined by equations (7.1) and (7.2) in Section 7.3, where the multiplying factor MF applies to underestimates of the predicted values C_P from the contours of coefficients on Map 2 at the back of this report compared with the derived values C_D, and the dividing factor DF applies to overestimates of the predicted C_P relative to the derived C_D. The expected numbers of deviations in various ranges of deviation ratios are compared in Table 8.1 with the actual numbers of catchments in each deviation range reproduced from Table 7.2. Distributions of the actual and expected numbers of deviations are also shown in Figure 8.1(a) and (b). Part (c) of the figure is discussed in Section 9.1. As noted for Table 7.2 in Section 7.3, the actual numbers of catchments in the various deviation ratings were based on a preliminary map of the contours of C(10) values, which was later amended in some areas to the configuration on Map 2 at the back of this report. However, these amendments would make little difference to the distribution of the actual deviations.

In investigating whether the deviations of the derived from the mapped values of C(10) could result from sampling errors in the observed flood series, the hypothesis is that the contours of C(10) values on Map 2 give the true values. Thus if the observed flood sample at a station gave a value of Q(10) and hence C(10) that was greater than the true population value (assumed by the hypothesis in this comparison to be the predicted value C_P), then the derived C_D would be too high, and the ratio C_D/C_P would be greater than one. This would correspond to the underestimate case in Section 7.3 where C_P is less than C_D and the deviation ratio is a multiplying factor MF. Thus a sampling overestimate corresponds with an underestimate deviation ratio and is listed in this way in Table 8.1. Similarly a sample of observed floods lower than the true population value corresponds with the overestimate case and dividing factor DF of Section 7.3.

Table 8.1 Expected C(10) Deviations Resulting from Sampling Error, and Actual Deviations

Direction of deviation of C_P from C_D	Type of deviation ratio	Range of deviation ratio	Numbers of values in various ranges: Expected from sampling theory	Actual from derived values
Under-estimate	MF $=C_D/C_P$	3-5	1	88
		2-3	3	3
		1.5-2	11	6
		1.3-1.5	19	14
		1.15-1.3	37	21
		1-1.15	72	88
Over-estimate	DF $=C_P/C_D$	1-1.15	83	
		1.15-1.3	34	26
		1.3-1.5	10	17
		1.5-2	1	5
		2-3		3
		Total	271	271

Comparison of the C(10) deviations expected from sampling theory with the actual deviations as presented in Table 8.1 and Figure 8.1 indicates that the actual deviations should in most part be expected from consideration of sampling errors in the basic flood data. Overestimates of the deviation ratios however are not entirely accounted for by the underestimate sampling errors. This trend probably results more from the assymetric shape of the assumed confidence limit distribution used than from physical factors.

The distribution of expected values in Table 8.1 and Figure 8.1 reflects the shape of the non-central t distribution in that there is a higher expectation of overestimates of flood samples and hence underestimates of deviation ratio values. The origin and justification for the use of this distribution to represent confidence limits seems fairly obscure (Beard 1962). The assumption made to obtain the confidence limits was that the population of logarithms of flood flows was normal or near normal (i.e., skew $\approx$ zero). Later work in this field however, has shown that for non-zero coefficients (between $\pm$ 0.5), confidence limits can be expected to be wider (U.S. Water Resources Council 1977, pp. 9.1, 9.8). The average skew coefficient for all stations used in this present report was 0.72. Thus the values shown in Table 8.1 are conservative estimates of expected deviations.

In view of the above considerations and comparisons, there is no statistical reason for rejecting the hypothesis that the contours of C(10) values from the mapped values could be entirely accounted for by the scatter inherent in the observed flood data and the frequency curves derived from them. Reduction of the random variations could only be achieved by the collection of longer and more accurate streamflow records. This also means that it is really invalid statistically to attempt to extract more information from the derived C(10) values than the mapped contours, such as to attempt to relate the deviations to catchment characteristics or other physical variables. Despite this, an investigation was carried out to determine whether any relations between C(10) deviations and other variables could be found, and this is described below.

8.1.5 Correlations of C(10) deviations

Simple regressions were determined for C(10) deviations on the logarithms of each of catchment area A, slope S_e, and slope non-uniformity index S_a/S_e. A multiple regression on the logarithms of A and S_a/S_e was also determined. Details of the regression and correlation coefficients are given in Table I7, Appendix I. All of the regression coefficients were very small, and the correlations were not significant. Other relationships were also examined with no better results. It therefore seems that the C(10) deviations cannot be related to any physical variables with the data currently available. This confirms the finding in Section 8.1.4 above that the C(10) deviations can be accounted for by the scatter in the basic flood data. The mapped contours thus provide the best design information on 10-year runoff coefficients.

8.2 DESIGN VALUES OF C(10) IN WESTERN PORTION OF EASTERN NEW SOUTH WALES

8.2.1. Lack of data for small catchments

On map 2 at the back of this report, a line has been drawn joining, from north to south, the towns of Ashford, Tamworth, Bathurst, Yass, Tumut and Jingellic. This line is approximatley along the upper Western Slopes. Runoff coefficients have been derived for thirty eight gauging stations on or west of this line, providing a rather sparse basis for the adoption of design data. However, a more serious deficiency is that only three of the catchments are smaller than 100 km^2. One of these is fairly large (67 km^2, station 410070) and is situated virtually on the line near Tumut. The second (31 km^2, station

421051) is also virtually on the line near Bathurst and partly urbanised. The other is very small (0.07 km^2, station 410351) in the same region as the first at the Wagga Wagga Soil Conservation Service Research Station. There is thus really no data for catchments smaller than 100 km^2 for most of the region west of the line on Map 2, and the contours of C(10) for this region strictly apply to catchments larger than this size.

8.2.2 <u>Possible variation of C(10) with catchment size</u>

As data were only available for catchments larger than 100 km^2 in the western part of eastern New South Wales, it was necessary to investigate the likely relation of these values to C(10) values for smaller catchments in this region. For the whole of the data for eastern New South Wales there was no significant relation between runoff coefficient values and catchment area, as discussed in Sections 8.1.4 and 8.1.5 above. Table I7 of Appendix I shows that for a regression of the deviation of derived values from mapped values of C(10) on the logarithm of area, the regression coefficient is only 0.022, which would be significant at only the 25% probability level. This indicates that overall, C(10) is virtually independent of catchment size. Longer records may indicate some relation in the future, but the lack of a trend probably results from the fact that although peak discharge per unit area decreases as catchment size increases, the mean rainfall intensity also decreases a corresponding amount as design rainfall duration increases.

However in the western part of eastern New South Wales where average rainfalls are low, there is a greater expectation that runoff coefficients would increase as catchment size decreases for two reasons. The first is that floods are more likely to result from isolated thunderstorms in inland regions in contrast to their frequent occurrence in general rains near the coast, as discussed by Pilgrim (1966) and noted in Section 8.1.1 above. As these thunderstorms generally cover only relatively small areas, a rapid decrease in discharge per unit area as catchment size increases would be expected for flood peaks in inland regions. Channel transmission losses provide the second reason. Although these also occur in humid regions in New South Wales (Baron et al. 1980), greater losses would be expected in drier inland regions where water tables are generally very low. This would also lead to an expected decrease in discharge per unit area and runoff coefficient values with increase in catchment size. As the derived values of C(10) west of the line on Map 2 virtually all apply to catchments larger than 100 km^2, the data were re-examined to determine whether they provide a satisfactory basis for design for this region.

Although the line on Map 2 does not closely follow an isohyet of median annual rainfall (MAR), possible trends in the relation of C(10) and catchment size with MAR were first investigated. The C(10) values from the 284 catchments were stratified into seven samples for the ranges of MAR of less than 700mm, 700-800, 800-900, 900-1100, 1100-1200, 1200-1500 and greater than 1500mm. Plots of C(10) against the logarithm of area showed no apparent trends for all samples with MAR above 800mm, although there was a slight apparent increase of C(10) with area for two of the samples and a decrease with another sample. For the sample of forty seven catchments with the lowest MAR of less than 700mm, there was a small trend of increasing C(10) values with decreasing area as expected although there was a large scatter. The catchments in this sample were from various parts of eastern New South Wales as well as from the western portion of this region.

Relationships for the thirty eight C(10) values west of the line on Map 2 were then investigated. A plot of C(10) values against logarithm of area again showed a slight trend of increasing C(10) with decreasing area, with values also increasing with MAR. Most catchments with C(10) values greater than 0.3 have MAR values greater than 750mm. However, a plot of C(10) against MAR showed a wide scatter. As there was no relation between C(10) and area for all of the data for New South Wales with all but the low range of MAR, only those C(10) values for catchments with MAR less than 750mm were then plotted against logarithm of area. This is shown in Figure 8.2, where the plotted points are labelled with MAR in cm. Again there is a trend of increasing C(10) values with decrease of area and increase of MAR. While the trend with decreasing area is apparent, it depends largely on the single value for area of 0.07 km^2 that is far removed from the remainder of the data. Even though this relation is a true regression model with the independent variable taking fixed values without the need to be normally distributed, the uneven spread of the data and the single low value of area make the derivation of a regression invalid. It is also evident that derivation of a relationship from the bunched data without the low value would be invalid.

Unfortunately, it must be recognised as a fact that there is not sufficient observed data available for the derivation of firm design information for catchments smaller than 100 km^2 west of the line on Map 2. It will not be possible to rectify this situation for many years until small catchments are instrumented and at least ten years of data are gathered in this region.

8.2.3 Development of an interim design relation between C(Y) and catchment size

As an interim measure, design data are developed below on the basis of the little relevant information available. However, these values are largely subjective and are based on reasonable assumptions and the desire to incorporate some conservatism by not adopting runoff coefficients with very low values. These design values are then of different nature to the values for the remainder of eastern New South Wales, as the latter are based strictly on observed flood data.

The bankfull discharge relationships for western New South Wales developed in Chapter 11 were used to provide some indication of the approximate relationship between C(10) and catchment area. Similar but more extreme conditions of the production of floods by small area storms and the occurrence of channel transmission losses would apply in the west of the State compared with the western part of eastern New South Wales under consideration here. In Chapter 11, the Y year discharge Q(Y) was found to vary with $(A.S_e)^{0.69}$. In Section 3.7 and Table 3.3, equal area slope S_e was found to be proportional to $A^{-0.31}$ for the Western Plains. Combining these two relations, Q(Y) is approximately proportional to $A^{0.48}$ with some scatter of values.

A relation was then found between rainfall intensity and area for Zone E of ARR covering most of the western part of eastern New South Wales. It is also assumed here that it applies to the linear interpolation zone of ARR in western New South Wales to which the bankfull relations of Chapter 11 apply. Calculations were based on a typical 12-hour, 2-year rainfall intensity of 4.5mm/h from Figure 2.18 of ARR, but the final exponent of area A which is of interest here does not depend on the base value of rainfall intensity selected. Rainfall intensities were calculated for a range of durations from 10 minutes to 12 hours, and the catchment areas that would have these times of concentration were calculated from equation 4.4. A log-log plot of these rainfall intensities and catchment areas was very close to linear, with intensity proportional to $A^{-0.239}$. Substituting this and the relation for $Q(Y)$ into equation 1.2 gave the relation that $C(Y)$ is proportional to $A^{-0.28}$.

The above relation is of a very approximate nature only and is based mainly on relationships for western New South Wales. In the entire region east of the line on Map 2, the observed data indicate that runoff coefficients are independent of catchment size. The western part of eastern New South Wales under consideration here may be considered to be a transition zone between these two regions. An exponent of area A between the values of -0.28 and zero has therefore been considered to be appropriate, and $C(Y)$ proportional to $A^{-0.15}$ has been adopted. As noted previously, this is really an arbitrary and subjective relation, and has been adopted as a reasonable basis for design in the absence of sufficient observed data to provide more definitive information.

8.2.4 Adopted design procedure for the western part of New South Wales

On the basis of the above considerations, the following design procedure has been adopted. Design contours are based generally on those on Map 2, but these are redrawn on a base map of the Division of National Mapping as Map 4 at the back of this report for convenience of use. West of the line joining Ashford - Tamworth - Bathurst - Yass - Tumut - Jingellic, the derived C(10) values for catchments with MAR less than 750mm were multiplied by $(A/100)^{0.15}$ to adjust them to correspond with a catchment area of 100 km^2, and the contours redrawn. Some smoothing of the contours has also been carried out relative to those on Map 2. Also, the minimum-valued contour shown on Map 4 is for C(10) of 0.10, as it seems reasonable and conservative to adopt this as a minimum design value in view of the limited observed data.

The western part of eastern New South Wales can then be considered to lie between the line noted above in the east and a line in the west formed by the boundary between rainfall zone E and the linear interpolation zone on Figure 2.17 in ARR. This western boundary approximately joins the towns of Mungindi, Nyngan, Condobolin Narrandera, and Tocumuwal. For catchments larger than 100 km^2 in this region, the values of C(10) from Map 4 should be used for design with a minimum value of 0.1. It is not worthwhile applying an area adjustment for catchments larger than 100 km^2, as this adjustment would be relatively small up to the recommended maximum size of 250 km^2 for the design method.

For catchment areas smaller than 100 km^2 in this western region, the value from the map should be used if it is greater than 0.40. If it is less than 0.40, the value from the map should be multiplied by $(100/A)^{0.15}$ where A is in km^2, but if this adjusted value is greater than 0.40, then 0.40 should be used. However, if this adjusted value is less than 0.20, a design value of C(10) of 0.20 should be used.

Adoption of the limiting values of 0.40 and 0.20 above is rather arbitrary. The latter has been adopted as conservative design practice, as the available data are inadequate to justify the use of very low C(10) values for small catchments. The value of 0.40 has been selected as most of the catchments with C(10) values greater than 0.30 to 0.40 have median annual rainfalls greater than 750mm, and the investigation showed that C(10) did not vary systematically with size for catchments with higher rainfalls. Also, the one very small catchment (410351) with long records in the western part of eastern New South Wales has a derived C(10) value of 0.43. It is recognised that a discontinuity will occur in runoff coefficient values for small catchments along many parts of the line marking the eastern boundary of the western region on Map 4. Where the C(10) values read from the map are less than 0.40, these values will be used for design for catchments east of the line. For small catchments west of the line, these values will be adjusted upwards by the factor $(100/A)^{0.15}$. The ceiling of 0.40 will limit the resulting disparity. Within the accuracy of the overall procedure for the western part of eastern New South Wales, a formal procedure for smoothing this discontinuity has not been considered to be justified. The values resulting from the procedure as described should be suitable for design. However, adjustment of the derived values by the individual designer to smooth the discontinuity would also be satisfactory.

The procedure described above for the selection of design 10-year coefficients in the western part of eastern New South Wales must be considered to be of an approximate nature. It will be difficult or impossible to make improvements until at least ten years of data are available from a considerable number of gauged catchments smaller than 100 km^2 in the region.

8.3 FREQUENCY FACTORS FOR Y-YEAR RUNOFF COEFFICIENTS

8.3.1 Approaches investigated

As noted in Section 6.4, the general procedure adopted for presenting design data for runoff coefficients of return periods other than 10 years was to determine frequency factors C(Y)/C(10), and to relate these systematically to location or to catchment characteristics. This procedure was adopted as being most likely to lead to overall consistency of the design data and to smoothing of sampling errors. Values of C(Y) were determined for every catchment and for return periods ranging from 1 to 100 years, the maximum value for a particular catchment being equal to or slightly larger than its length of record. The derived values for the 284 catchments are listed in Appendix J.

Analysis of the derived C(Y)/C(10) values or factors, denoted herein as FF_y, indicated that there were at least three general approaches to developing a systematic method for presenting the design data:-

(a) Determination of average values of FF_y for each return period Y years for the whole region or for several zones within the region of interest.

(b) For each return period, the values of FF_y for the individual catchments could be related to one or more physical characteristics by regression. This would result in a single equation for each value of Y for the whole region of interest.

(c) Combination of approaches (a) and (b), with separate regressions of FF_y on physical characteristics for each return period Y within each of several zones.

These three approaches are discussed in turn in the following sections, and then their results are compared and the design procedure is adopted. For the western part of eastern New South Wales, derived runoff coefficients rather than the design values discussed in Section 8.1.6 have been used.

8.3.2 Average frequency factors for zones

For the first of the three approaches outlined above, the average and standard deviation were calculated for all of the FF_y values for each return period within each of the five rainfall zones in Figure 2.17 of ARR. These zone boundaries are reproduced in Figure 5.2. Values from the Victorian and Queensland stations which fell in each rainfall zone were used in calculating the average FF_y values. However, there was no significant difference between the average FF_y values for zones A and B, and for zones D and E. The original five zones were thus grouped into the three zones (A + B), C and (D + E). The differences between these three zones were statistically significant.

The calculated average FF_y values were plotted on log-normal probability paper against Y for each zone, and a curve of best fit was drawn by eye, keeping within one standard deviation confidence limits plotted about each average value. The adopted FF_y values were taken from these lines of best fit to reduce the effect of sampling errors in the individual average values for each return period. The adopted average FF_y values for each of the three zones are listed in Table 8.2, and the numbers of stations used in calculating the averages for the zones and different return periods are shown in parentheses. The values can only be regarded as reliable up to a return period of 50 years. The 100-year values, which involve extrapolation of the flood frequency curves, have been included as rough guides only.

Table 8.2 Average Frequency Factors $FF_y = C(Y)/C(10)$ for Regions in Eastern New South Wales.

Rainfall Zones on Figure 2.17 of ARR are shown in Parantheses.

Key:- n - number of catchments used
$\overline{FF}$ calculated average frequency factor
δ - calculated standard deviation about $\overline{FF}$
FF - adopted frequency factor

Return Period (Years)	Central & Sthn Coastal (A + B)				Northern Coastal (C)				Eastern Interior & Nthn Tblds (D + E)			
	n	$\overline{FF}_y$	δ	FF_y	n	$\overline{FF}_y$	δ	FF_y	n	$\overline{FF}_y$	δ	FF_y
1	150	.62	.28	.60	52	.59	.26	.60	82	.49	.26	.49
2	150	.72	.29	.70	52	.75	.17	.73	82	.63	.20	.62
5	150	.85	.09	.86	52	.88	.11	.88	82	.81	.11	.82
10	150	1.00	-	1.00	52	1.00	-	1.00	82	1.00	-	1.00
20	116	1.12	.13	1.13	38	1.07	.09	1.08	60	1.17	.13	1.18
50	46	1.36	.30	1.36	26	1.22	.22	1.22	33	1.58	.42	1.48
100	29	1.55	.51	1.55	18	1.36	.31	1.36	26	1.74	.57	1.75

8.3.3 Regression relations for FF_y for all catchments

For approach (b) outlined in Section 8.3.1, average values $\overline{FF}_y$ of the frequency factors were calculated for each return period for all of the 284 catchments used for the eastern New South Wales study. The ratio FFR_y of the individual frequency factor FF_y to the overall average value $\overline{FF}_y$ was then determined for each catchment and return period. Regression relationships and correlations were then examined between FFR_y and a wide range of physical variables for each return period. The best relationships were obtained by multiple regressions with the three variables logarithm of area A, slope nonuniformity index S_a/S_e, and logarithm of the 12-hour rainfall intensity I (12, Y). Table I8 in Appendix I lists the overall average values $\overline{FF}_y$, the regression coefficients, the correlation coefficients (actually coefficients of multiple correlation) and statements whether the correlations are significant at the 5% probability level for these best relationships for each of the return periods. In most cases the regression coefficients are small, indicating a small effect of each independent variable, and the correlation coefficients are low in each case, indicating that a maximum of about 10% of the variance in FFR_y is explained by the relations. However, all of the relations are significant at the 5% probability level, so that their use would be justified in estimating design values of the frequency factor FF_y for a particular catchment.

The regression coefficients of the logarithms of the 12-hour rainfall intensities I(12,Y) are quite large for return periods of 50 and 100 years, indicating that this variable does have a relatively large effect on FFR_y for these two return periods. Where the rainfall intensities are high, the slopes of the flood frequency curves are relatively low and the floods thus exhibit relatively low variability. This is also reflected in the average frequency factors for the different zones in Table 8.2. The relatively high regression coefficient for log I(12,Y) also means that the shape of the frequency curves and hence the skew of the logarithms of the floods is related to the rainfall intensity. For example, skews tend to be relatively low on the North Coast where rainfall intensities are generally high.

8.3.4 Regional regression relations for FF_y

The third approach was a combination of the two previous approaches. Separate average values $\overline{FF}_y$ were calculated for each of the three regions of rainfall zones (A + B), C and (D + E) from Figure 2.17 of ARR or Figure 5.2 of this report. The value of FFR_y then applied to the ratio of frequency factor FF_y of a particular catchment to the average value $\overline{FF}_y$ for its region. Multiple regressions and correlations were then determined for each region of FFR_y on the logarithm of area A, slope uniformity index S_a/S_e, and 12-hour rainfall intensity I(12, Y). Details for the three regions are given in Tables I9, I10 and I11 in Appendix I. In general, regression and correlation coefficients were low, and the relations were either not or just significant at the 5% probability level. The signs of the regression coefficients were also inconsistent. Overall, it was concluded that use of regional regressions could not be justified statistically.

8.3.5 Comparison of approaches

The three approaches described above provide alternative procedures for evaluating the frequency factor FF_y (= C(Y)/C(10)), and hence the Y year runoff coefficient C(Y) from the value of C(10). From the values derived from the observed floods, the first two methods are valid statistically but present the data in quite different forms. Statistically, either could be used.

As noted above, use of the third approach is not justified statistically.

To aid in the selection of the approach to be adopted for the design method, the performances of the three approaches were compared for the 2 and 50 year return periods. Frequency factors were predicted for each of the catchments used in the study, and these were compared with the frequency factors derived from the data. The ratio of these frequency factors was calculated as

$$\Delta = [FF_y]_D / [FF_y]_P \tag{8.1}$$

where Δ = ratio of derived and predicted frequency factors

$[FF_y]_D$ = value of FF_y derived from observed data

$[FF_y]_P$ = value of FF_y predicted from particular approach

For the 2-year return period, data from the 284 catchments were available, while 105 values were available for the 50-year return period. Means and standard deviations of the frequency factor ratio Δ for each of the three approaches and the two return periods are given in Table 8.3.

Table 8.3 Comparison of Performance of Three Approaches for Determining Frequency Factors $FF_y = C(Y)/C(10)$

Approach (Sect. 8.3.1)	Procedure Used	Ratio Δ of derived & Predicted Frequency Factors			
		2-year Return Prd.		50-year Return Prd.	
		Mean $\bar{\Delta}$	Std. Devn.	Mean $\bar{\Delta}$	Std. Devn.
(a)	Zone averages	1.025	0.282	1.022	0.234
(b)	Overall regression on catchment variiables	1.000	0.269	0.996	0.234
(c)	Zonal regressions on catchment variables	1.023	0.274	0.997	0.210

The statistics for the three approaches are not significantly different to each other. The second and third approaches may be slightly more accurate, but any differences are very small, and there is no reason in the statistics in Table 8.3 for choosing any one approach in preference to the others. The choice seems to be between approaches (a) and (b), as the relations in (c) were not significant.

As the performances of the approaches were equal, the choice was based on the requirements for a practical design method outlined in Section 1.3. The regression relationships of approach (b) are rather complex and cannot be expressed in a simple graphical form for ease of use. They could thus hinder the adoption of the entire procedure in design practice. On the other hand, the zone average values are simple and easy to use. In addition, the frequency factors for different return periods in a particular zone are consistent with one another as a curve of best fit was drawn through a plot of the derived values against return period. Consistency is not assured between the different regressions derived for the various return periods.

On the basis of these considerations, the first approach using average frequency factors for each of the three zones (A + B), C and (D + E) was adopted for the design procedure. However, this was modified for return periods of 50 and 100 years where the logarithm of the 12-hour rainfall intensity I(12,Y) was shown to have a relatively large effect on FFR_Y. Simple relations to adjust the values of FFR_Y for the effects of this variable were developed from the average values $\overline{FF}_Y$ and the regression coefficients c in Table I8 of Appendix I. For convenience, the 50-year rainfall I(12,50) has been used in the relations for both return periods, as these values can be read directly from Figure 5.4 or from Figure 2.19 of ARR. The regression coefficients for the 100-year frequency factors were adjusted by average values of the ratio log I(12,100)/logI(12,50) for each zone. Very small variations occur in this ratio over each zone. The adopted relations for FFR_Y for return periods of 50 and 100 years give the zone average values of Table 8.2 for the average rainfalls within each zone, but adjust the values of FFR_Y for specific locations for the effects of the rainfall intensity I(12,50). Where I(12,50) is high, the frequency factor ratio FFR_Y is reduced, reflecting the lower slopes of the flood frequency curves in areas of high rainfall.

The frequency factors adopted for design are shown in Table 8.4. For return periods of 1 to 20 years, the values in Table 8.2 are reproduced, while the simple relations using $\log_{10}$I(12,50) have been adopted for return periods of 50 and 100 years. Again it should be noted that the 100-year values provide approximate guides only.

8.4 EXAMINATION OF THE DEPENDENCE OF RUNOFF COEFFICIENTS ON LOCATION

In sections 8.1 and 8.3, the concept has been adopted that both the 10-year and the Y-year runoff coefficients depend only on geographical location. This was not a fundamental premise of the study, but resulted from analysis of the large amount of observed data, and as discussed, the fact that significant relationships with other physical variables could not be found for either 10-year runoff coefficients or frequency factors.

It has generally been considered that the value of the runoff coefficient at a particular location should decrease as catchment size increases. This is based primarily on consideration of the Rational Method as a deterministic model. No evidence for any variation of this type was found in this study for eastern New South Wales where data were available from a wide range of catchment sizes. As discussed, no significant correlations were obtained in the regression studies, and no real trends were evident in the plots of C(10) against area for the stratified sets of mean annual rainfall described in Section 8.2.2.

This lack of dependence of runoff coefficient on catchment size was further investigated by examining the implied variation of discharge with catchment area for a constant value of runoff coefficient at particular locations. The approach involved finding a power relation between design rainfall intensity and catchment area. This type of relation was also used in developing the design approach for the western part of eastern New South Wales in Section 8.2.3. For each rainfall zone in Figure 2.17 of ARR, the average 12-hour, 2-year rainfall intensity was found from figure 2.18 of ARR as a base for calculations, although the final exponent of area A which is of interest here does not depend on this base value. Rainfall intensities were then calculated for each zone for a range of durations from 10 minutes to 12 hours, and the catchment areas that would have these times of concentration were calculated from equation 4.2. A log-log plot of these rainfall intensities and catchment areas for each zone was very close to linear.

The slope of the line of best fit and thus the exponent of area A varied with the zone factor AFACT.

Zone A in the south east with the lowest value of AFACT of 2.60 gave a nearly linear log-log plot with the lowest exponent of A. This yielded the relation

$$I \propto A^{-0.20} \tag{8.2}$$

Substituting this into equation (1.2) gives

$$Q(Y) \propto A^{0.80} \tag{8.3}$$

For the highest value of AFACT of 4.70 for Zone D on the Northern Tablelands, the log-log plot of I against A was not quite linear, but an average relation was

$$I \propto A^{-0.25} \tag{8.4}$$

Substituting into equation (1.2) gives

$$Q(Y) \propto A^{0.75} \tag{8.5}$$

From the slope of the log-log plot at large values of A of about 1000 km^2,

$$I \propto A^{0.27} \tag{8.6}$$

$$\text{and} \qquad Q(Y) \propto A^{0.73} \tag{8.7}$$

From the slope of the log-log plot at small values of A of about 0.1 km^2,

$$I \propto A^{-0.19} \tag{8.8}$$

$$\text{and} \qquad Q(Y) \propto A^{0.81} \tag{8.9}$$

Empirical power relationships between flood discharges and area have frequently been found from observed data. As discussed in Section 4.4.4, Alexander (1972) suggested

$$Q \propto A^{0.7} \tag{4.5}$$

Equations (8.3) and (8.5) for the average relations for the zones with the highest and lowest values of AFACT are in close agreement with equation (4.5). Also, it has been suggested that the exponent should be higher for small areas and lower for large areas. This is supported by the general shape of flood envelope curves, such as those of Creager (Creager, Justin and Hinds, 1945). The variation of exponent values for Zone D in equations (8.7) and (8.9) conforms with this trend.

These results show that a constant value of runoff coefficient for all catchment sizes at a given location yields the expected form of variation of flood discharge with catchment area. The results thus support the adopted approach that the design runoff coefficients can be related solely to location.

The validity of these conclusions is further re-examined in Appendix L in an extension of the above investigation. The approach used is quite different to that used in this chapter to develop the design information, although it uses the same basic data. The results are shown to support the conclusions of location dependence adopted here.

8.5 DISCONTINUITIES AT ZONE BOUNDARIES

Two types of discontinuity problems arise in the adopted design procedure when the location considered is close to a boundary of the rainfall zones in Figure 2.17 of ARR. The zones are reproduced in Figure 5.2. As ARR rainfall intensities for durations less than 12 hours change abruptly at these boundaries, equation (1.2) indicates that the derived runoff coefficient values should also change abruptly at the boundaries. It would have been possible to draw the C(10) contours on Map 2 and 4 at the back of this report with these discontinuities at the boundaries. However, this was not done as the overall acuracy of the data was not considered to warrant this refinement, and smooth contours were adopted.

A further discontinuity occurs at some of the boundaries with runoff coefficients and design floods of return periods other than 10 years. This results from the different frequency factors for zones (A + B), C and (D + E) adopted for the design procedure. In some cases the effects of the two discontinuities will tend to compensate one another, especially for return periods greater than 10 years, but in other cases the effects will be additive.

The practical solution adopted to the problem of these two types of discontinuities is to assume a linear transition of design flood magnitudes over a distance of 25 km each side of a zone boundary. The flood at any location within this 50 km transition strip is found by linear interpolation between the flood magnitudes at adjacent points in the two zones. This procedure is detailed and illustrated in Chapter 10.

8.6 ADOPTED DESIGN VALUES

To summarise the design runoff coefficient data adopted from the studies in this chapter, values of the 10-year runoff coefficient C(10) can be determined from Map 4 at the back of this report. For the region east of the line joining Ashford, Tamworth, Bathurst, Yass, Tumut and Jingellic, values of C(10) are read directly from the map. West of this line, design values of C(10) are read directly from the map for catchments larger than 100 km^2, with a minimum recommended value of 0.10. For catchment areas smaller than 100 km^2, the value from the map should be used if it is greater than 0.40. If it is less than 0.40, the map value should be multiplied by $(100/A)^{0.15}$ where A is in km^2, but with a maximum value of 0.40. However, if this adjusted value is less than 0.20, a design value of C(10) of 0.20 should be used. For all regions, some small-scale variations of coefficients from the values given by the contours may be justified on the basis of local conditions.

Runoff coefficients for return periods other than 10 years are obtained by multiplying the C(10) value for the location by an appropriate frequency factor FF_y. Values of FF_y for the three Zones (A + B), C and (D + E) from Figure 2.17 of ARR or Figure 5.2 are given in Table 8.4.

For all of the runoff coefficients discussed above, the duration of the design rainfall must be calculated by equation 4.2.

Flood magnitudes for locations within 25 km of a zone boundary are found by a linear interpolation procedure between values for the adjacent zones.

The steps in the complete procedure for estimation of design floods, and examples of its application, are described in Chapter 10.

Table 8.4 Adopted Design Values of Average Frequency Factors $FF_y = C(Y)/C(10)$ for Regions in N.S.W.

Region (ARR Rainfall Zones)	Frequency Factors FF_y for Return Periods of:-						
	1	2	5	10	20	50	100 very approx. only
Central and Southern Coastal (A & B)	0.60	0.70	0.86	1.00	1.13	$1.75-0.37\log_{10}I(12,50)$	$2.28-0.70\log_{10}I(12,50)$
Northern Coastal (C)	0.60	0.73	0.88	1.00	1.08	$1.67-0.37\log_{10}I(12,50)$	$2.20-0.70\log_{10}I(12,50)$
Eastern Interior & Northern Tablelands (D + E)	0.49	0.62	0.82	1.00	1.18	$1.84-0.37\log_{10}I(12,50)$	$2.42-0.70\log_{10}I(12,50)$

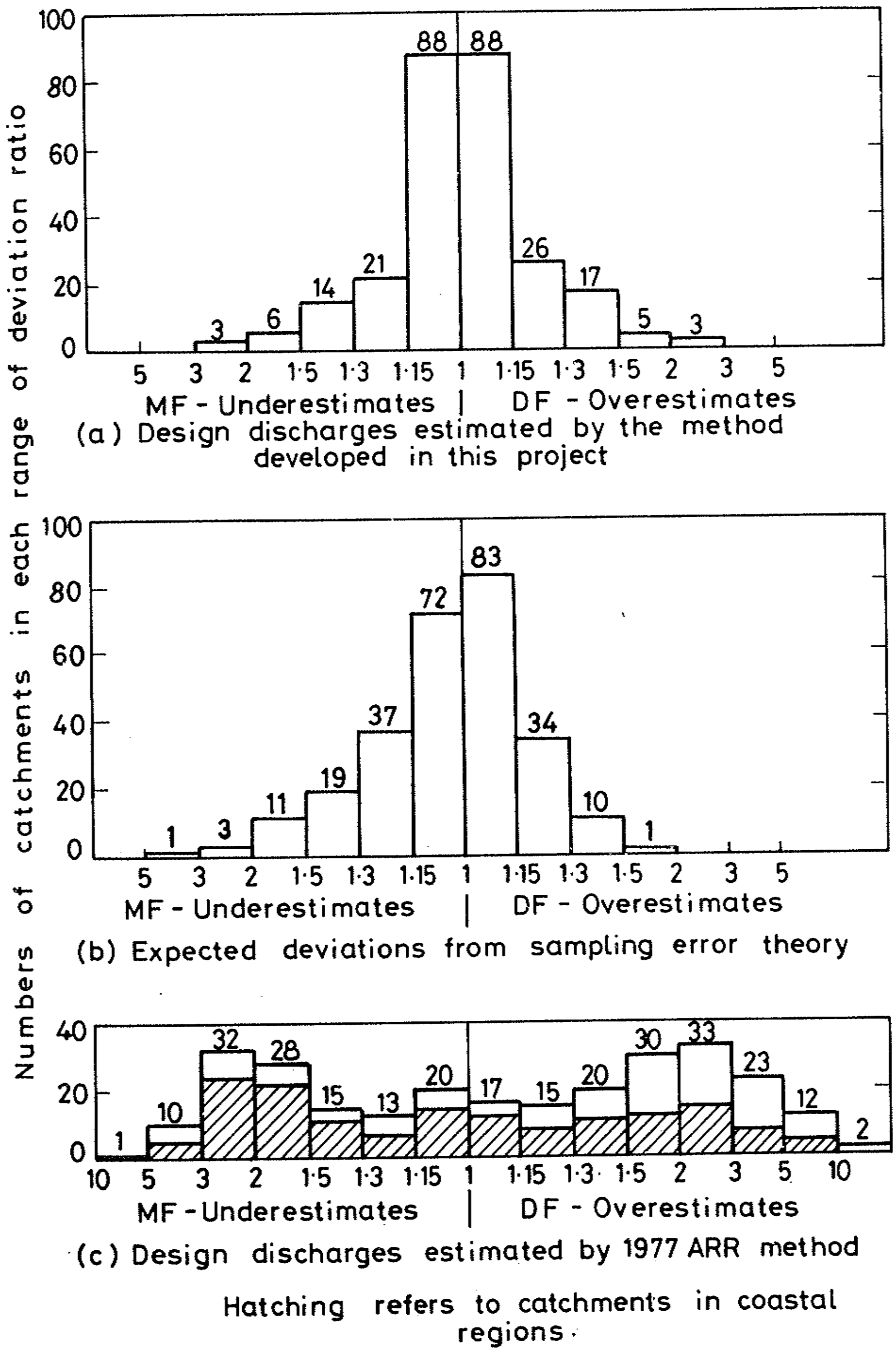


Figure 8.1 Frequency distributions of deviation ratios of estimates of 10-year flood magnitudes and runoff coefficients to actual values for 271 catchments in and adjacent to eastern New South Wales.

Note: The figure against each point represents median annual rainfall of the catchment in cm.

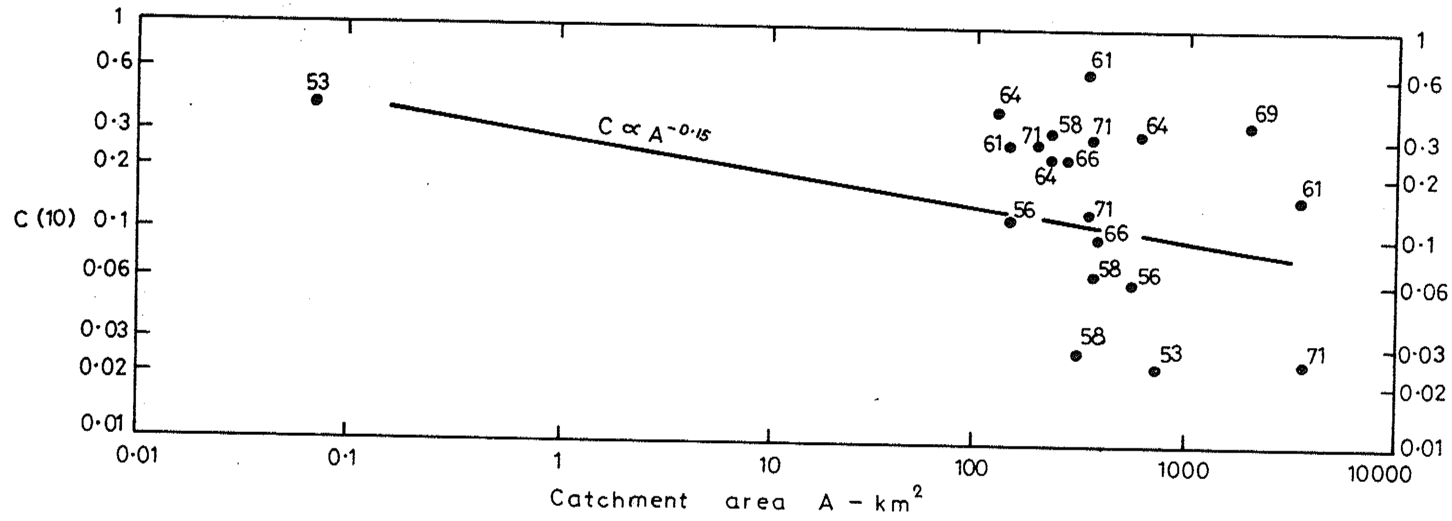


Figure 8.2 Relation of 10-year runoff coefficients to area for those catchments in the western part of eastern New South Wales with median annual rainfalls less than 750mm.

9. ECONOMIC SIGNIFICANCE OF USE OF THE DESIGN METHOD

9.1 COMPARISON OF RESULTS WITH THOSE OF OTHER METHODS.

9.1.1 Comparison with 'Australian Rainfall and Runoff'

As noted in Section 1.2, the Rational Method is used for the great majority of design flood estimes on small rural catchments (Pilgrim and Cordery 1980). For New South Wales, design data from both ARR and the manual of the Department of Main Roads (1976) are used in practice. The examples given by Pilgrim (1978) and Cordery and Pilgrim (1980) show that these and other current design procedures give runoff coefficients and flood estimates that differ considerably from one another, and that deviate widely from observed floods.

These comparisons are extended in this subsection, where estimates from ARR and the data derived in this study are compared. In Section 9.1.2, comparisons with other design methods are discussed, particularly that of Cordery and Webb (1974). The economic significance of the deviations of design data from the observed values is considered in the remainder of the Chapter, and the probable savings that would result from the design procedure derived in this project are assessed.

The design procedures of ARR were used to estimate floods of various return periods for the 271 catchments used in the comparison to select the design method (Section 7.3) and in the study of expected random scatter (Section 8.1.4). These were the catchments in eastern New South Wales, those in Queensland (all of which are close to the border), and those in Victoria close to the border. Times of concentration were calculated by the Bransby Williams formula as recommended in ARR, using the average slope S_a as in the original method. A major problem in selecting the runoff coefficients from Figure 7.3 of ARR was the choice of the appropriate design curves specified in terms of idealised catchment characteristics. The choice was based on the available information, which was of similar standard to that often available in practical design. The curves for clay and medium soil were used in all cases. For the majority of catchments, use of the curves for sandy soil would have led to runoff coefficients of zero. Even the curves for medium soil would have given zero coefficients in many cases for all return periods from one to one-hundred years for all but very small catchments, and it was often necessary to use the curve for clay and rock regardless of the actual soil type to obtain a non-zero runoff coefficient. This constitutes a major practical problem with the ARR data, especially for larger catchments in areas of relatively low rainfall. It was also noted that almost all of the flood frequency curves constructed from floods estimated from ARR had lower slopes that those derived from observed floods at the stations considered. The estimated values also exhibited slightly negative skew coefficients, whereas most of the flood series derived from observed data were positively skewed.

The 10-year return period was used for the basic comparison between floods estimated using ARR and those derived from observed data. Ratios of these floods were calculated for all catchments. These ratios correspond with those used for runoff coefficients in equations (7.1) and (7.2) and in Chapter 8. Reciprocal ratios were used for under-and over-estimates to keep all values above unity and to indicate comparable spreads of values for under-and over-estimates. The ratios were calculated as:-

$$\text{Underestimate - MF} = \frac{Q_o}{Q_e} \qquad (9.1)$$

$$\text{Overestimate - DF} = \frac{Q_e}{Q_o} \qquad (9.2)$$

where MF = factor by which to multiply Q_e to obtain Q_o.

DF = factor by which to divide Q_e to obtain Q_o.

Q_o = 10-year flood magnitude derived from observed data.

and Q_e = 10-year flood magnitude estimated from ARR.

A frequency diagram of the numbers of catchments in various intervals of MF and DF is shown as Figure 8.1 (c) in the previous chapter. The results have been shown in that figure to facilitate comparison with the estimates of the 10-year floods from the design data developed in this project (Figure 8.1 (a)), and with the deviations that would be expected from the sampling errors in the original observed data (Figure 8.1 (b)). As noted earlier, the design estimates on which Figure 8.1 (a) were based were derived from a preliminary map of contours of C(10), which was later amended in some areas to the configuration on Maps 2 and 4 at the back of this report. However, these amendments would make little difference to the distribution of the design estimates. The comparison between the floods estimated by ARR and by the method developed in this report is biased as a result of the fact that the latter estimates are for the same catchments from which the design data were derived. Better results should therefore be expected for these catchments from the method developed herein. However, comparison of Figures 8.1 (a) and (c) shows that the accuracy of the method derived in this report is far better than that of the design data in ARR, which gives grossly inaccurate results. This supports the results in the examples given by Pilgrim (1978) and Cordery and Pilgrim (1980). The examples in these papers also indicate that similar poor results would be given by the design data in the manual of the Department of Main Roads (1976) and other current design procedures.

The shaded area in Figure 8.1 (c) gives the distribution of the deviations of the ARR estimates for the catchments in the coastal region of New South Wales. This indicates that the ARR results are not only inaccurate, but that there is also a bias towards underestimation of observed floods near the coast. However, this does not occur with all catchments, and there is a very wide spread of deviations for both coastal and inland regions, with under-and over-estimates for some catchments in each region,

The above results were extended by estimating the 2 and 50-year floods by the ARR procedure for 75 catchments with flood records of high reliability and selected from as many river basins and catchment types as possible. The overall reliability of each flood frequency curve was represented by a reliability index defined and discussed in Appendix D, and the indices for all of the catchments used in the study are listed in Appendix A. For the selected 75 catchments, those in coastal river basins had reliability indices of 2 or better, while those on the tablelands and west of the divide generally had indices of 3 or better. In a few cases it was necessary to select catchments with an index value of 4 to obtain representation of all regions. Ratios of the 2 and 50-year estimates to the values obtained from the frequency curves fitted to the observed flood series were calculated by equations (9.1) and (9.2), as had been done for the 10-year floods. The multiplying factor MF and its inverse ratio, dividing factor DF, were again used to give comparable spreads of under-and over-estimates to aid comparison. For the 2 and 10-year estimates, values were available from all 75 catchments, while 59 values were available for the 50-year floods. Frequency distributions of the deviation ratios are presented in Figure 9.1. The same ranges of deviation ratio are used as in Figure 8.1. A wide spread of deviations similar to that for the 10-year floods for the larger sample of 271 catchments was obtained for each of the return periods.

In fact, the spread was not reduced by restriction of the comparison to catchments with relatively reliable records, indicating that the reason for the deviations is the inaccuracy of the ARR design data.

The shaded portions of the frequency histograms in Figure 9.1 represent the catchments in the coastal region. This again demonstrates the bias of under-estimation of observed floods near the coast and over-estimation west of the Divide.

Numbers of catchments and average values of the deviation ratios MF and DF for the separate groups of under-and over-estimates for each return period are shown in Table 9.1. The approximate overall averages of 2 and 3 for all of the under-and over-estimates respectively again demonstrate the poor accuracy of the design data in ARR. Although there was a bias towards over-estimation at all return periods for the particular sample of 75 catchments, relatively fewer over-estimates occurred at the higher return period of 50 years. This reflects the fact that the frequency curves of floods estimated by ARR had lower slopes than those derived from observed floods, as noted earlier.

Table 9.1 Average Deviation Ratios of Flood Estimates from ARR Relative to Observed Floods for 75 Catchments with Flood Records of High Reliability

Return Period (yrs)	Under-estimates		Over-estimates	
	No. of Catchments	Av. Dev. Ratio $\overline{MF}$	No. of Catchments	Av. Dev. Ratio $\overline{DF}$
2	30	2.01	45	3.62
10	33	1.87	42	2.92
50	30	2.07	29	2.64

The results presented in this Section indicate that the ARR design data are of low accuracy, do not reproduce the observed data, and give biased estimates with regard to distance of the location from the coast. Despite the lack of independence in the comparison of the design methods noted earlier, there is little doubt that the design data developed in this report are of much greater accuracy than those in ARR and other current design procedures. The derived data reproduce all of the available observed flood data with as much accuracy as is possible in view of the scatter inherent in the available sample of observed data.

9.1.2 Comparison with other methods

Webb and O'Loughlin (1981) have compared results of flood estimates by all available design methods on a selection of thirty six catchments in New South Wales. The comparison was between the different estimates rather than with observed flood data. On this basis, Webb and O'Loughlin questioned the accuracy of the procedure developed in this report.

Several comments are relevant to this criticism. In the comparison, only an interim version of the procedure developed in this report was used, as described in Pilgrim and McDermott (1980). Subsequent revision of the procedure would reduce considerably the differences between flood estimates by the procedure and the other methods used in the comparison, especially in the limited geographical regions in which the comparisons were made. An incorrect measure of slope was used by Webb and O'Loughlin for the Bransby

Williams formula for time of concentration in the comparison for the methods of ARR and French et al. (1974), although the correct measures unfortunately are not specified in these publications. Use of the correct overall average slope in these methods would have increased the flood estimates and brought them closer to the estimates of the method described in this report.

One of the methods recommended by Webb and O'Loughlin (1981) on the basis of their comparison was that of Cordery and Webb (1974). An assessment of the accuracy of this method was carried out by applying it to the 271 catchments used in Section 9.1.1. Deviation ratios of estimated 10-year flood magnitudes to the observed 10-year floods are shown in Figure 9.2. The ratios are the same as those used in Section 9.1.1 above, and the ranges of the ratios in Figure 9.2 are the same as those in Figures 8.1 and 9.1. Although the distribution of deviation ratios in Figure 9.2 is centred on unity, there is a wide scatter of results with two peaks. The scatter is less than for the ARR method in Figure 8.1 (c), but there is a similar tendency for estimates to be too low near the coast and too high inland. In concept, the Cordery and Webb (1974) method has the potential for greater accuracy than most other procedures including regional frequency methods such as the statistical Rational Method. The unit hydrograph component of the Cordery and Webb method was based on a considerable amount of data and has been supported by more recent data (Baron et al. 1980). The problem seems to be in the evaluation of losses, which were formulated from a much smaller data base. Design rainfalls from ARR may also contribute to the inaccuracies.

The inaccuracies of the Cordery and Webb method, and the problems with the incorrect measure of slope in the ARR and French et al. (1974) methods, illustrate the fact that the most appropriate basis of assessment of accuracy is comparison of results with observed data, rather than comparison with results of other procedures.

The distributions in Figures 8.1, 9.1 and 9.2 based on all available observed data indicate that the procedure described in this report is of greater accuracy than other available methods, and that appreciable improvements in accuracy will be unlikely until longer and more numerous streamflow records are available.

9.2 ECONOMIC SIGNIFICANCE OF IMPROVED DESIGN DATA

9.2.1 Introduction

While the comparisons in the previous sections of the accuracies of the ARR and other procedures and the derived data are of technical interest and importance, the costs of the construction and performance of structures designed by the different approaches are of greater practical importance. As noted in Section 1.2, the average annual expenditure on works sized by design flood estimates on small rural catchments in Australia as at 1979 is about $180 million (Cordery and Pilgrim 1979, 1980). Greater economic efficiency resulting from more accurate design estimates could thus result in large savings of public money. This section attempts to estimate the costs resulting from the inaccurate design information in ARR and the savings that could be achieved with the design data derived in this project. However, considerable assumptions are necessarily involved with the costs data currently available. In the analysis, it will be assumed that the optimum return period for design that will give minimum overall costs is known. In practice, the design return period is normally selected arbitrarily, and it may differ appreciably from the optimum. This problem is discussed briefly in Section 9.2.7.

9.2.2 Costs and damages relations

The general form of the relationship between total costs and design return period for a given structure is illustrated in Figure 9.3. The total costs are the sum of the capital costs and the present worth of the expected flood damage costs over the design life of the structure. In this, the term "expected" is used in the statistical sense and damages are weighted by their probabilities of occurrence in calculating the expected damage curve. Little definitive information is available on costs. Capital costs are readily quantifiable and represent the initial outlay for the design and construction of the structure. Even here, little general information is available. Expected damage costs have often been considered to be almost impossible to quantify. However, Polin (1978) and Polin and Cordery (1979) have estimated damage costs for two crossings of small streams in eastern New South Wales. Although various assumptions were involved, these estimates give at least the order of expected damages, and indicate that it would be possible to develop a larger body of data covering a wide variety of conditions that would provide a relatively firm basis for economic analysis. The data of Polin (1978) and Polin and Cordery (1979) have been used here to estimate expected flood damage costs.

9.2.3 Effect of inaccuracy of flood estimates on costs

Figure 9.4 illustrates typical over-and under-estimates of design flood discharges by the ARR procedure. The flood frequency curves from observed floods and the ARR estimates are shown. For a return period of 10 years, the ratios of the estimated Q_e and Q_o from the observed floods for the two examples on Figure 9.4 are close to the average deviation ratios in Table 9.1. The diagrams show that the actual return periods Y' of the estimated floods are 63 and 2.4 years. Reference to the general form of the relation between total costs and return period in Figure 9.3 shows that if the optimum return period had been 10 years, the actual costs of the structures sized on the basis of the estimated floods would be considerably greater than the optimum costs aimed at in the designs.

Two analyses to obtain semi-quantitative information on costs resulting from inaccurate estimates of design floods are described in Sections 9.2.4 and 9.2.5 below. The first uses the examples of Polin (1978) and Polin and Cordery (1979), while the second is more comprehensive and combines the damage data of the above authors with capital cost information from the N.S.W. Department of Main Roads. Both approaches involve various assumptions and are thus of only a semi-quantitative nature.

For both approaches, it was necessary to use a simple relationship between flood magnitude and return period. This facilitated computation of the actual return period Y' of the inaccurate estimate Q_e of the design flood of return period Y years. This approximate and simple relation developed in Appendix M is

$$Y' = \left[2^n + 1.4 \left(\frac{Qe}{Q(2)} - 1\right)\right]^{\frac{1}{n}} \qquad (9.3)$$

where Q(2) = the flood of 2-year return period from the frequency curve derived from observed floods.

and n = an exponent calculated by equation (M.6) from the observed flood frequency curve for the station.

9.2.4 Examples of Polin (1978)

Two examples of economic analysis of structures on road crossings of streams are given by Polin (1978) and summarised in Polin and Cordery (1979). One was on an access road to a residential and industrial area over South Creek at The Kingsway at Penrith west of Sydney with a catchment area of $260km^2$. The second was at the motorway crossing of Double Crossing Creek north of Coffs Harbour on the north coast of New South Wales, with a catchment area of 5 km^2.

Based on the economic optimum design floods given by Polin (1978) and Polin and Cordery (1979), flood estimates were considered that gave under-and over-estimate deviation ratios equal to the class interval divisions used in Figures 8.1 and 9.1. Equation (9.3) was used to estimate the corresponding return periods of these flood magnitudes, and the total cost curves given by the above authors were then used to determine the costs of the structures that would be required to pass each of the flows. The additional costs to those for the optimum solutions are shown in Figure 9.5. The costs of the optimum designs are approximately $320,000 for South Creek and $105,000 for Double Crossing Creek. With the average deviation ratios from Table 9.1 of 2 and 3 for under-and over-estimates by the ARR procedures, the likely additional costs indicated on Figure 9.5 as a result of inaccuracies of flood estimates using ARR are very high. In two of the four cases they exceed the cost of the optimum solutions.

For the design method developed in this project, the average deviation ratios from Figure 8.1(a) for under- and over-estimates of the 10 year floods are 1.20 and 1.19. Assuming these are representative of all return periods, Table 9.2 gives the additional costs from Figure 9.5 for design flood estimates deviating average amounts from the actual floods of the design return period for the ARR procedure and the design method developed in this project. Costs are given for the two examples

Table 9.2 Additional costs resulting from average deviations of ARR and derived method flood estimates - Polin's (1978) examples

Design Method	Under- or Over-estimate (Av. deviation ratio)	South Creek	Double Crossing Creek
Optimum design	-	$320,000	$105,000
ARR	Under $\overline{MF}$ = 2	$165,000	$130,000
	Over $\overline{DF}$ = 3	$470,000	$ 10,000
This report	Under $\overline{MF}$ = 1.20	$ 20,000	$ 10,000
	Over $\overline{DF}$ = 1.19	$ 40,000	

As discussed earlier, the error analysis for the design data developed in this project is not independent as the data are tested on the same catchments from which they were derived. Despite this, the costs listed in Table 9.2 indicate that very large costs result from the inaccuracy of the ARR design data with typical structures, and that these costs would be greatly reduced by the more accurate design method developed in this project.

9.2.5 Comparison of costs for 174 catchments

The second comparison of the costs resulting from inaccuracies in the design methods of ARR and of this report was more comprehensive and involved 174 of the gauged catchments used in the project. The use of this number out of the total of 284 catchments in eastern New South Wales resulted merely from convenience in application of the analysis as discussed later. However, the 174 catchments used in the analysis provide a large sample.

A detailed description of the analysis is given in Appendix N, and only a summary will be given here. The approach used was to carry out an approximate economic analysis of structures at the outlets of each of the 174 catchments using generalised cost relationships. Several simplifying assumptions were also made to facilitate the analysis. As a result of these generalisations and assumptions, the results can only be considered to be of an approximate nature, but they indicate the relative costs involved in use of the design methods.

Data on capital costs of seventeen typical small bridges and culverts were supplied by the Department of Main Roads of N.S.W. With costs estimates given by Polin (1978) and additional estimates of costs of small structures, a relationship between capital costs and design discharge was developed. The relationship is only of an approximate nature and the individual values show considerable scatter, as many variables affect the cost of a bridge and its capacity to pass flood flows. These include the allowable velocities and afflux at the bridge, the stream cross-section,the overall size of the bridge,its skew, the type of superstructure and the types of piers and abutments. Despite the scatter in the values, the relationship gives the order of capital costs, and is considered to be valid for use in the semi-quantitative analysis described here. The relationship between capital costs and design discharge was approximated by two linear segments, one for flows less than 100 m^3/s and the other for flows from 100 to 1000 m^3/s. These relations, also given as equations (N.1) and (N.2), were

$$100\ m^3/s < Q(Y) < 1000\ m^3/s: \quad CC\$ = 279\ Q(Y) + 118000 \tag{9.4}$$

$$Q(Y) < 100\ m^3/s: \quad CC\$ = 1460\ Q(Y) \tag{9.5}$$

where CC\$ = capital cost in dollars at December 1979

and $Q(Y)$= design discharge (m^3/s) of return period Y years.

Little information is available on costs of expected damages, and as noted in Section 9.2.2, the data used here were obtained from the two case studies described by Polin (1978) and summarised by Polin and Cordery (1979). The following relation (also given as equation N.4) was developed from these data:-

$$D\$ = \frac{B}{Y} \tag{9.6}$$

where D\$ = present worth of expected damage costs in dollars at December 1979 for a structure designed to pass the Y-year flood.

and B = a parameter depending on the particular site and the optimum return period.

The total costs for a given site covering a range of floods of various return periods were then

$$TC\$ = CC\$ + D\$ \tag{9.7}$$

where TC\$ = total cost in dollars at December 1979.

As the 10-year return period had been used as a base throughout the study, it was assumed that this was the true optimum return period for each of the study catchments. This is also the approximate average design value currently used in Australia for minor bridges and culverts (Pilgrim and Cordery 1980). For each catchment, the value of B in equation (9.6) and hence (9.7) was found that gave a minimum value of TC\$ at a return period of 10 years.

For each of the gauged catchments used in the study, it was assumed that the value of the 10-year flood derived from the frequency curve of observed floods was the true value. 10-year discharges were then estimated by the design methods of ARR and of this report. The latter estimates were again based on a preliminary map of C(10) contours, but the subsequent amendments incorporated in Maps 2 and 4 at the back of this report would make little difference to the results. The estimates of the 10-year discharges generally deviated from the true values, and their 'true' return periods from the observed flood frequency curve for the particular catchment were estimated from the simplified relation of equation (M.9) of Appendix M, reproduced above as equation (9.3). The costs of the structures designed by these inaccurate estimates as well as by the optimum 10-year flood were then calculated by equation (9.7).

Computations were less complex when all three discharges for a given catchment were within one of the linear capital cost ranges either above or below 100 m^3/s. This occurred with 174 of the catchments, and as these gave a large and representative sample of structures on small catchments, the analysis was only carried out for these sites.

Based on the true values of the assumed optimum 10-year floods, the sum of the total costs of the structures for the 174 structures was approximately \$34 million, made up of \$28 million capital costs and \$6 million costs of expected damages. The additional costs resulting from the inaccuracies of the discharges estimated by the design methods in ARR and in this project were \$15 million and \$2 million respectively. Table 9.3 (which is reproduced as Table N1 in Appendix N) expresses these basic and additional costs in terms of average values per km^2 of catchment area and per m^3/s of the observed flood frequency discharge. While such unit measures may be over-simplified in that costs are not linear functions of catchment size or discharge, they provide useful and convenient indicators.

Table 9.3 Average Costs of Structures Sized by Different Design Methods

Design Approach	Cost per km^2 of catchment area	Cost per m^3/s of observed flood frequency discharge
Basic cost of structure sized by true 10-year flood.	$1500	$790
Additional costs-design by ARR	$ 660	$350
Additional costs-design procedure of this project.	$ 80	$ 40

9.2.6 Summary of comparison of costs

The results of the two analyses described in Sections 9.2.4 and 9.2.5 are biased to some extent in that the costs were estimated for many of the catchments used for the derivation of the design data developed in this project. In addition, the analyses are of only an approximate nature and involve several assumptions. Despite these difficulties, the general orders of magnitude of the two analyses are similar and indicate that the design procedure developed in this project should lead to large savings compared with use of the ARR procedures. While the latter was the only currently available method tested, the comparison of flood estimates in Pilgrim (1978) and Cordery and Pilgrim (1980) indicates that similar results would apply to other methods with design data based on judgement rather than on observed data, such as the procedures of the Department of Main Roads of NSW (1976).

The additional costs that result from inaccuracies in design flood estimates will obviously vary from site to site, as illustrated by the two examples in Table 9.2 and Section 9.2.4. However, the results of the analyses described in Sections 9.2.4 and 9.2.5 and summarised in Tables 9.2 and 9.3 indicate that on the average, inaccuracies in flood estimates by the ARR method increase total costs by nearly 50 per cent of the optimum costs. As structures are currently designed by the ARR and similar methods, about one third of present costs would thus be saved if accurate flood estimates could be made. Corresponding figures for the design method developed in this report are about 10 per cent for both cost increases and savings. Of more practical importance, use of the design procedure developed in this project in place of the ARR and similar methods would result in savings of about 30 percent on present costs.

While the above figures are necessarily approximate, they represent large savings and expenditures. No specific data are available for eastern New South Wales, but on the basis of the figures given by Pilgrim (1978) and Cordery and Pilgrim (1980), the average annual expenditure in this region as at December 1979 on waterway crossings of rural roads would be about $30 million, and a further $10 million would be spent on other structures on small catchments such as spillways of farm dams. These figures are based on works expenditure and should thus reflect both capital and repair costs. Other costs of expected damages are not included, but the results of the analyses reported here indicate that these costs are relatively small.

For the costs of optimum designs for the 174 catchments, the costs of expected damages were only $6 million or 18 per cent of the total costs of $34 million. Exclusion of the repair costs would mean that the other costs of expected damages would be small. This is also indicated by the economic analyses of Polin (1978) and Polin and Cordery (1979). However, omission of these costs means that the above figures tend to be slightly low. Thus replacement of current procedures for estimating design floods for small structures in eastern New South Wales by the method developed in this project should lead to annual savings of about $9 million on road structures and $3 million on other works. This would be achieved with no loss of safety or reliability, because the inaccuracies of present methods lead to over- and under-estimates, both of which increase total costs (see Figure 9.3).

9.2.7 Effect of lack of knowledge of optimum return period

One of the assumptions in the second and more comprehensive of the above cost analyses merits additional comment. This is the assumption that the optimum return period is known. In practice, the design return period is generally selected rather arbitrarily and may be considerably different from the optimum. It is then possible that an inaccurate flood estimate could be closer to the true optimum than an accurate estimate and hence lead to lower total costs.

It seems unlikely that this would greatly affect the validity of the conclusions in the preceding section. The 10-year return period is close to an average value for small structures used at present in Australia. Actual return periods of estimated 10-year floods cover a very wide range, as illustrated by the values of 2.4 and 63 years for the typical examples in Figure 9.4, so that it is likely that many of the estimated floods deviate widely from the true optimum values. Also, the results of the first analysis, which does not involve assumed optimum return periods, are similar to those of the second analysis.

Although it seems unlikely that the general conclusions of this study are affected by lack of knowledge of the true values of the optimum return periods, investigation of the effects would be desirable. This was not possible in the time available in this project.

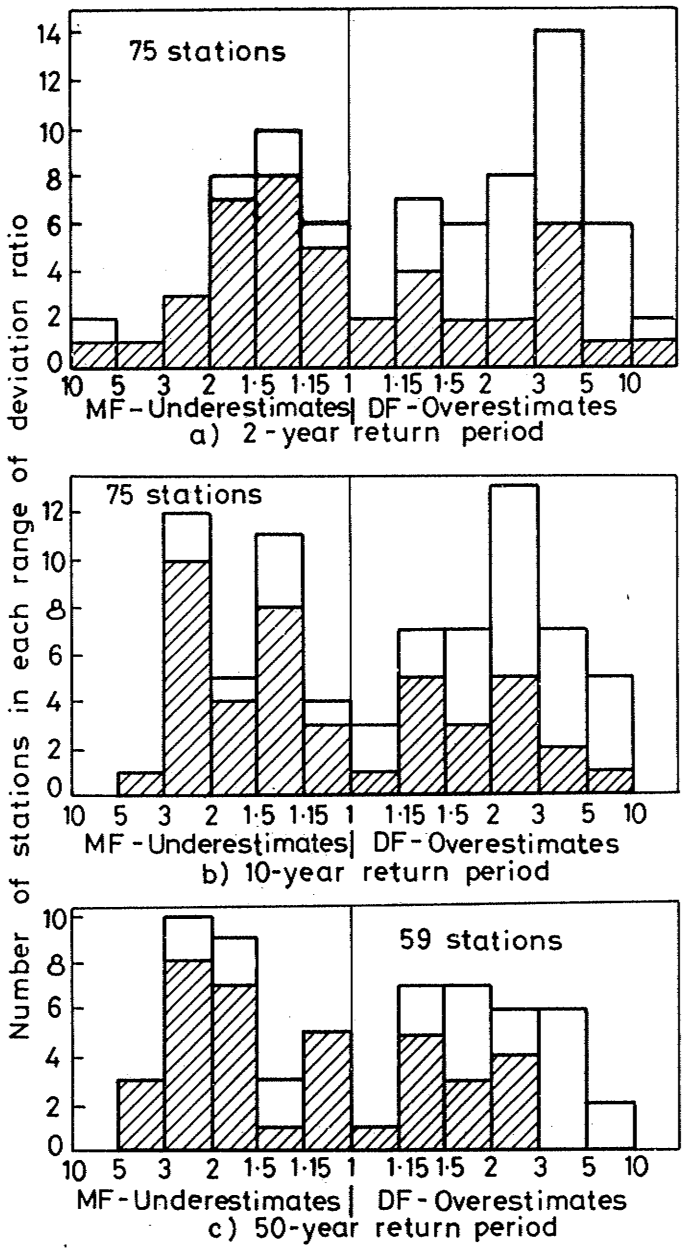


Figure 9.1 Frequency distributions of deviation ratios of observed floods to estimates by ARR for 75 catchments with reliable records. Hatching refers to catchments in Coastal regions.

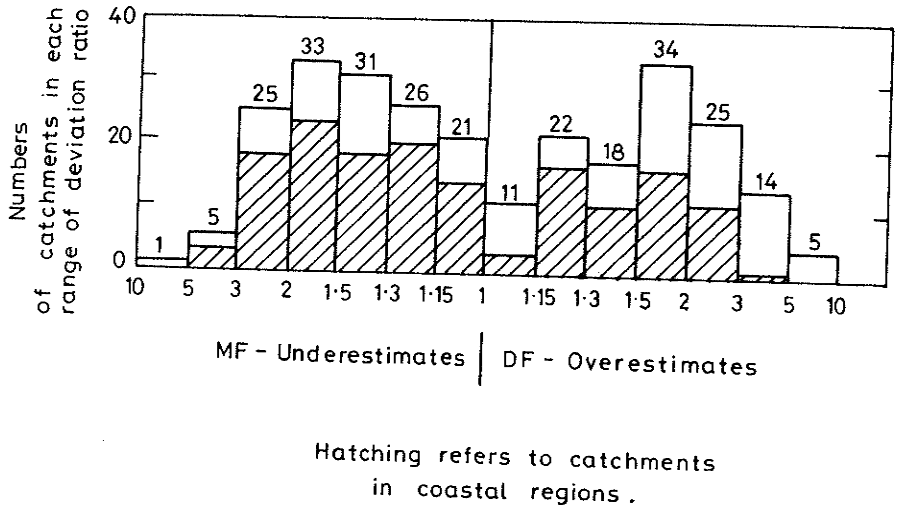


Figure 9.2 Frequency distribution of deviation ratios of estimates of 10-year floods by method of Cordery and Webb (1974) to actual values for 271 catchments in and adjacent to eastern New South Wales.

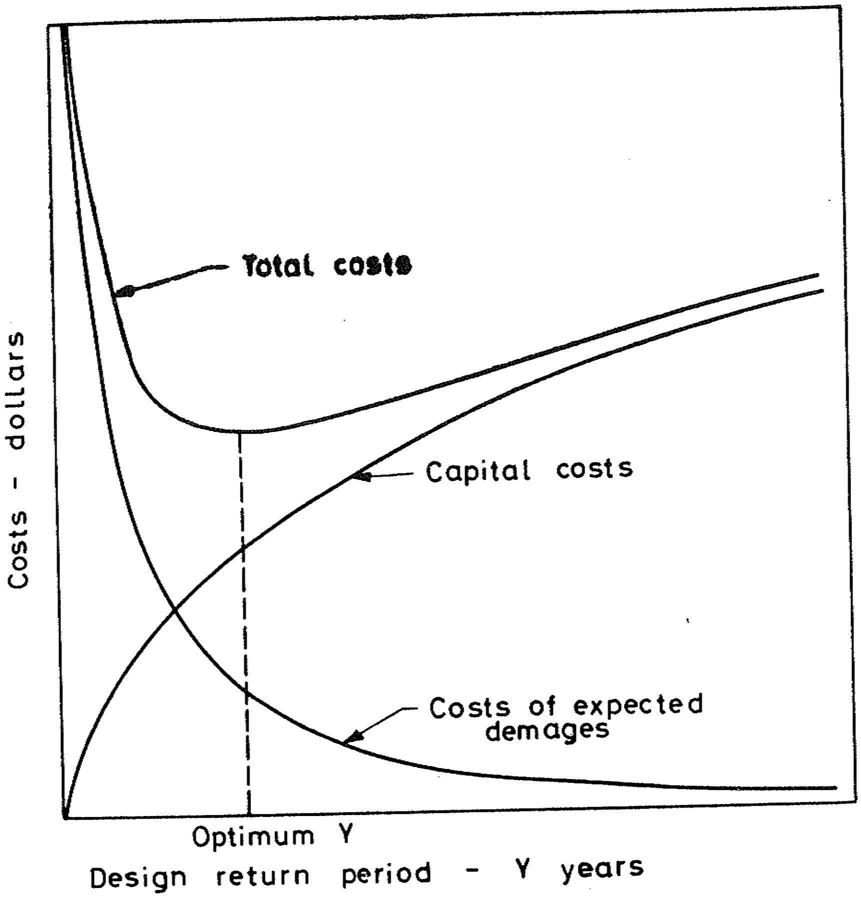


Figure 9.3 Form of the relationship between costs of a waterway structure and design return period.

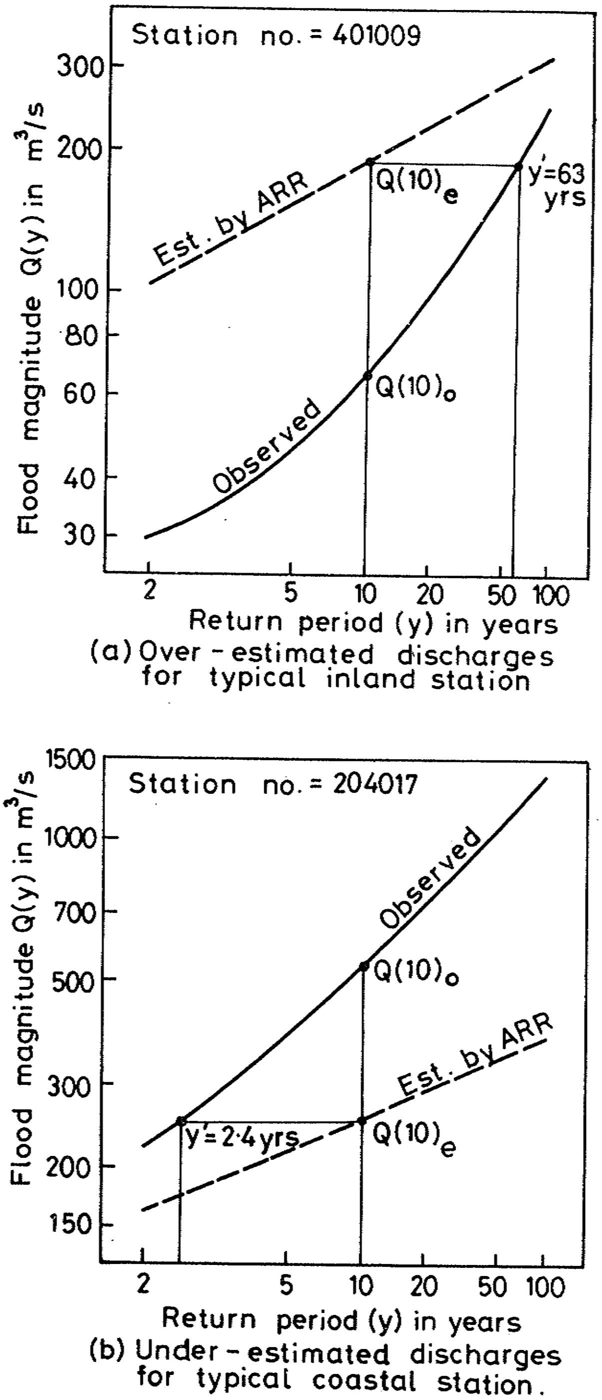


Figure 9.4 Typical frequency curves of observed floods and estimates by ARR.

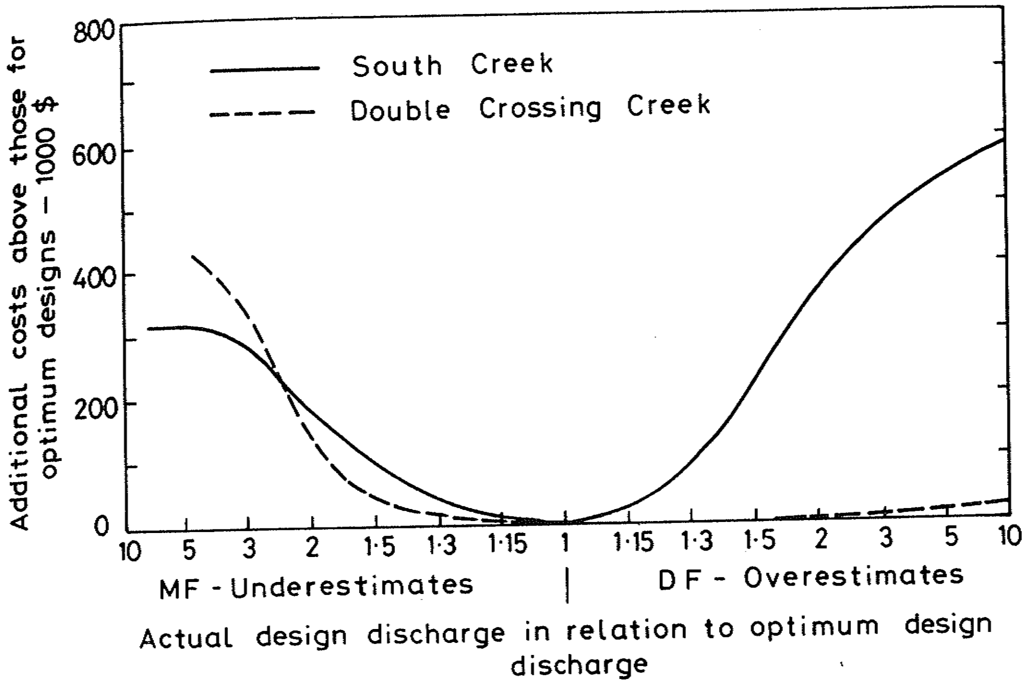


Figure 9.5 Additional costs above those for optimum design of two typical structures of Polin (1978) resulting from inaccuracies in design flood estimates.

10. APPLICATION OF THE DESIGN METHOD

10.1 SCOPE

The design procedure for flood estimation developed in this project is applicable to eastern New South Wales. The areal coverage of the method is shown on Maps 2 and 4 at the back of the report. An approximate method of flood estimation for western New South Wales is described in Chapter 11.

The design procedure is primarily applicable to catchments up to 250 km^2 in area and for return periods of one to fifty years. In the statistical interpretation of the Rational Method, the constraint of small size of catchments does not apply to nearly the same extent as with the more traditional deterministic interpretation. Also, no significant trend of variation of runoff coefficients with catchment size was detected in this study. Use of the procedure with larger catchments might thus be reasonable for preliminary design, but a limit of 250 km^2 for final design is recommended. The lengths of the flow records for most of the gauged catchments used in developing the method indicate that it should only be applied for return periods up to 50 years. Frequency factors have been given for a return period of 100 years, but these should be regarded as approximate only. They may thus be used to give an approximate estimate, especially in those practical cases where an estimate is required and no other design data are available. The same caution with regard to the 100-year return period applies to at least some extent with all methods of flood estimation. As discussed in more detail in subsequent sections, the procedure is simple to apply and involves similar calculations to currently-used applications of the Rational Method.

10.2 DESIGN RUNOFF COEFFICIENTS

The derived 10-year runoff coefficients for the 284 catchments in eastern New South Wales and adjacent regions, and the contours or isopleths of these coefficients, have been presented in Map 2 at the back of this report. This figure is useful for indicating the fit of the design isopleths to the derived values. For convenience of use in practical design, the isopleths have been redrawn with some adjustments as discussed previously on a 1:2 500 000 standard base map of the Division of National Mapping of the Department of National Development and Energy. This is presented as Map 4 at the back of this report, and should be used for selection of 10-year runoff coefficients in design. Based on local conditions, some small-scale variations of coefficients from the values given by the isopleths may be justified. For the region west of the line on Map 4 joining Ashford, Tamworth, Bathurst, Yass, Tumut and Jingellic and extending to the boundary of rainfall zone E and the linear interpolation zone on Figure 2.17 of ARR, design values of the 10-year coefficient depend on catchment size as well as location. The procedure for evaluating C(10) is detailed in Sections 8.2.4 and 10.4 below. The design values for this western region must be regarded as being of an approximate nature.

Frequency factors by which to multiply the 10-year coefficients to obtain runoff coefficients of other return periods were developed in Section 8.3. The adopted values were presented in Table 8.4. For convenience, this is reproduced below as Table 10.1

Table 10.1 Average Frequency Factors $FF_y = C(Y)/C(10)$ for Regions in New South Wales

Region (ARR Rainfall Zones)	FREQUENCY FACTORS FF_y FOR RETURN PERIODS OF:-						
	1	2	5	10	20	50	100 very approx. only
Central and southern coastal (A + B)	0.60	0.70	0.86	1.00	1.13	$1.75-0.37\log_{10}I(12,50)$	$2.28-0.70\log_{10}I(12,50)$
Northern coastal (C)	0.60	0.73	0.88	1.00	1.08	$1.67-0.37\log_{10}I(12,50)$	$2.20-0.70\log_{10}I(12,50)$
Eastern interior and Northern Tablelands (D + E)	0.49	0.62	0.82	1.00	1.18	$1.84-0.37\log_{10}I(12,50)$	$2.42-0.70\log_{10}I(12,50)$

10.3 LOCATIONS NEAR ZONE BOUNDARIES

As discussed in Section 8.5, discontinuities in flood estimates occur at the boundaries of the rainfall zones in Figure 2.17 of ARR, reproduced in this report as Figure 5.2. These discontinuities result from abrupt changes in rainfall intensities for durations of less than 12 hours and in the frequency factors FF_y in Table 10.1 above. The solution adopted for this problem is to assume a linear transition of flood magnitudes over a distance of 25 km each side of a zone boundary. The recommended procedure for a site within this transition strip is to calculate the flood magnitude assuming that the location is first in one zone (Z1) and then in the other zone (Z2). If the two estimates for a selected return period are Q_{Z1} and Q_{Z2}, and the site is actually in zone Z1, then the design flood Q may be calculated as:-

$$Q = \frac{(Q_{Z1} + Q_{Z2})}{2} + \frac{D_{Z1}}{50} (Q_{Z1} - Q_{Z2}) \qquad (10.1)$$

where D_{Z1} = distance of the catchment centroid from the zone boundary into zone Z1 (km).

An example of the use of this procedure is given in Section 10.5.3.

10.4 STEPS IN USE OF DESIGN PROCEDURE

As noted earlier, the steps in the design procedure are similar to those in currently-used applications of the Rational Method. The steps are set out below.

(i) Location of approximate catchment centroid is determined from a topographic map and the catchment area (A km^2) determined.

(ii) Critical duration of the design rainfall (t_c hours) is calculated from equation (4.2)

$$t_c = 0.76\ A^{0.38}$$

(iii) Extract parameters for estimation of design rainfall:

- zone and zone factor (AFACT) from Figure 5.2:
- 12 hour, 2 year rainfall intensity I(12,2) mm/h from Figure 5.3;
- 12 hour, 50 year rainfall intensity I(12,50) mm/h from Figure 5.4.

Alternatively, these values may be obtained from the original Figures 2.17, 2.18 and 2.19 and Table 2.3 of ARR.

(iv) Calculation of the design rainfall intensity for the critical duration and the selected return period. This is detailed as steps 3 to 5 in Section 5.4 and involves calculation of the 2 and 50 year rainfalls of the critical duration t hours by

I(t,2) = MF x I(12,2)
and I(t,50) = MF x I(12,50)

where the multiplying factor MF is read from Figure 5.5 (a) or 5.5 (b). The rainfall intensity for the design return period is then interpolated on Figure 5.6. Alternatively, the corresponding procedures in ARR may be used. As noted in Sections 5.3 and 5.4, no reduction of the point estimate of rainfall should be made to the size of the catchment.

(v) Determine the 10-year runoff coefficient C(10) for the site from Map 4 at the back of this report. For the region east of the line on Map 4 joining Ashford, Tamworth, Bathurst, Yass, Tumut and Jingellic, values of the 10-year coefficient are read directly from the map. West of this line, design values are read directly from the map for catchments larger than 100 km^2, with a minimum recommended value of 0.10. For catchment areas smaller than 100 km^2, the value from the map should be used if it is greater than 0.40. If it is less than 0.40, the map value should be multiplied by $(100/A)^{0.15}$ where A is in km^2, but with a maximum value of 0.40. However if this adjusted value is less than 0.20, a design value of the 10-year coefficient of 0.20 should be used. These design values for the western region must be regarded as approximate, and can be used for the region between the line specified above and the boundary of the rainfall zone E and linear interpolation zone on Figure 2.17 of ARR. This western boundary corresponds approximately to a line joining the towns of Mungindi, Nyngan, Condobolin Narrandera and Tocumwal.

(vi) For the design return period Y, determine the runoff coefficient C(Y) as $C(Y) = FF_y . C(10)$.

where the frequency factor FF_y is found from Table 10.1 (or Table 8.4) for the zone in which the catchment is located.

(vii) The design flood magnitude Q(Y) m^3/s is calculated by the Rational Method formula (equation 1.1)

$$Q(Y) = 0.278\ C(Y).\ \ I(t,Y).A$$

(viii) If the catchment centroid is within 25 km of a zone boundary, steps (iii) to (vii) are carried out assuming that the site is in one zone and then the other, and the design flood magnitude is then calculated by the linear interpolation equation (10.1).

10.5 EXAMPLES OF USE OF THE DESIGN METHOD

Three examples of the calculation of design flood magnitudes are given below. The first is for the normal case where the catchment is within a rainfall zone in the eastern part of eastern New South Wales, and the second is for the same catchment but for a 50-year return period involving evaluation of a frequency factor for the runoff coefficient that depends on I(12,50) as well as the rainfall zone. The third example illustrates the procedure where the site is within 25 km of a zone boundary. A fourth example is given of determination of 10-year runoff coefficients only for small catchments at a site in the western part of eastern New South Wales. The remainder of the procedure for calculating the design flood magnitudes in this case would be identical with those in the other examples.

10.5.1 Example 1 - normal design within a zone

It is required to estimate the 20-year design flood for a site on Wild Cattle Creek in the Clarence River Valley.

(i) Location and catchment area:
Topographic map - Brooklana, scale 1:31 680
Grid reference of site - 868 488
Approximate catchment centroid

- latitude 30° 17' (or 30.28°)
- longitude 152° 46' (or 152.77°)

Catchment area - 31.0 km^2

(ii) Critical duration of design rainfall:

$$t_c = 0.76\ A^{0.38}$$
$$= 0.76\ (31.0)^{0.38}$$
$$= 2.80 \text{ hours}$$

(iii) Parameters for estimation of design rainfall:
Figure 5.2 - zone C and AFACT = 2.95
Figure 5.3 - I(12,2) = 9.0 mm/h
Figure 5.4 - I(12,50) = 17.0 mm/h

(iv) Calculation of design rainfall intensity:
Figure 5.5(a), MF = 2.15 for t = 2.80 h
Then I(2.80,2) = 2.15 x 9.0 = 19.4 mm/h
and I(2.80,50) = 2.15 x 17.0 = 36.6 mm/h

Interpolating on Figure 5.6, I(2.80,20) = 32.0 mm/h
No reduction of this point estimate should be made for the size of the catchment.

(v) 10-year runoff coefficient C(10):
From Map 4 at back of report,

C(10) = 1.20

Note that the catchment is in a region of high runoff coefficients, and that the value is greater than one, as discussed in Section 8.1.1.

(vi) Runoff coefficient of design return period of Y = 20 years:
Table 10.1, for zone C and Y = 20 years,

$FF_y = 1.08$

$$\text{Then } C(10) = FF_y.C(10)$$
$$= 1.08 \times 1.20$$
$$= 1.30$$

(vii) Design flood magnitude:

$$Q(20) = 0.278.C(20).I(2.80,20).A$$
$$= 0.278 \times 1.30 \times 32.0 \times 31.0$$
$$= 359 m^3/s \text{ - say } 360\ m^3/s$$

10.5.2 Example 2 - as for example 1 in Section 10.5.1 above, but with a 50-year return period

Steps (i) to (v) are the same as above, except that the rainfall intensity is I(2.80,50) = 36.6 mm/h

(vi) Runoff coefficient of design return period of Y = 50 years.

$$\text{Table 10.1, for zone C, } FF_y = 1.67 - 0.37\log_{10}I(12,50)$$
$$= 1.67 - 0.37\log_{10}17.0$$
$$= 1.67 - 0.46$$
$$= 1.21$$

Then $C(50) = FF_Y . C(10)$
$= 1.21 \times 1.20$
$= 1.45$

(vii) Design flood magnitude:

$Q(50) = 0.278 . C(50) . I(2.80,50) . A$
$= 0.278 \times 1.45 \times 36.6 \times 31.0$
$= 458\ m^3/s$ - say $460\ m^3/s$

10.5.3 Example 3 - design for site within 25 km of zone boundary

It is required to estimate the 5-year design flood for a site on Omadale Creek in the Hunter River Valley.

(i) Location and catchment area:
Topographic map-Ellerston, Scale 1:31 680
Grid reference of site - 300 584
Approximate catchment centroid
- latitude 31° 52' (or 31.87°)
- longitude 151° 18' (or 151.30°)
Catchment area-104 km^2

(ii) Critical duration of design rainfall:

$t_c = 0.76A^{0.38}$
$= 0.76\ (104)^{0.38}$
$= 4.44$ hours.

(iii) Parameters for estimation of design rainfall:
Figure 5.2 - zone D, but very close (5 km) to boundary of zone B. As the catchment centroid is within 25 km of the zone boundary, calculations are carried forward for both zone D and zone B.
Figure 5.2 - AFACT = 4.70 (zone D)
= 3.78 (zone B)

Figure 5.3 - I(12,2) = 5.0 mm/h
Figure 5.4 - I(12,50) = 9.5 mm/h

(iv) Calculation of design rainfall intensity:

	Zone D	Zone B
Figure 5.5 (a), for t = 4.44 h,	MF = 2.01	MF = 1.81
I(4.44,2)	= 2.01 x 50 = 10.1 mm/h	= 1.81 x 5.0 = 9.1 mm/h
I(4.44,50)	= 2.01 x 9.5 = 19.1 mm/h	= 1.81 x 9.5 = 17.2 mm/h
Interpolating on Figure 5.6, I(4.44,5)	= 12.9 mm/h	= 11.6 mm/h

(v) 10-year runoff coefficient C(10):
From Map 4 at back of report,

C(10) = 0.22

(vi) Runoff coefficient of design return period Y = 5 years:
Table 10.1 for Y = 5 years,

		Zone D	Zone B
	FF_y =	0.82	0.86
	$C(5) = FF_y.C(10)$ =	0.82 x 0.22	0.86 x 0.22
		= 0.18	= 0.19

(vii) Design flood magnitude for each zone:

For zone D, $Q(5)_D = 0.278.C(5).I(4.44,S).A$
$= 0.278 \times 0.18 \times 12.9 \times 104$
$= 67.1\ m^3/s$

For Zone B, $Q(5)_B = 0.278 \times 0.19 \times 11.6 \times 104$
$= 63.7\ m^3/s$

(viii) Interpolation of design flood in boundary zone:
The distance D_{z1} of the catchment centroid from the boundary into Zone D is 5 km. Equation (10.1) then gives

$$Q(5) = \frac{(Q(5)_D + Q(5)_B)}{2} + \frac{D_{z1}}{50}(Q(5)_D - Q(5)_B)$$

$$= \frac{(67.1 + 63.7)}{2} + \frac{5}{50}(67.1 - 63.7)$$

$$= 65.4 + 0.1 \times 3.4$$
$$= 65.7\ m^3/s \quad - \text{ say } 66\ m^3/s$$

10.5.4 - <u>10-year runoff coefficients in the western part of eastern New South Wales</u>

It is required to estimate 10-year runoff coefficients for rural catchments of various sizes at Forbes in central New South Wales.

(i) Catchment location:
Latitude 33.4°
Longitude 148.0°

(ii) 10-year runoff coefficient from Map 4:
C(10) = 0.12

(iii) Determination of the design C(10) values appropriate to catchments of different sizes is illustrated in Table 10.2 below. The criteria and adjustments are detailed in step (v) of Section 10.4

Table 10.2 C(10) Values for Catchments of Different Sizes At Forbes

Catchment area (km^2)	Multiplying factor $(100/A)^{0.15}$	Calculated C(10) value	Adopted C(10) value
≥100	–	–	0.12
10	1.41	0.17	0.20
1.0	2.00	0.24	0.24
0.1	2.82	0.34	0.34
0.01	3.98	0.48	0.40

(iv) Calculation of discharges:

The remainder of the procedure for calculating the design flood magnitudes would be identical with those in the previous examples.

11. BANKFULL DISCHARGE ESTIMATES FOR WESTERN NEW SOUTH WALES

11.1 USE OF BANKFULL DISCHARGES

Of the 290 gauged catchments used in the project, 284 are situated in eastern New South Wales and in adjoining areas of Victoria and Queensland. Only three of the gauged catchments are located in the arid western two-thirds of New South Wales, and these had short lengths of record, two of seven years and the other of only four years. One of these catchments is near Cobar, and the other two are at Fowlers Gap north of Broken Hill. The remaining three catchments used in the project are near Alice Springs in the Northern Territory. Although these three catchments provide the best available arid zone data, having record lengths of eleven, twenty and twenty three years, they are distant from the border with New South Wales. The available gauged catchments thus give only an indication of flood potential but do not provide an adequate basis for the derivation of data on design floods in western New South Wales.

To obtain a better indication of flood potential over a wider area of western New South Wales, bankfull discharge estimates were made for seventy five catchments in the region. Most of the sites are on the Silver City Highway between Tibooburra and Broken Hill and on the Barrier Highway between Broken Hill and Cobar, with a concentration of thirty five sites on or near the Fowlers Gap Arid Zone Research Station operated by the University of New South Wales 110 km north of Broken Hill. Most of the sites are in undulating or hilly country. Seven sites are in the sub-humid region on the Mitchell Highway between Dubbo and Orange.

Bankfull discharge estimates can only give an approximate indication of the flood potential of a catchment. The principle underlying their application is that the channel cross-section is formed mainly by flood flows. Frequent small flows have low erosive power, and very large floods have high erosive power but occur so infrequently as to have relatively little effect on the stream cross-section. There is some 'dominant' flood discharge of intermediate frequency which has greatest effect on the channel section which is conveyed just within the stream banks. Studies in humid regions indicate that this bankfull flow has a return period of about one year (based on the partial series). While there is little firm evidence, it is generally considered that the corresponding return period is higher in arid regions. Investigations of several types of data to indicate an appropriate return for western New South Wales are described later in this chapter.

For bankfull estimates to be valid, the banks or flood plains must be in adjustment with the the present flow regime, and the stream must be in quasi-equilibrium. ARR suggests that for these reasons, the approach should be restricted to catchments larger than 50 km^2. This restriction was not adopted here, and over half the sites used had catchment areas smaller than this limit. Inspection of the stream cross-sections in the field and the consistency of results indicate that the use of the smaller catchments is justified, although any results based on bankfull estimates must be approximate.

The use of bankfull discharge estimates is described briefly in Section 9.6 of ARR. More detailed applications in New South Wales are described by Woodyer (1968), Woodyer and Fleming (1968) and Pickup and Warner (1970). A general review and discussion of problems that can occur with bankfull estimates is given by Williams (1978).

11.2 BANKFULL SITES

As noted above, seventy five sites were used for bankfull discharge estimates. Of these, sixty eight were located along or near the Silver City Highway between Tibooburra and Broken Hill and on the Barrier Highway between Broken Hill and Cobar with a concentration of thirty five sites at Fowlers Gap. From about half way between Cobar and Nyngan to about 50 km north-west of Dubbo, no definite channels draining small catchments are apparent. This flat country near the Barrier and Mitchell Highways is transversed by distributary channels and anabranches from larger streams. The remaining seven sites were on the Mitchell Highway between Dubbo and Orange. These seven sites in the sub-humid region were markedly different to the sixty eight in the arid zone further west in that their stream cross-sections tended to be more square or "V" shaped rather than the flat and wide rectangular shapes observed in western streams. Also, more than one bench was usually present, whereas only one normally occurred on the western streams.

The distribution of the locations of the seventy five sites is listed in Table 11.1, and the locations are shown on Figure 11.1.

Table 11.1 Locations of Bankfull Sites

Location of Sites	Number of sites
On and near Fowlers Gap Research Station	35
On the Silver City Highway between Fowlers Gap and Tibooburra	10
Near Broken Hill on the Silver City and Barrier Highways	12
On the Barrier Highway between Broken Hill and Cobar	11
On the Mitchell Highway between Dubbo and Orange	7

The catchment areas drained by the bankfull sites were measured from available topographic maps and covered a large range, with all but five being less than 250 km^2. Figure 11.2 shows the distribution of catchment areas. The median area is about 15 km^2.

For the seventy five bankfull sites, data for fifty five were obtained during a field trip in this project. Information for fourteen of the sites at Fowlers Gap was obtained during a previous field trip by D. H. Pilgrim, I. Cordery and B.H. Nickels of the University of New South Wales, and the data were extracted from field books and a report of this trip. For the remaining six sites, information was obtained from field measurements reported by Bell and Vorst (1980).

11.3 ASSESSMENT OF SITE CHARACTERISTICS AND BANKFULL DISCHARGE

As noted above, bankfull levels and site characteristics were assessed for each station by field inspections. Identification of the bankfull level is rather subjective and various procedures have been proposed, leading to somewhat different levels (Williams 1978). The method used in the site inspections in this project and in the two other source studies generally followed the procedure given in Section 9.6.2 of ARR and in more detail by Woodyer (1968). At many sites in the arid zone, only one or sometimes two benches were observed, rather than the three that often occur in more humid areas. The fact that the selected bench levels were in adjustment with the present stream regime and were not terraces was indicated at many sites by the presence of flood debris.

The bankfull discharge at each site was estimated by the Manning formula:

$$Q_b = \frac{A_x}{n} R^{2/3} S^{1/2} \qquad (11.1)$$

where Q_b = estimated bankfull discharge (m^3/s)

A_x = cross sectional area of the stream channel for the selected bankfull stage (m^2)

R = hydraulic radius A_x/P (m) where P is the wetted perimeter for the bankfull stage (m).

S = slope of the energy grade line (m/m) (note that the units of slope are different here to those used in the remainder of the report).

and n = Manning roughness coefficient.

Values of A_x, P and R were obtained by survey of a typical cross section at each site. The roughness coefficient n was estimated by Cowan's summation method described in Chow (1959). The slope of the energy grade line is probably best approximated by the slope of the bench or berm adopted as the bankfull level. This is not usually defined for a sufficient distance along the stream (about fifteen times the width of the section) to obtain an accurate estimate of slope. In most cases, slope had to be estimated from the bed of the stream. The choice of the longitudinal section for this measurement was often difficult as a result of the flat slopes normally encountered and the variability in the sediment deposition patterns in the beds of the streams. For many flat streams, the slope was considered to be better and more consistently defined from topographic maps than from the field survey of an isolated site. For steeper streams, the slope measured by survey at the site was adopted. To aid consistency of estimates from the various sources, the sites at Fowlers Gap used by D.H. Pilgrim, I.Cordery and B.H. Nickels were visited on the field trip in this project. Estimates at several sites were compared to obtain some calibration of procedures, and satisfactory agreement of estimates was obtained.

Details of the locations, characteristics and bankfull discharge estimates for each of the seventy five sites are given in the supplementary report for this project, 'Flood Data and Catchment Characteristics' (McDermott and Pilgrim 1980).

11.4 RELATIONSHIP BETWEEN BANKFULL DISCHARGE AND CATCHMENT CHARACTERISTICS

The approach used for developing design data by mapping runoff coefficients in eastern New South Wales was not possible in the west of the State. The sites utilised were distributed linearly along two highways as shown in Figure 11.1, and gave insufficient spatial coverage for mapping of isopleths.

Also, estimated runoff coefficients appeared to be related to catchment area in contrast to those in the east, and the coefficient values for small catchments were always much larger than those for larger catchments near the same location. This probably results from two factors. Large transmission losses occur from floods as they move downstream in arid catchments (Renard and Keppel 1966). Some magnitudes of transmission losses are discussed by Baron et al. (1980). The second factor is that floods on the larger arid catchments probably result from runoff from only part of the catchment areas.

As runoff coefficients could not be mapped, a different approach was used for the derivation of approximate design data for western New South Wales. This involved regression relations between bankfull discharges for the sites and catchment characteristics. As the sites are in undulating and hilly country, it is probable that the derived design relations should only be applied in regions with these characteristics.

Initially, the bankfull discharge estimates for the thirty five sites at Fowlers Gap were related to the single variable (AS_e^m), where A is the catchment area (km²), S_e is the equal area slope (m/km) of the main stream from source to site as defined in Section 3.2.1, and m is an exponent. The values of S_e obtained from maps were adjusted to a standard map scale of 1:100000 This is different to the standard scale of 1:25 000 adopted in Chapter 3 and other parts of this report, but was chosen as being most suitable for practical use as it is the most detailed map scale generally available in western New South Wales. Measured stream lengths and slopes were adjusted to values corresponding to the adopted standard scale of 1:100 000 by the multiplying factors in Table 11.2. These are based on the factors in Table 3.1, but a single factor has been used for all slopes. For all of the slopes in Table 3.1, the multiplying factors were identical to two decimal places for scales of 1:250 000 and 1:50 000. Variations occurred for a scale of 1: 25 000, but maps of this scale are not available in western New South Wales at present, and few if any will be available in the future. Thus a single average factor is also adequate for this scale. In application of the derived design formula, adjustment of slopes by the factors in Table 11.2 is necessary if they are determined from maps with scales other than 1:100 000.

Table 11.2 Multiplying Factors for Calculating Stream Length and Equal Area Slope Corresponding to Standard Map Scale of 1:100 000

Map scale used to measure L	Multiplying Factors to obtain values corresponding to scale of 1:100 000	
	Stream Length L	Equal area slope S_e
1:250 000	1.06	0.94
1:100 000	1.00	1.00
1:50 000	0.96	1.05
1:25 000	0.88	1.13

The values of A and S_e were combined into a single variable as they are highly correlated as discussed in Section 3.7, and their use as independent variables would invalidate regression as discussed in that section. The best value of the exponent was found by trial and error by calculating simple logarithmic regressions of Q_b on $(AS_e{}^m)$ for various values of m, and adopting the relation giving the highest correlation coefficient.

This occurred with m approximately equal to 1.0, and the adopted relation for Fowlers Gap was

$$Q_b = 0.40\ (AS_e)^{0.77} \tag{11.2}$$

The correlation coefficient was 0.89, and the standard error of the logarithms corresponded to a range of x 1.8 to ÷ 1.8.

Insufficient values were available for the derivation of similar relations for the other regions in Table 11.1. The values from each of the regions were then combined separately with those for Fowlers Gap and regressions calculated for each of the combined sets of data. In each case, the value of the exponent m was set at 1.0. The resulting regression relations are summarised in Table 11.3.

Table 11.3 Bankfull Discharge Regression Relations for Combined Regions

Data set number	Regions	Number of sites used	Relation obtained:- $Q_b = k\ (AS)^b$		
			k	b	Correl. coeffic. of logs.
1	Fowlers Gap (only)	35	0.40	0.77	0.89
2	Fowlers Gap and to Tibooburra	45	0.42	0.74	0.87
3	Fowlers Gap and to Broken Hill	47	0.50	0.69	0.89
4	Fowlers Gap and Broken Hill to Cobar	46	0.39	0.74	0.84
5	Fowlers Gap and Cobar to Orange	42	0.73	0.54	0.70

The differences between the various relations are illustrated more clearly in Figure 11.3. As expected from general stream characteristics, the only relation that was significantly different from that for Fowlers Gap data alone was for data set 5 involving the addition of the sites near Dubbo and Orange. Geographically, these sites are in the eastern part of New South Wales with a sub-humid climate. As discussed in Section 11.2, the shapes of the stream channels of these sites were markedly different to those of the channels in the arid zone. These seven sites were thus excluded from further analysis for derivation of relations for western New South Wales catchments.

As the relations for the data sets 1 to 4 were very similar, values from all of the remaining sixty eight sites in western New South Wales were pooled, and a regression was derived for the combined data. Once again, the exponent m was set at 1.0

The resulting relation was

$$Q_b = 0.44 \, (AS_e)^{0.69} \tag{11.3}$$

The correlation coefficient of the logarithms was 0.85, which is quite high when the inaccuracies inherent in bankfull discharge estimation and the wide geographical spread of the sites are considered.

A multiple regression relation was also derived using the logarithmic values from the sixty eight sites. A representative rainfall intensity was used as the additional variable. As it is readily available from Figure 2.18 of ARR, the 12-hour duration, 2-year return period intensity I(12, 2) (mm/h) was adopted. The derived relation was

$$Q_b = 42 \, (AS_e)^{0.68} / \, [I(12, 2)]^{3.7} \tag{11.4}$$

The coefficient of multiple correlation of the logarithms was 0.86, so that the relation explains only a further 2% of the variance than that accounted for by the simpler equation (11.3). The small magnitude of this improvement may result from the fact that there is only a small variation from 3.2 to 4.0 mm/h in the value of I(12, 2) over the sixty eight sites. This small variation also probably explains the large value of 3.7 for the exponent of I(12, 2) in equation (11.4). The choice of a design relation between equations (11.3) and (11.4) is not clear. The latter accounts for slightly more of the variance in the data and should therefore be slightly more accurate. However, this small improvement is probably meaningless in view of the approximate nature of the bankfull discharge estimates. Also, it is not clear that bankfull discharge should depend on the 12-hour, 2-year rainfall intensity, and it is certainly not logical that the value of discharge should decrease as the rainfall intensity increases, as implied by equation (11.4). It is possible that as I(12, 2) increases from west to east, the presence of this term in the equation reflects the effects of other variables related to geographical location rather than the effects of the rainfall itself. The term indicates that bankfull discharges tend to decrease from west to east, and this could reflect the effects of different climatological regimes, different soils and vegetation, or decreasing return periods of bankfull flow. The equation could thus operate in a logical fashion with I(12, 2) as a surrogate of other factors, even if the inverse relation of bankfull discharge and rainfall intensity is not logical.

In view of the smallness of the increase in explained variance, the lack of clarity of its logical justification, and the additional difficulty in its use, it seems that the use of equation (11.4) is not really justified. It is therefore recommended that equation (11.3) be used to provide approximate design data for bankfull discharge for undulating and hilly regions in western New South Wales.

11.5 FREQUENCY OF BANKFULL DISCHARGE IN WESTERN NEW SOUTH WALES

As noted in Section 11.1, it is thought that the return periods of bankfull discharges are larger in arid than in humid areas, but very little evidence is available for selection of appropriate values of return period. Four types of evidence have been investigated in this project. None is conclusive, but together they give a useful indication of the appropriate value.

(a) Three gauged catchments in western New South Wales.

Only three small catchments with streamflow records were available in western New South Wales. The lengths of records were only four, seven and seven years, so that large sampling errors are likely in flood frequency estimates from their records and the estimates can only be regarded as very approximate indications. The sampling errors are likely to be worse in the arid zone with its high variability than in humid regions. Despite this, the return periods of the bankfull discharges at the three gauging stations were estimated from the frequency curves of the partial series of floods, and these are listed in Table 11.4. The average return period for the three stations is 3.0 years.

Table 11.4 Return Periods of Bankfull Discharges at Three Gauging Stations in Western New South Wales

Gauging station number	Location	Ref. number of bankfull site	Catchment area (km^2)	Bankfull discharge Q_b (m^3/s)	Return Period of Q_b (years)
011001	Fowlers Gap	75	20	41.6	4.7
011315	Fowlers Gap	64 and 65	4.0	5.0	2.6
425016	Cobar	48	15	3.1	1.6

(b) Three gauged catchments near Alice Springs.

Although bankfull estimates could not be carried out for the three gauged catchments near Alice Springs for which data were obtained in this project, equation (11.3) was used to estimate discharges for these stations. These were then compared with the frequency curves derived from the partial series of observed flows at the stations, and return periods corresponding to the bankfull discharges were determined. These are listed in Table 11.5. The average return period for the three stations is 1.9 years.

Table 11.5 Return Periods of Estimated Bankfull Discharges at Three Gauging Stations Near Alice Springs

Gauging station number	Catchment area (km^2)	Estimated Bankfull discharge Q_b (m^3/s)	Return Period of Q_b (years)
006003	3.8	9.6	2.7
006009	450	100	1.0
006047	42	27	2.0

(c) Daily inflows to reservoirs at Broken Hill

Two water supply reservoirs are operated by the Broken Hill Water Board near the city. These are Umberumberka Creek Reservoir and Stephens Creek Reservoir. Records of daily inflow volumes to each reservoir have been kept since 1939 and were available for the thirty seven years to 1975. Bankfull discharge estimates were made at sites upstream of Umberumberka Creek Reservoir and downstream of Stephens Creek Reservoir, and transferred to the sites by assuming that peak flow is proportional to catchment area to the power of 0.7 (Alexander 1969). The 24 hour volumes of inflow corresponding to these bankfull discharges were estimated by assuming the hydrograph shape shown in figure 11.4 and developed in the UK 'Flood Studies Report' (Natural Environment Research Council 1975). The time to peak (T_p) was calculated by the time of concentration equation (4.2) developed in this study.

Volumes of the bankfull floods occurring within a 24 hour period were then compared with the observed volumes of daily inflows. The bankfull volume for Umberumberka Creek was found to have been exceeded fifteen times over the 37 years of record, and at Stephens Creek thirty four times. The corresponding return periods of bankfull flow are 2.5 and 1.1 years respectively, with an average of 1.8 years.

(d) Daily rainfalls at Corona.

A record of daily rainfall is available for the 90 year period 1885 to 1974 at Corona near Fowlers Gap, north of Broken Hill. Pilgrim, Cordery and Doran (1979) in a study of runoff characteristics at Fowlers Gap showed that a daily rainfall of 20 mm or greater is likely to produce runoff. Rainfalls of this magnitude occurred 184 times in the 90 year record at Corona, indicating that the average frequency of runoff events is about twice per year. For the gauging station on Homestead Creek at Fowlers Gap (station number 011001), the average of the peak flows of the four highest floods in the seven year record is approximately equal to the estimated bankfull discharge. The average of the daily rainfalls for these four events was 60 mm. Daily falls equal to or exceeding this have occurred forty times over the 90 year record at Corona. This indicates that the return period of bankfull flow at Homestead Creek is approximately 2.3 years.

A summary of the return periods of bankfull flow indicated by each of the four types of investigation is given in Table 11.6. The average of the return periods for each of the four types of investigations is 2.3 years. An attempt was made to use a fifth type of investigation involving the Cordery and Webb (1974) method of design flood estimation based on synthetic unit hydrographs. This method applies to eastern New South Wales and the loss function was modified in an attempt to apply the method to western regions of the State. However, the average return period of bankfull flows indicated by the method at eleven sites was 7 years. As this is much larger than any of the other indicated return periods, this approach was rejected as unreliable.

A return period of 2.5 years was thus adopted for bankfull discharge in arid western New South Wales. As expected, this is considerably higher than the value of one year (partial series) generally considered appropriate for humid regions, although values similar to those in western New South Wales have been reported for the Cumberland Plain south-west of Sydney (Pickup and Warner 1976).

Table 11.6 Summary of Indicated Return Periods of Bankfull Discharge in Western New South Wales

Subject of investigation	Site investigated	Indicated return period - years: At site	Indicated return period - years: Average
(a) Gauging stations in western N.S.W.	Homestead Ck. (011001), Fowlers Gap	4.7	
	Nelia Dam (011315), Fowlers Gap	2.6	3.0
	Box Ck. (425016), Cobar	1.6	
(b) Gauging stations near Alice Springs	Gillen Ck. (006003)	2.7	
	Todd R. (006009)	1.0	1.9
	Charles Ck. (006047)	2.0	
(c) Reservoirs near Broken Hill	Umberumberka Ck.	2.5	
			1.8
	Stephens Ck.	1.1	
(d) Daily rainfalls	Corona and Fowlers Gap	2.3	2.3
Overall average			2.3

11.6 FLOOD FREQUENCY CURVE FOR USE WITH BANKFULL DISCHARGE IN WESTERN NEW SOUTH WALES

As for eastern New South Wales, a log Pearson Type III frequency distribution was adopted for the west of the State as an arbitrary but consistent means of estimating floods of various frequencies. The use of this distribution requires values of the standard deviation SDEV and skew coefficient SKEW of the logarithms of the partial series floods. For the runoff coefficients in the east of the state, average frequency factors and hence values of SDEV and SKEW were adopted in Section 8.2 for each of several regions. This approach was used rather than relating values for individual catchments to catchment characteristics, even though both approaches were equally justified. The same approach of using average values for the whole region was adopted for western New South Wales, as flood data were available from insufficient catchments to relate individual values to catchment characteristics.

The statistics of the N highest floods are listed in Table 11.7 for each of the six gauged catchments in the arid zone for which flood data were available. N is equal to the number of years of record at each station.

With short records, skew coefficients are very unreliable as a result of sampling errors. As the skew coefficient for Nelia Dam was very different from the other values, and the station had only four years of record, this station was excluded from further consideration. For the remaining five stations, the average values of the statistics of the logarithms were $\overline{\text{SDEV}}$ = 0.320 and $\overline{\text{SKEW}}$ = 0.246. These values can still only be considered as very approximate, especially the skew coefficient, as a result of the short lengths of record of the stations.

Table 11.7 Statistics of Logarithms of Partial Series Floods of Arid Zone Stations

Gauging Station	Gauging Station Number	Record lengths N (years)	Catchment area A (km^2)	Statistics of logarithms of highest N floods	
				SDEV	SKEW
Homestead Ck. Fowlers Gap	011001	7	20	0.365	0.183
Nelia Dam, Fowlers Gap	011315	4	4	0.318	-1.386
Box Ck. at Cobar	425016	7	15	0.329	-0.025
Gillen Ck.	006003	11	3.8	0.247	0.469
Todd R.	006009	23	450	0.252	0.038
Charles Ck.	006047	20	42	0.406	0.565

To increase the sample size, data were also considered from seventeen stations in eastern New South Wales closest to the arid zone and situated in undulating to hilly country of similar slopes to those of the catchments in the arid zone. The average values of the statistics of the logarithms of the partial series floods of these stations were $\overline{SDEV}$ = 0.301 and $\overline{SKEW}$ = 0.894. These values are not greatly different from those above for the arid zone stations. The skew coefficients are somewhat different, but the difference is not large in comparison with likely sampling errors.

Accordingly, overall average values of $\overline{SDEV}$ = 0.305 and $\overline{SKEW}$ = 0.747 were adopted for estimating average frequency factors for the arid zone. Using these statistics and the log Pearson Type III distribution, Table 11.8 lists frequency factors FF_{by} by which to multiply the bankfull discharge of return period 2.5 years to obtain discharges of other return periods. The maximum return period listed is 20 years, as extrapolation beyond this would involve large probable errors and would not be justified. Use of any of the other combinations of $\overline{SDEV}$ and $\overline{SKEW}$ noted above would change the frequency factors by less than one per cent for return periods up to 5 years, and by up to eight per cent at 20 years.

Table 11.8 Frequency Factors for Bankfull Discharge to Obtain Floods of Other Return Periods - Western New South Wales

Return Period Y (years)	Frequency Factor FF_{by}
1	0.42
2	0.84
2.5	1.00
5	1.58
10	2.33
20	3.29

11.7 SUMMARY OF APPROXIMATE METHOD FOR ESTIMATION OF DESIGN FLOODS FOR WESTERN NEW SOUTH WALES

To summarise the results of the investigation of bankfull discharges described in this chapter, the derived method for estimation of design floods entails three steps.

These are:-

(a) Estimate the bankfull discharge Q_b for the catchment by means of equation 11.3, viz

$$Q_b = 0.44\ (AS_e)^{0.69}$$

The return period of this discharge is 2.5 years.

(b) The frequency factor FF_{by} appropriate to the design return period is determined from Table 11.8, up to a maximum of 20 years.

(c) The design flood discharge is calculated as;

$$Q(Y) = FF_{by} \times Q_b \qquad (11.5)$$

where Q(Y) = flood magnitude (m^3/s) of the design return period Y years.

FF_{by} = frequency factor from Table 11.8

Q_b = bankfull discharge (m^3/s).

The procedure is applicable to catchments up to 250 km^2 in undulating and hilly areas in arid western New South Wales. It is probably not applicable to very flat areas with characteristics different to those of the catchments from which the method was derived. The procedure can only be expected to give approximate estimates. It is based on estimates of bankfull discharges and of their return periods, and the basic data therefore contain considerable uncertainties. However, no alternative design data are available, and the procedure provides at least a guide to design floods in the absence of any other data.

11.8 EXAMPLE OF USE OF APPROXIMATE DESIGN METHOD

(a) Location of stream cross sect on of interest
Box Creek at Cobar
Topographic Map:- Cobar
Scale:- 1:100 000
Grid reference of section:- 855185

(b) Catchment characteristics
Approximate catchment centroid:-
Latitude: 31° 28' (or 31.47°)
Longitude: 145° 49 (or 145.82°)
Catchment area = 15.0 km^2 (A)

Main stream slope (equal area) S_e = 3.2 m/km, calculated from the 1:100 000 scale map. No adjustment by the factors in Table 11.2 is necessary as the map used is of the standard scale.

(c) Estimation of bankfull discharge
Ideally the site should be visited to carry out the bankfull estimate by site survey, as in Section 9.6 of ARR. In the normal case without

a site survey, the bankfull discharge is estimated by equation (11.3):-

$$Q_b = 0.44\ (AS_e)^{0.69}$$
$$= 0.44\ (15.0 \times 3.2)^{0.69}$$
$$= 6.4\ m^3/s$$

(d) Estimation of flood magnitudes for other return periods. As the catchment is in undulating to hilly country, the frequency factors from Table 11.8 can be used to calculate these flood magnitudes from the 2.5 year bankfull flow, as listed in Table 11.9

Table 11.9 Estimated Flood Discharge of Various Return Periods for Example of Flood Design Method

Return period Y (years)	Estimated flood magnitude $Q(Y)$ (m^3/s)
1	2.7
2	5.4
2.5	6.4
5	10
10	15
20	21

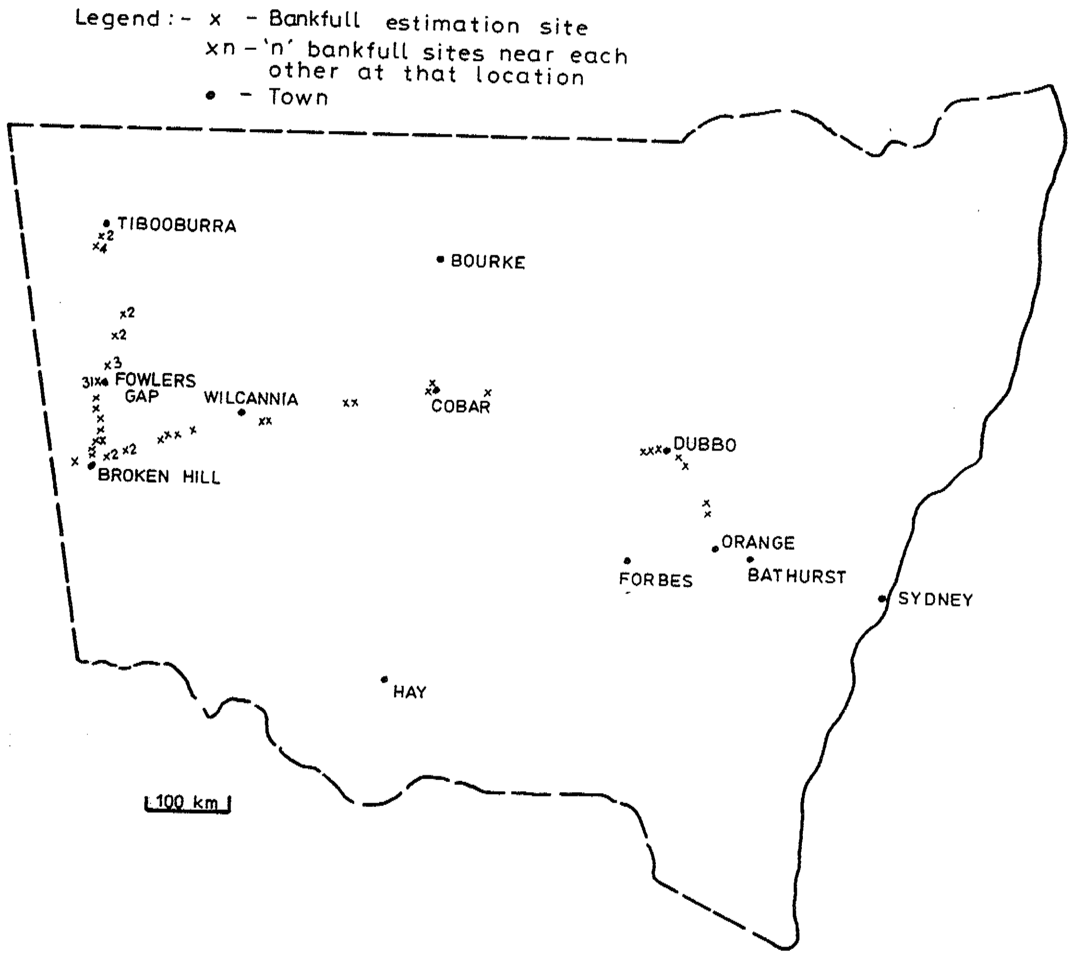


Figure 11.1 Locations of 75 bankfull sites in western New South Wales.

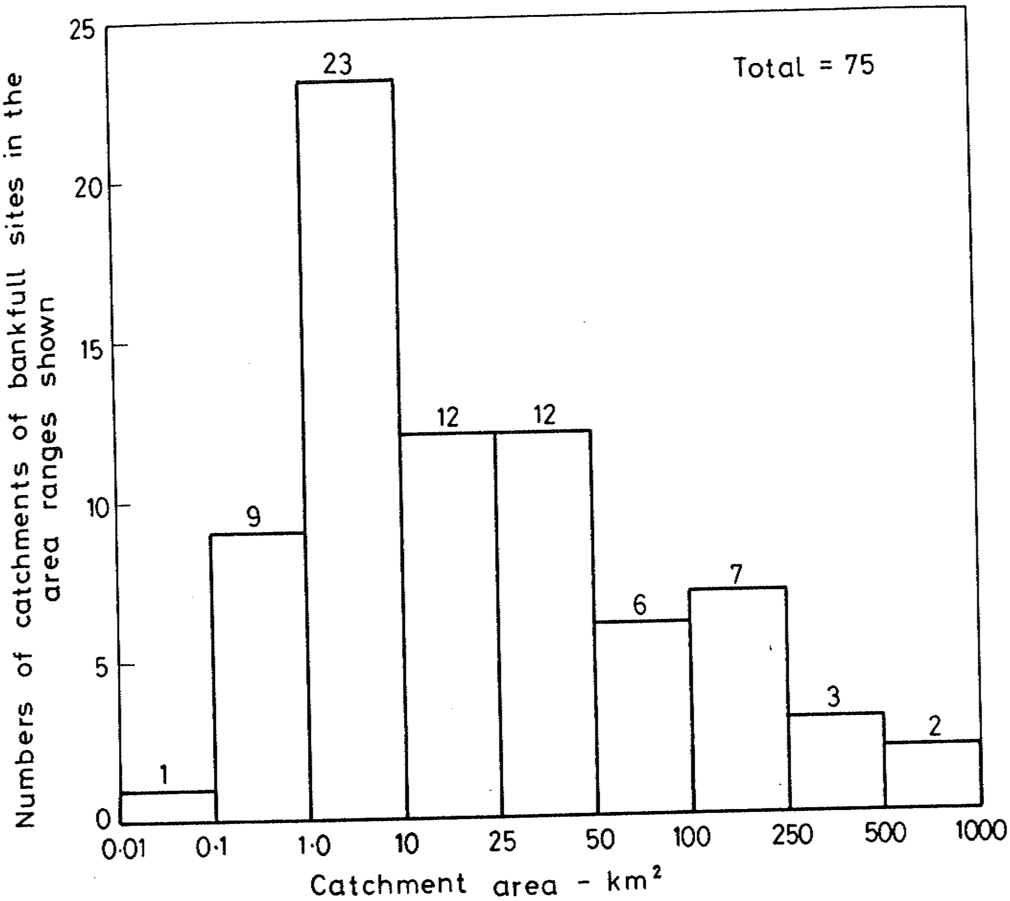


Figure 11.2 Frequency distribution of catchment areas of 75 bankfull discharge sites in western New South Wales.

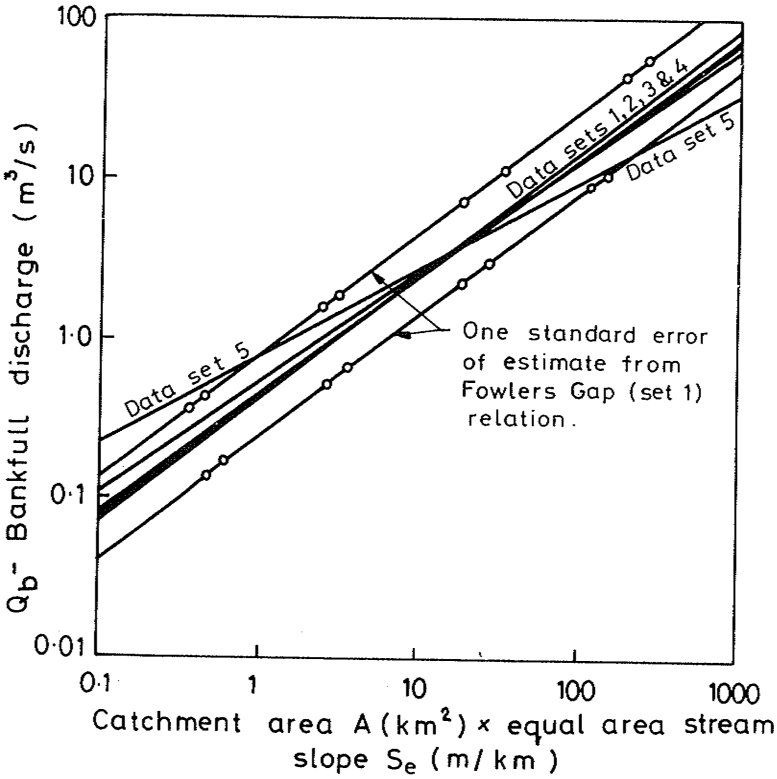


Figure 11.3 Bankfull discharge regressions for combined regions.
The combined regions for each data set are defined in Table 11.1.

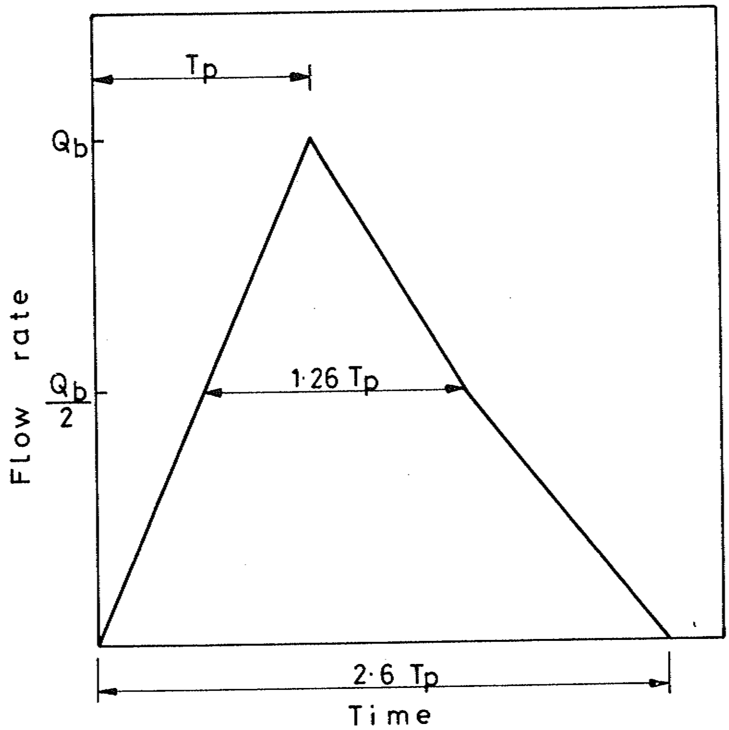


Figure 11.4 Assumed hydrograph shape for floods with bankfull peak discharges on Umberumberka and Stephens Creeks. From U.K. Flood Studies Report (National Environment Research Council 1975).

12. CONCLUSIONS

12.1 GENERAL CONCLUSIONS

A procedure has been developed in this report for estimation of design floods for small ungauged rural catchments in eastern New South Wales. The methodology used in developing the procedure would be equally applicable to other regions where observed flood data are available for a considerable number of small catchments. For eastern New South Wales, all available flood data were used in the development of the design procedure, involving streamflow records from 284 catchments. Most were smaller than 250 km^2, although some larger catchments were used where no other data were available.

The procedure was developed to be as simple as possible and to suit the needs of the ordinary designer. It is based on the statistical interpretation of the Rational Method, and its application follows the familiar procedures of the Rational Method to encourage its adoption in practice. However, the method is conceptually a type of regional flood frequency procedure with rainfall intensity as one of the independent predictor variables. As rainfall intensity is one of the major determinants of the flood characteristics of a catchment, the statistical interpretation of the Rational Method is an attractive and efficient form of regional flood frequency analysis.

The design procedure is simple to apply in practice, and the values of runoff coefficient are presented as a map of 10-year coefficients, together with tabulated frequency factors for various return periods in three geographical zones. The design rainfalls used in the method are the generalised data in ARR, presented in map form and based mainly on daily rainfalls, as these are available for all of the region of interest. Simplified design graphs are included in the report. Rainfall durations are based on a new formula for 'time of concentration' or characteristic response time, derived from observed minimum times of hydrograph rise for 96 catchments in eastern New South Wales. This formula involves catchment area A(km^2)only, and is

$$t_c\ (h) = 0.76\ A^{0.38}$$

where A is in km^2 and t_c is in hours.

The procedures used in deriving the design runoff coefficients exactly mirror the steps used in design. It is important that for valid application of the method, the same design data should be used as were utilised in developing the method. This will ensure that the estimated floods reproduce as closely as possible the magnitudes derived from frequency analysis of all the observed floods. If new rainfall data become available, for example, it would really be necessary to rederive the runoff coefficients using these rainfalls.

As the observed flood data and the flood values obtained by frequency analysis are of fundamental importance to the design procedure, considerable effort was expended in obtaining flood data and flood frequency curves of the highest possible accuracy. Almost half of the available time in the project was spent on this aspect. In terms of practical results, refinement of analysis in deriving the design procedure could not compensate for deficiencies in the data.

The recommended maximum catchment size for application of the design procedure is 250 km^2. This is larger than the traditional limit suggested for the deterministic interpretation of the Rational Method, but the problems in that approach with larger catchments do not apply to nearly the same extent with the statistical interpretation. The results of this study show no trend or bias of runoff coefficient values with increasing catchment size, even to areas greater than 250 km^2, although only a limited number of large catchments was analysed.

Use of the design method presented in this report should lead to important practical benefits. A very large expenditure is involved in works sized by design flood estimates on small rural catchments. For the whole of Australia, the average annual expenditure has been estimated as $180 million as at mid 1979, while an equivalent figure for eastern New South Wales would be approximately $40 million. Currently used methods of design are based mainly on judgment and experience rather than on observed data as in the method developed here, and this is less than satisfactory considering the very large expenditures involved. Previous studies have shown that estimates by these methods are often inconsistent and deviate widely from observed values. Flood magnitudes estimated from the procedure developed in this report and from ARR were compared with the values derived from frequency analysis of observed floods. Although this comparison involved some lack of independence, the design data developed herein were shown to be of much greater accuracy than those in ARR and other current design procedures. Analysis of sampling errors indicated that the derived data reproduced the observed flood data with as much accuracy as is possible in view of the scatter inherent in the available observed data.

Two economic analyses were carried out on the likely costs of the inaccuracies involved in design flood estimates. Several assumptions were involved so that the analyses were necessarily of an approximate nature. Both analyses indicated that the inaccuracy of the ARR design method added about 50 per cent to the optimum costs that would be incurred if error-free design estimates could be made. The corresponding figure for the method developed in this report is only 10 per cent. Of more practical importance, use of the design procedure developed in this project in place of the ARR and similar methods should result in savings of about 30 per centon present costs, or $12 million annually in eastern New South Wales. The full potential savings would only be realised if the true optimum return period of the design flood was known in each case. This is not known in practice, and optimum return periods had to be assumed in part of the analysis. Although the analysis indicates that these assumptions would only affect the magnitude of potential savings to a small extent, further research on the selection of optimum return periods for the design of individual structures would be very desirable.

The design procedure discussed above could not be extended to arid and semi-arid western New South Wales. Only short records were available from a few catchments, and channel transmission losses and partial area runoff would have caused difficulties. However, an approximate procedure for determination of design floods on small catchments in undulating to hilly terrain was developed from bankfull flows estimated at 68 sites.

Detailed information on the 290 gauged catchments used in the study, including the observed floods, flood frequency analyses and estimates, and physical and rainfall data, are given in a supplementary report (McDermott and Pilgrim 1980). Information on the 75 bankfull sites in western New South Wales investigated during the project is also detailed in that report.

12.2 SPECIFIC CONCLUSIONS FROM THE STUDY

12.2.1 Flood flow frequency

(a) The partial series of flood events was considered to be more appropriate for use in the study than the annual series, as it gives better estimates at low return periods. Also, damage and inconvenience caused by surcharging of structures are not limited to annual events, and design rainfalls are based on partial series. In practice, the partial series is difficult to extract from the full record, especially for small catchments with rapid response and frequenct multiple-peaked flood events. Monthly maximum floods extracted from computer printouts were found to provide a good practical approximation to the partial series.

(b) It is both worthwhile and desirable to spend considerable time checking and supplementing the recorded data. The derived results cannot be better than the data base used. Painstaking checking, investigation and analysis enabled considerable improvement to be made in the data base used in this project.

(c) The amount of missing data in the observed flood records was found to be considerable, warranting the expenditure of major effort to fill in or otherwise deal with these periods. A detailed investigation provided guidelines for handling periods of missing data, and showed that consistently the worst of seven procedures investigated was that recommended in ARR of eliminating the entire year in which a large missing event is suspected to have occurred. Where a nearby station record exists and a good relation between flood peaks on the two catchments can be obtained, the best policy is to use this relation to estimate missing peaks. Where no relation of this type is available, the best policy is generally to simply ignore the missing data and include the missing period in the overall period of record. However, for record lengths greater than 20 years, it is better to subtract an amount from each year with missing data proportional to the ratio of the number of peaks missed to the total number of ranked peaks in that year.

(d) Although it is normally used for annual floods, the log Pearson Type III (LP3) probability distribution provided a useful and simple means of objectively fitting a frequency curve to the observed data. For the partial series, the fitted curve is quite empirical.

(e) In using the LP3 probability distribution to fit the partial series of highest floods for a gauged catchment, it was necessary to choose the number of floods to be included in the fitting calculations. The results of an investigation showed that the most appropriate number of floods to include was N, the number of whole years in the record.

(f) As all flood records involve at least some uncertainties and errors, no observed records from gauged catchments were rejected outright, even when record lengths were short and the station rating curve involved considerable extrapolation. Instead, a reliability index was devised and values assigned for all gauged catchments. The reliability values were used later in the project in assessing the weight given to runoff coefficients derived from each catchment in developing the design relationships for the region.

(g) The slope of the flood frequency curve was found to exhibit strong regional dependence. The slopes tended to be flat in the higher rainfall North Coast area and steep in the drier western regions. However, skew coefficients of the logarithms of the flows in the partial series varied randomly about a mean value of 0.72.

12.2.2 Catchment characteristics

(a) Many measures have been used for slope of the main stream. The equal area slope was chosen for use in this project, defined as the slope of a line drawn through the outlet and intersecting the longitudinal profile of the main stream such that the areas enclosed above and below the profile are equal. This was considered to be more likely to be related to hydrograph response than the simple average slope from source to outlet. It is also reasonably simple to derive and is thus suitable for use in a practical design method.

(b) Values of physical characteristics of catchments such as stream length and slope depend on the scale of the map from which they are measured. Use of different map scales is thus unsatisfactory for the derivation of consistent design values and relations, and for the objective application of the relations in design practice. To overcome this problem, a standard map scale of 1: 25 000 was selected as this was the largest scale of maps covering much of the area, and maps for much of the remainder of eastern New South Wales are being prepared. All lengths and slopes used in the project have been standardised to this scale. Relationships between lengths and slopes for different map scales were developed from measurements on different scale maps for 268 catchments. These relationships are thus available to convert values appropriate to one map scale to those appropriate to another scale.

(c) Relationships between stream lengths, slope and catchment area were derived from data for 429 catchments in eastern New South Wales. A strong and consistent relation over the entire region was derived between stream length and area, and the relation was very similar to those derived in other parts of the world. Relationships between slope and stream length and area were different for different regions, but were consistent within regions. As expected, predicted slopes decrease from the mountainous regions to the plains. This dependence of slope on region gives implicit support to the concept used in the study that catchment response and runoff coefficients can be related to the location of the catchment.

12.2.3 Design duration of rainfall

(a) In the statistical intepretation of the Rational Method, it is not necessary for the rainfall duration to equal a physical time of concentration of travel of water from the most remote point to the outlet of the catchment. The duration could equal any characteristic response time of the catchment that gave consistent values of runoff coefficients derived from observed data, that could be simply predicted from a design formula involving catchment characteristics, and that gave runoff coefficient values that were meaningful and acceptable to designers.

(b) Investigation of existing formulae for time of concentration or other response times indicated that they were unsatisfactory because they led to widely varying and inconsistent derived values of runoff coefficients, and these values were sometimes unacceptably high to designers used to coefficients less than unity. The high values resulted partly from the adoption of equal area slope as the measure of stream slope.

(c) Data were obtained from 96 catchments in eastern New South Wales for the derivation of a new formula for characteristic response time of flood hydrographs. Some of these data were typical minimum times of rise of observed hydrographs and the remainder were values of the parameter C in the

Clark unit hydrograph model. These measures should be similar to one another and approximate the concept of time of concentration.

(d) From many forms of relations investigated, three formulae were derived which were simple and gave almost equally good fits to the data from the 96 catchments. Only area, length and slope were involved in these formulae, and other variables such as other aspects of topography, median annual rainfall, and vegetation cover were found to have a negligible effect. The three formulae were of similar form to many of the published relations for various types of hydrograph response times. Although the three formulae accounted for most of the variation in the data (over 80% of the variance in the logarithms), some scatter was still evident, and further investigation of rainfall duration would provide a useful field of study.

(e) The adoption of the design formula was based on the two criteria of goodness of fit of the observed values of time of concentration or characteristic response time, and consistency of runoff coefficients derived using each formula. On these criteria, the formula selected was

$$t = 0.76\ A^{0.38}$$

where t is in hours and A in km². As the formula involves only area, it should only be used in eastern New South Wales where it was derived. In this region, however, it is essential that this formula be used in application of the design procedure to ensure the best possible reproduction of the observed flood data. The formula gives the complete rainfall duration for design, and no allowance for overland flow or other factors should be added.

12.2.4 Rainfall frequency data.

(a) ARR was used as the source of rainfall data. Of the several forms of data in that publication, the generalised data based mainly on daily rainfalls were selected for use in this project. These are presented basically in the form of rainfall maps. The generalised data were selected as they cover the whole of eastern New South Wales whereas the pluviograph data only cover about half this region. They are also simple to use, they are derived from a much larger data base in space and time, and they give comparable intensities at short durations with those from the pluviograph stations.

(b) Simplified procedures have been included in the report for application of the generalised rainfall data.

(c) The generalised rainfall data in ARR must be used with the design data in this report for best reproduction of the observed flood data, as these rainfalls were used in derivation of the design runoff coefficients. The use of any improved or more detailed rainfall data would lead to less accurate flood estimates. For valid use of other rainfall data, rederivation of the runoff coefficients would be required.

12.2.5 Selection of the design method

(a) Seven design procedures were investigated and compared. Four were similar in form to the conventional Rational Method, one used a fixed rainfall duration, and the other two involved regional frequency procedures.

(b) All runoff coefficients or corresponding design parameters were derived in exactly the reverse procedure of those which would be used in design. Use of the derived parameters should thus give the best possible reproduction of the observed flood data.

(c) Derived coefficients with a return period of 10 years were chosen as the basic data in each procedure and for comparison of the different procedures. This was the highest return period for which all of the catchments in the data base could be used, and is high enough to be relevant to practical design. Coefficients for other return periods were related to 10-year values by ratios or frequency factors.

(d) Inspection of the 10-year coefficients derived for each of the several procedures showed that values were strongly related to location. Mapping of the 10-year coefficients was thus adopted as the basic form of presentation of the design data.

(e) Selection of the adopted design procedure was based on the two criteria of goodness of fit of mapped contour lines to individual derived values of runoff coefficients or other parameters, and general smoothness of the contour patterns.

12.2.6 Design runoff coefficients

(a) The 10-year runoff coefficients for the adopted design procedure are presented as a map of contours or isopleths of values in Map 4 at the back of this report. Based on local conditions, some small-scale variations of coefficients from the values given by the contours may be justified. The coefficient values are generally high near the coast, in some locations being greater than unity, which is possible with the statistical interpretation of the Rational Method. The values decrease rapidly away from the coast, and are low on the western slopes at the limit of eastern New South Wales covered by the study. The patterns of the contours reflect variations in average annual rainfall, rainfall intensities, and topography and relief.

(b) Although the mapped contours showed a good general agreement with the individual derived 10-year coefficients, considerable deviations were also evident. The largest of the deviations were examined, but no general explanation could be determined, apart possibly from rating curve extension problems. Based on sampling error theory, a probability distribution of expected deviations resulting from sampling errors in the basic observed flood data was derived. This distribution was very similar to that of the observed deviations, indicating that to a very large degree, these deviations can be accounted for by the random scatter inherent in the observed sample of floods available for analysis. This also means that correlations between the deviations and catchment characteristics cannot be expected, and it is not really statistically valid to attempt to extract more information from the data than the mapped contours. An attempt to find such correlations revealed no significant relations.

(c) East of the line on Map 4 at the back of this report joining Ashford, Tamworth, Bathurst, Yass, Tumut and Jingellic, the catchments analysed sampled a wide range of sizes. As no correlations between coefficient values and catchment areas or other characteristics were evident, the values from the map can be used as design 10-year runoff coefficients. West of this line, the data base was sparse as only thirty eight catchments were available and all but three of these are larger than 100 km^2, so that the mapped contours are really only applicable to catchments larger than this size. Based on the available evidence, the following procedure was adopted for design 10-year coefficients west of the line on Map 4. For catchments larger than 100 km^2, the values can be read directly from the map. For catchment areas smaller than 100 km^2, the value from the map should be used if it is greater than 0.40. If it is less than 0.40, the map value should be multiplied by $(100/A)^{0.15}$ where A is in km^2, but with a maximum value of 0.40. However, if the adjusted value is less than 0.20, a design value of the 10-year coefficient of 0.20 should be used. This procedure adopted for the western part of eastern New South Wales must be considered to be of an approximate nature. It will be difficult or impossible to improve it until at least ten years of data are available from a considerable number of small catchments in the region.

(d) Average frequency factors by which to multiply 10-year coefficients to obtain Y-year runoff coefficients were derived for three regions in eastern New South Wales. For return periods of 50 and 100 years, the frequency factors also depend on the rainfall intensity. This approach was adopted rather than the two alternatives of regressions of frequency factors of individual catchments on catchment variables for eastern New South Wales as a whole or for each of the three regions. The average values for each region are simpler to use, of similar accuracy to the other two approaches, and lead to more consistent values between catchments and for different return periods.

(e) Discontinuities of floods estimated by the design procedure occur at the boundaries of the rainfall zones in ARR. A linear interpolation procedure was adopted to smooth these discontinuities for locations within 25 km of a zone boundary.

12.2.7 Economic significance of use of the design method

(a) Flood peaks predicted by the Rational Method procedure in ARR and by the procedure developed in this report were compared with the flood frequency estimates from the observed data on 271 catchments used in the study. Although the comparison was not entirely unbiased, the procedure developed here was shown to be of much greater accuracy than the ARR method. The latter was also shown to be biased in that flood estimates tended to be too low near the coast and too high inland. Results of the comparison were not affected when only the 75 catchments with the most reliable records and when different return periods were considered. The procedure in this report was also shown to be more accurate than the method of Cordery and Webb (1974).

(b) As noted in Section 12.1, two economic analyses were carried out on the likely costs of the inaccuracies involved in design flood estimates. The much greater accuracy of the procedure developed in this report could lead to savings of about $12 million per year or 30 per cent of total costs for flood-passing structures on small catchments in eastern New South Wales.

(c) Although the procedures can only be considered to be approximate, data are presented in the report on capital costs and damage costs for small bridges and culverts, and a methodology is developed for the economic analysis of the effects of inaccurate design flood estimates for these structures.

(d) For the full potential savings to be realised, further research on selection of the optimum design return period is required.

12.2.8 Bankfull discharge estimates for western New South Wales

(a) Insufficient observed data are available in arid and semi-arid western New South Wales for application of the methodology for the statistical Rational Method used in the east of the State. Channel transmission losses and partial area runoff would also cause difficulties.

(b) Bankfull discharges were estimated at 75 sites in undulating to hilly country along the highways connecting Tibooburra, Broken Hill, Nyngan and Orange. The seven sites in the more humid region from Nyngan to Orange were found to have different characteristics to the remaining 68, and were omitted from further analysis.

(c) A regression equation relating bankfull discharge to catchment area and equal area slope was derived for the 68 sites.

(d) Although the available data could not provide definitive results, analysis of four types of evidence indicated that the average frequency of bankfull flows in western New South Wales is approximately 2.5 years.

(e) Approximate average frequency factors were derived from 22 stations in or close to the arid zone of western New South Wales.

(f) Combined use of the three relationships in (c) to (e) above provides an approximate method for estimating design floods in undulating to hilly country in western New South Wales.

(g) Estimates of bankfull discharge can be very useful in providing approximate information where no observed data are available. More work would be desirable on procedures for estimating bankfull discharges and comparing them with observed flows.

12.2.9 Availability of streamflow data

(a) The project has shown that in regions such as eastern New South Wales, sufficient streamflow data for small catchments are now available for worthwhile analysis on a regional basis. There is now no excuse for design methods on which huge total expenditures depend to be based only on judgment and experience. This would apply to several regions in Australia as well as to eastern New South Wales.

(b) Computer processed data in the form of computer files or printouts such as provided by the Water Resources Commission of New South Wales in this project are extremely helpful and enable large-scale analysis on a regional basis. However, it has been demonstrated in this and many other projects that careful examination of the data is desirable, and that the quality and completeness of the data base can be improved appreciably with effort and attention.

(c) Despite the availability of data in a processed form as discussed in (a) and (b) above, there are still many inadequacies in the available data, and collection of more data for small catchments remains a pressing need. It would also lead to appreciable economic returns, as demonstrated by the large potential savings that would result from adoption of the design procedure developed in this report. Streamflow data from small catchments were not available for many regions in eastern New South Wales, and necessitated the use of data from catchments greater than 250 km^2 in area.

The relatively short lengths of record from the catchments that are gauged result in the large sampling errors illustrated in this project. More accurate design data will not be possible without more and longer records.

(d) Data deficiencies were particularly obvious in arid and semi-arid western New South Wales. Short records are only available at two locations. Although they are difficult to obtain, there is a need for the gauging of more small catchments in western New South Wales.

(e) The situation is only a little better in the sub-humid western slopes of New South Wales. West of a line joining Ashford, Tamworth, Bathurst, Yass, Tumut and Jingellic, suitable data were available from only thirty eight catchments. All but three of these were larger than 100 km^2, so that virtually no information was available for catchments smaller than this size. There is thus a need for the collection of streamflow data from catchments smaller than 100 km^2 on the western slopes.

(f) The investigations carried out in the project have again illustrated the fact that any design procedure can only be as good as the data from which it was derived.

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APPENDIX A. SUMMARY OF THE STREAM GAUGING STATIONS USED IN THIS PROJECT

Five items of information about each of the 290 gauging stations used in this project are presented in Table A1. These are the national gauging station number, the current name of the station, the adopted record length, the catchment area and the relative reliability index assigned to the flood frequency curve derived for the station.

National gauging station numbers and station names are taken from 'Stream Gauging Information Australia 1974' (Department of National Resources, 1976), which was the latest edition available during the first part of the project.

The record length adopted refers to the length (in years) of the partial series used to derive the flood frequency curve for the station.

Catchment area refers to the area of the catchment (km^2) as extracted from available maps.

The relative reliability indices were assigned using the procedures described in Appendix D and represent the degree of acceptability of the flood frequency curve derived for each catchment in this project. An index of 1 represents the highest relative reliability, and 5 represents the lowest.

The stations listed after the sub-heading '55 Additional Stations' are those of the total of 290 which were included late in the study as noted in Section 1.5. Most of these catchments have areas larger than 250 km^2, and they were included to indicate general flood runoff potential in regions where little or no other data were available. Information on these additional catchments was obtained mainly from the report series 'Survey of Thirty Two River Valleys' (NSW Water Conservation and Irrigation Commission, various dates).

Table A1 Listing of Gauging Stations Used in the Project

NATIONAL STATION NUMBER	NAME OF STATION	ADOPTED RECORD LENGTH (YEARS)	CATCHMENT AREA (KM^2)	RELATIVE RELIABILITY INDEX OF FLOOD FREQUENCY CURVE
201001	Oxley at Eungella	28.0	213.00	2
201004	Tweed at Kunghur	23.0	49.00	4
201005	Rous at Boat Harbour	28.0	111.00	3
202001	Brunswick R @ Durrumbul			
203002	Coopers At Repentance	26.0	62.00	2
203007	Terania at Blakes	22.0	44.00	4
203009	Back at Bentley	21.2	109.00	2
203012	Byron at Binna Burra	26.0	39.00	4
203013	Wilsons at Federal	24.0	54.00	3
203014	Wilsons at Eltham	20.1	223.00	2
203023	Iron Pot at Toonumbar	7.4	98.00	5
204006	Bookookoorara at Undercliffe	10.0	127.00	5
204008	Guy Fawkes at Ebor	17.0	31.00	2
204011	Deer Park at Deervale	11.0	9.00	4
204016	Little Murray at Nth Dorrigo	27.4	104.00	2
204017	Bielsdown at Dorrigo	30.0	82.00	1
204019	Nymboida at Bostobrick	25.0	220.00	2
204020	Blicks at Dundurrabin	29.0	251.00	1
204021	Blicks at Hernani	26.5	70.00	1
204022	Dandahra at the Huts	11.0	39.00	4
204023	Dandahra at Dam Site	11.0	26.00	5
204024	Wild Cattle at Megan	26.0	31.00	1
204025	Orara at Karangi	24.0	132.00	3

TABLE A1. Cont/d...

NATIONAL STATION NUMBER	NAME OF STATION	ADOPTED RECORD LENGTH (YEARS)	CATCHMENT AREA (KM^2)	RELATIVE RELIABILITY INDEX OF FLOOD FREQUENCY CURVE
204026	Bobo at Bobo Nursery	26.0	80.00	1
204027	Little Nymboida at Timmsvale	18.0	31.00	2
204030	Aberfoyle at Aberfoyle	26.0	200.00	2
204032	Rocky at Glen Elgin	10.0	60.00	4
204035	Boonoo Boonoo at Wilnor	10.5	135.00	4
204036	Cataract at Sandy Hill	23.0	300.00	2
204037	Clouds at Clouds Ck.	10.0	62.00	4
204038	Sheep Station, U/S Clouds Creek Junction	10.0	18.00	4
204040	Kooreelah at Hewetsons Hill	19.0	231.00	3
204043	Peacock at Bonalbo	13.0	47.00	4
204044	Gorge at Bonalbo	16.0	41.00	4
205007	Woolgoolga at Woolgoolga	18.0	11.00	4
206001	Styx at Jeogla	47.0	163.00	1
206004	Gara at Gara	36.0	407.00	2
206005	Oakey at Kempsey Road	15.2	202.00	3
206009	Tia at Tia	37.0	251.00	1
206010	Yarrowitch at Yarrowitch	30.0	70.00	1
206013	Oakey at Yooroonah	12.5	47.00	5
206015	Chandler at Euringilly	27.0	205.00	1
206017	Serpentine at the Hatchery	26.0	22.00	2
206020	Styx at Serpentine	15.0	78.00	4

206020	Styx at Serpentine	15.0	78.00	4

TABLE A1. Cont/d...

NATIONAL STATION NUMBER	NAME OF STATION	ADOPTED RECORD LENGTH (YEARS)	CATCHMENT AREA (KM^2)	RELATIVE RELIABILITY INDEX OF FLOOD FREQUENCY CURVE
206021	Oakey U/S Oakey Dam	19.0	135.00	2
206023	Georges at Big Hill	13.2	130.00	4
207003	Ellenborough at Glenwarren	24.7	60.00	4
207006	Forbes at Birdwood	17.5	363.00	4
207008	Stewarts at Stewarts River	9.0	60.00	4
207009	Camden Haven at Kendall	6.6	181.00	5
208001	Barrington at Bobs Crossing	28.0	21.00	2
208002	Manning at Tomalla	21.0	52.00	3
208007	Nowendoc at Nowendoc	27.0	218.00	1
208008	Gloucester at Forbesdale	26.0	207.00	3
208009	Barnard At Barry	20.0	150.00	4
208015	Lansdowne at Lansdowne	6.5	96.00	5
209005	Wallamba near Nabiac	10.0	259.00	4
210011	Williams at Tilligra	42.0	194.00	1
210017	Moonan at Moonan	35.0	98.00	1
210019	Omadale at Roma	34.0	104.00	1
210022	Allyn at Halton	26.0	205.00	1
210025	Stewarts at Windamere	19.0	168.00	4
210026	Congewoi at Eglingford	27.0	85.00	4
210029	Rouchel at Upper Rouchel	20.0	330.00	4
210037	Krui at Neverfail	15.0	585.00	5
210042	Bowmans at Ravensworth	22.0	205.00	2

TABLE A1. Cont/d...

NATIONAL STATION NUMBER	NAME OF STATION	ADOPTED RECORD LENGTH (YEARS)	CATCHMENT AREA (KM^2)	RELATIVE RELIABILITY INDEX OF FLOOD FREQUENCY CURVE
210043	Saddlers at Bowfield	19.0	78.00	4
210045	Saltwater at Plashett	21.0	41.00	4
210046	Goulburn at Ulan	22.0	153.00	2
210049	York at Ravensworth	11.0	9.00	4
210053	Fishery at Kurri Kurri	17.5	83.00	3
210054	Wallis at Richmond Vale	17.0	95.00	3
210059	Gardiners at Liddell	18.0	70.50	5
210063	First at Pokolbin Site 1	14.0	14.80	3
210067	Middle at Pokolbin	15.0	7.80	3
210068	Deep at Pokolbin Site No. 3	13.3	24.90	3
210069	Deep at Pokolbin Site No. 4	13.3	4.90	3
210074	McMahons at Liddell No. 5	9.5	1.04	4
210076	Gardiners at Antiene	9.3	14.20	5
210078	Tinkers Sth at Liddell No. 1	9.0	6.50	5
210084	Glennies at the Rocks No. 2	5.0	249.00	5
210999(ASS)	Scone - SCS	21.0	.18	2
211001	Wyee at Wyee	18.5	18.00	4
211005	Ourimbah at Tuggerah	12.0	150.00	3
211006	Wallarah at Warnervale	10.0	8.80	4
211008	Sandy at Avondale	8.0	52.00	4
212003	Burralow at Kurrajong	36.0	25.90	1
212008	Cox's at Bathurst Road	27.0	199.00	2

... Road 27.0 199.00 2

TABLE A1. Cont/d...

NATIONAL STATION NUMBER	NAME OF STATION	ADOPTED RECORD LENGTH (YEARS)	CATCHMENT AREA (KM^2)	RELATIVE RELIABILITY INDEX OF FLOOD FREQUENCY CURVE
212012	Wollondilly at Goulburn	15.0	622.00	4
212013	Megalong at Narrow Neck	10.0	24.60	4
212014	Blackheath at Mt. Boyce	9.5	18.90	4
212016	Kedumba at Kedumba Crssing	8.0	75.00	5
212019	Mangrove at Fairview	7.5	202.00	4 5
212209 203	Nepean at Pheasants Nest Nepean at Maguires Crossing	26.0	72.50	2
212210	Avon at Avon Weir	24.0	148.00	4
212231	Cataract at Jordans Crossing	46.0	163.00	4
212291	Grose D/S Burralow Ck.	28.0	650.00	2
212301	Coxs at Lidsdale No. 1	14.1	.06	4
212302	Coxs at Lidsdale No. 2	13.0	.13	5
212304	Coxs at Lidsdale No. 4	13.5	.08	4
212306	Coxs at Lidsdale No. 6	13.5	.11	5
212307	Coxs at Lidsdale No. 7	14.0	.04	5
212320	South at Mulgoa Rd	24.0	89.60	1
212333	Mt. Vernon at Mt. Vernon	23.5	.70	1
212340	Eastern at the Bridge	22.0	24.90	3
213200	O'Hares at Wedderburn	47.0	74.50	2 = 5 overestimates Q by about 2.5 x
214003	Macquarie at Albion Park	20.3	31.00	4
214310	Kellys at Kellys Creek	12.8	2.54	4
214320	Boora at Boora Creek	17.0	.93	2
214330	Research at Research Creek	8.5	.39	4

TABLE A1. Cont/d...

NATIONAL STATION NUMBER	NAME OF STATION	ADOPTED RECORD LENGTH (YEARS)	CATCHMENT AREA (KM^2)	RELATIVE RELIABILITY INDEX OF FLOOD FREQUENCY CURVE
214334	Cawleys at Lower Cawleys	17.5	5.52	2
214340	Hacking at Upper Causeway	18.0	40.20	2
215004	Corang at Hockeys	52.0	166.00	1
215006	Mongarlowe at Mongarlowe	18.0	130.00	3
215009	Endrick at Nowra Road	23.0	210.00	3
215010	Kangaroo at Kangaroo Valley	19.0	241.00	2
215223	Brogers at Clinton Park	11.7	67.30	3
215233	Yarrunga at Wildes Meadow	10.0	7.30	3
216001	Currowan at Shallow Crossing	7.0	161.00	4
217003	Araluen at Araluen Lower	8.0	129.00	5
218001	Tuross at Tuross Vale	19.0	93.00	5
218003	Yowrie at Yowrie	20.0	103.00	3
218006	Wandellow at Wandella	11.5	64.00	4
219001	Rutherford at Brown Mtn.	23.0	18.60	4
219004	Tantawanglo at Tantawanglo	20.0	148.00	2
219006	Tantawanglo at Tantawanglo Mtn	15.0	88.00	4
219008	Nunnock at Brown Mtn.	9.0	18.00	4
219009	Yankeys D/S Bega Swamp	10.0	7.80	5
219010	Bonar at Brown Mtn.	16.0	3.60	4
219015	Nutleys near Bermagui	10.0	25.00	4
219016	Narira near Cobargo	12.5	90.00	4

219016 Narira near Cobargo 12.5 90.00 4

TABLE A1. Cont/d...

NATIONAL STATION NUMBER	NAME OF STATION	ADOPTED RECORD LENGTH (YEARS)	CATCHMENT AREA (KM^2)	RELATIVE RELIABILITY INDEX OF FLOOD FREQUENCY CURVE
219017	Double at Brogo	12.1	144.00	4
219018	Murrah at Quaama	6.0	38.00	5
219020	Sandy at Mogilla	11.5	30.00	4
219021	Bemboka at Bemboka	13.0	124.00	4
220002	Stockyard at Rocky Hall	14.0	72.00	5
220003	Pambula at Lochiel	11.5	129.00	4
222012	Coolumbooka near Bombala	13.0	160.00	2
222400	Moyangull at Lookout	8.7	28.50	4
222404	Mellick Munjie at Gillingall	13.3	69.90	4
222513	Perisher at Blue Cow	15.8	12.40	3
222522	Eucumbene at Providence No.2	17.0	165.00	3
224402	Moroka at Horse Yard	14.5	49.70	3
401006	Paddys, above Granite Falls	26.0	116.00	2
401007	Tumbarumba at Tumbarumba	28.0	134.00	2
401009	Maragle at Maragle	26.4	220.00	1
401205	Tallangatta at Tallangatta	34.3	743.00	1
401508	Cootapatamba at Ramshead	10.8	5.00	5
401517	Khancoban at Bradneys Gap	13.5	38.90	3
402400	Watchbed at Bogong High Plains	29.5	3.00	1
403213	Fifteen Mile at Greta Sth	11.0	223.00	3
405229	Wanalta at Wanalta	9.0	109.00	5
407214	Creswick at Clunes	26.5	308.00	3

TABLE A1. Cont/d...

NATIONAL STATION NUMBER	NAME OF STATION	ADOPTED RECORD LENGTH (YEARS)	CATCHMENT AREA (KM^2)	RELATIVE RELIABILITY INDEX OF FLOOD FREQUENCY CURVE
407221	Jim Crow at Yandoit	15.5	163.00	3
410009	Jounama at Talbingo	35.0	134.00	1
410010	Yarrangobilly at Yarrangobilly	14.4	97.90	4
410029	Buddong at Buddong Falls	30.0	29.50	2
410034	Gilmore at Batlow	32.0	95.50	1
410059	Gilmore at Gilmore	30.0	233.00	2
410061	Adelong at Batlow Road	28.0	155.00	2
410063	Rock Flat near Bunyan	25.0	220.00	4
410066	Nacka Nacka at Thuro	24.0	124.00	3
410067	Big Badja at Numeralla	25.5	220.00	3
410070	Bumbolle at Bombowlee	16.0	67.00	4
410071	Brungle at Red Hill	14.3	114.00	4
410075	Kybeyan at Kybeyan	19.2	69.00	3
410076	Strike-a-light at Jerangle Rd	21.5	217.00	3
410077	Bredbo at Laguna	19.0	75.00	4
410080	Billabung at Glenfield	15.0	708.00	4
410081	Cooma at Cooma No. 2	12.0	103.00	3
410090	Yass at Gunderoo	10.00	320.00	4
410351	Wagga - SCS	28.0	.07	1
410506	Yarrangobilly at Hospital Flat	18.5	227.00	2
410507	Wallaces at Hospital Flat	18.5	43.50	2
410514	Goorudie at Bolaro	18.0	116.00	2

410514 Goorudie at Bolaro 18.0 116.00 2

TABLE A1. Cont/d...

NATIONAL STATION NUMBER	NAME OF STATION	ADOPTED RECORD LENGTH (YEARS)	CATCHMENT AREA (KM^2)	RELATIVE RELIABILITY INDEX OF FLOOD FREQUENCY CURVE
410523	Honeysuckle, below Maragle Road	16.0	32.10	2
410524	Murrumbidgee at Rules Pt.	11.4	104.00	3
410533	Tumut, above Happy Jacks	11.8	131.00	3
410534	Happy Jacks above Happy Jacks Pondage	12.0	109.00	3
410535	Murrumbidgee, above Tantangarra Reservoir	12.4	216.00	2
411001	Mill Post at Bungendore	15.2	15.50	4
412031	Hovels at Hovels Crk.	30.00	272.00	2
412063	Lachlan at Gunning	17.0	570.00	3
412064	Bolong at Golspie No.2	14.3	181.00	3
412068	Goonigal at Gooloogong	10.0	363.00	5
412073	Nyrang at Nyrang Ck.	12.0	225.00	4
412074	Isabella at Ballyroe	12.0	127.00	5
412075	Mandagery at Manildra	12.0	350.00	4
412077	Belubula at Carcoar	10.5	233.00	4
412090	Boree at Cudal No.2	6.5	272.00	4
415202	MacKenzie at Wartook	82.0	80.30	1
415217	Fyans at Grampians Rd. Bridge	9.0	36.30	4
416024	Swan at Campbells	11.0	181.00	4
416303	Pike at Barelli	35.0	1036.00	3
416304	Quart Pot at Upper Eukey	24.0	70.00	3

TABLE A1. Cont/d...

NATIONAL STATION NUMBER	NAME OF STATION	ADOPTED RECORD LENGTH (YEARS)	CATCHMENT AREA (KM^2)	RELATIVE RELIABILITY INDEX OF FLOOD FREQUENCY CURVE
416404	Bracker at Terraine	17.0	686.00	3
416407	Canning at Woodspring	15.0	1243.00	5
418025	Halls at Bingara	13.0	116.00	4
418027	Horton at Dam Site	10.0	220.00	4
418030	Copes at Tingha	11.1	70.00	4
419029	Halls at Ukolan	11.0	389.00	4
419036	Duncans at Woolomin	9.0	100.00	4
419037	Mulla at Bullimball	7.7	277.00	4
419038	MacDonald at Cobrabald	12.7	358.00	4
419044	Maules at Dam Site	9.0	171.00	4
420013	Castlereagh at Coonabarabran No.2	26.0	124.00	3
420003	Belar at Warkton	18.0	142.00	4
420009	Merrygoen at Mendoran	11.0	331.00	5
420011	Baronne near Gulargambone	7.5	320.00	5
421010	Bogan at Peak Hill No.1	43.0	543.00	1
421032	McKeons at Dam Site	12.0	34.00	4
421033	Bindo, D/S of Gum Valley Ck.	19.0	34.00	3
421034	Slippery at Dam Site	23.0	15.50	2
421036	Duckmaloi Below Dam Site	21.0	104.00	3
421038	Cudgegong at Rylstone	18.0	544.00	4
421041	Crudine, U/S of Turon Junction	15.0	280.00	4

Junction 15.0 280.00 4

TABLE A1. Cont/d...

NATIONAL STATION NUMBER	NAME OF STATION	ADOPTED RECORD LENGTH (YEARS)	CATCHMENT AREA (KM^2)	RELATIVE RELIABILITY INDEX OF FLOOD FREQUENCY CURVE
421050	Bell at Molong	8.0	365.00	4
421051	Blackmans Swamp near Orange	13.0	31.00	5
421053	Queen Ch. Vale at Georges Plains	9.0	202.00	5
421056	Coolaburragundy at Coolah	12.0	225.00	5
421066	Green Valley at Hill End	11.0	119.00	4
421067	Pyramul at Hill End	11.1	179.00	5
422301	Condamine at Long Crossing	65.0	80.00	1
422302	Spring Nth at Killarney	47.0	21.00	1
422303	Spring Sth at Killarney	47.0	10.00	1
422305	Emu at Gillespies	21.3	98.00	4
422313	Emu at Emu Vale	30.0	140.00	2
422321	Spring at Killarney	13.0	34.0	4
422306	Swan at Swanfels	57.0	83.00	2
422319	Dalrymple at Allora	22.0	254.00	3
425016	Box at Cobar	7.0	15.00	5
011001	Homestead at Fowlers Gap	7.0	19.90	5
011315	Nelia at Nelia Dam	3.7	4.00	5
006003	Gillen at Soil Erosion Project	11.0	3.80	4
006009	Todd at Wills Tce.	25.0	450.00	2
006047	Charles at Big Dipper	20.0	42.00	3

TABLE A1. Cont/d...

NATIONAL STATION NUMBER	NAME OF STATION	ADOPTED RECORD LENGTH (YEARS)	CATCHMENT AREA (KM^2)	RELATIVE RELIABILITY INDEX OF FLOOD FREQUENCY CURVE
<u>55 ADDITIONAL STATIONS</u>				
202001	Brunswick at Durrumbul	11.0	34.00	5
203010	Leycester at Rock Valley	15.0	179.00	4
203015	Goolmagar at Coffee Camp	15.0	109.00	5
204014	Mann at Mitchell	21.0	881.00	4
204015	Little at Broadmeadows	22.0	2670.00	5
204033	Rocky at Billyrimbah	16.0	985.00	4
205002	Bellinger at Thora	14.0	433.00	4
205003	Never Never at Slingsbys Rd	10.0	13.00	5
205004	Bellinger at Scotchmans	10.0	166.00	5
205006	Bowra at Bowraville	10.0	539.00	5
206008	Commissioners Water at Tiverton	18.0	383.00	2
206014	Wollombi at Coninside	25.0	376.00	2
206018	Apsley at Apsley Falls	23.0	894.00	3
209001	Karuah at Monkerai	20.0	202.00	5
210021	Paterson at Lostock	24.0	277.00	3
210034	Widden at Widden	24.0	733.00	5
210040	Wybong at Wybong	14.0	658.00	5
210052	Pages at Gundy	22.0	1050.00	3
211002	Wyong at Wyong	12.0	249.00	4
212011	Cox's at Lithgow	12.0	404.00	5

TABLE A1. Cont/d...

NATIONAL STATION NUMBER	NAME OF STATION	ADOPTED RECORD LENGTH (YEARS)	CATCHMENT AREA (KM^2)	RELATIVE RELIABILITY INDEX OF FLOOD FREQUENCY CURVE
212203	Nepean at Pheasants Nest	72.0	686.00	5
216002	Clyde at Brooman	9.0	880.00	5
217001	Moruya at McGregors Ck.	14.0	891.00	5
221202	Genoa at Wangarabell	14.0	777.00	4
222001	MacLaughlin at Dalgety Road	15.0	277.00	5
222004	Little Plains at Wellesley	31.0	621.00	3
222007	Wulwye at Woolway	23.0	556.00	4
222009	Bombala at The Falls	21.0	543.00	5
402202	Yackandandah at Allans Flat	24.0	282.00	3
403206	Buckland at Lower Buckland	35.0	302.00	1
404207	Hollands at Kelfeera	10.0	448.00	4
405208	Seven at Euroa	17.0	251.00	4
407213	McCallums at Carrisbrook	27.0	471.00	3
408200	Avoca at Coonooer	81.0	2670.00	2
412020	Abercrombie at Abercrombie	40.0	2770.00	3
415210	Richardson at Banyena Sth.	14.0	658.00	4
416003	Tenterfield at Clifton	46.0	570.00	2
416004	Mole at Trenayr	45.0	1606.00	4
416006	Severn at Ashford	32.0	3160.00	5
416008	Beardy at Haystack No.4	24.0	907.00	3
416010	Macintyre at Wallangra	29.0	2020.00	2

TABLE A1. Cont/d...

NATIONAL STATION NUMBER	NAME OF STATION	ADOPTED RECORD LENGTH (YEARS)	CATCHMENT AREA (KM^2)	RELATIVE RELIABILITY INDEX OF FLOOD FREQUENCY CURVE
416401	Macintyre at Whetstone	29.0	3650.00	5
418014	Gwydir at Yarrowyck	11.0	855.00	5
419004	Peel at Bowling Alley Point	54.0	310.00	1
419015	Peel at Pialamore	32.0	1140.00	2
419016	Cockburn at Mulla Crossing	32.0	907.00	2
419020	Manilla at Brabri	21.0	2020.00	4
419024	Peel at Paradise Weir	15.0	2409.00	5
419027	Mooki at Breeza	13.0	3630.00	5
420004	Castlereagh at Mendoran	13.0	3470.00	5
421007	Macquarie at Bathurst	58.0	2771.00	2
421035	Fish U/S Tarana Rd Bridge	13.0	570.00	4
422307	King at King Ck.	31.0	332.00	2
422317	Glengallon at Rocky Pond	20.0	466.00	3
422318	Sandy at Allan	14.0	648.00	5

APPENDIX B. THE NUMBER OF DATA VALUES FOR FITTING A PROBABILITY DISTRIBUTION TO A PARTIAL SERIES

B.1 THE PROBLEM

In Chapter 2 it was decided to use a probability distribution to fit flood frequency curves to the partial series flood data and hence avoid the subjectivity involved in drawing arbitrary curves through the plotted data by eye. The distribution chosen was the Log-Pearson Type III (referred to here as LP3).

The partial series was defined as the time series of events which exceeded a selected base level magnitude. Current US Geological Survey practice is to select this base magnitude such that 3N values are used in the partial series (where N is the length of record in years). When a probability distribution is used and is fitted by the method of moments, there exists a problem as to how many data points to use, as the values of the moments of the data series depend on the number of values used in the series. This then causes changes in the parameters and hence in the shape and position of the derived flood frequency curve. This problem does not occur with the annual series, as the number of values in the data series is fixed at N. An investigation was therefore carried out to determine the appropriate number of data points to use in fitting the partial series.

B.2 APPROACH USED IN THE INVESTIGATION

The general approach adopted in the investigation was that for selected gauging stations, long partial series (up to 4N events) were first extracted from the records, and these events were ranked and plotted on log-normal probability paper using equation (2.1) from Chapter 2. Then for each station, moments were calculated and LP3 distributions fitted to several subsets of each of the original series containing different numbers of the highest events. These distributions fitted to the series of different lengths (or numbers of data points) were then plotted on the same graph as the original full number of observed values and compared to them and with each other.

Partial series data were extracted for nine stations, representing a range of flood frequency curve shapes and record lengths. These stations were 201005, 203013, 204043, 205007, 206020, 212304, 214320, 214334 and 421033. Between 3N and 4N events were extracted from each station record and ranked. Sub-series were formed from these full series starting with the highest ten peaks. Subsequent sub-series were formed by adding constant increments in the number of data points such that each series included the data points from the previous one.

The top N events were plotted for each station on log-normal probability paper, representing the frequency of the observed magnitudes. Each sub-series for the station was fitted using the LP3 distribution (method of moments) so that a probability distribution was obtained for each, with shift, scale and shape parameters of the distributions calculated. Occurrence probabilities for each of these distributions were adjusted to allow for the use of different numbers of data points for the same record length (as in Jennings and Benson, 1969).

B.3 RESULTS

The results for each station were summarized in the form of a table showing the variation of flood frequency curve statistics with variation of n, the number of largest floods used in the partial series. The resulting flood frequency curves of the fitted distributions were also plotted.

Due to space limitations, results are only presented for the two stations 201005 and 203013, which illustrate results for stations with low and high skew coefficients respectively. Tables B1 and B2, and Figures B.1 and B.2 present these results.

B.4 DISCUSSION OF RESULTS

From a study of the plotted frequency curves it was noted that for station records with high positive skew coefficients, there was very little difference between the sub-series plots for the different data increments (Figure B.2). Conversely, for stations with low to negative skew, there was a large difference between the plots (Figure B.1). The fitted frequency curves diverged from the plotted observed data as the number of data points considered in the partial series was increased. This divergence is greatest for the high return periods so that the flood magnitudes predicted become larger as the number of data points considered increases. The differences from plotted points were often up to at least 200%.

An examination of the summary tables reveals that the shape paramater b changes drastically from one data increment to the next, and tends to become constant when the partial series contains 3N or more data points. This shape parameter b is indicative of the shape of the probability distribution calculated for the series. The stations for which the plotted flood frequency curves exhibited little change when different numbers of data points were used in the partial series also exhibited little change in the shape parameter. Conversely the stations showing large divergence in flood frequency curve plots showed large differences in shape parameter values. This effect can be explained by considering the effects on the fitted probability distribution of adding incremental numbers of data values to the partial series. From the summary tables (e.g. B1 and B2) the mean of the logs decreases and the standard deviation increases. The standard deviation is related to the second moment of the fitted distribution about the mean, and as such its increase reflects the larger relative spread of the data. The value of b depends on the skewness coefficient, as b is calculated as:

$$b = \frac{4}{\gamma^2} - 1 \tag{B.1}$$

where b = shape parameter
and γ = skewness coefficient of partial series data

Where large changes occur in the skewness coefficient, large changes will also occur in the value of b and hence in the shape of the fitted distribution. The largest changes in b occur with very small values of skewness, especially as it approaches zero. For skewness approaching zero the shape parameter approaches infinity, so that a partial series with low skew (smaller than 0.5, say) will be fitted with a distribution of considerably different shape to that fitted to a partial series with a higher skew. Very different flood frequency curves should thus be expected.

Of the 290 stream gauging station records used in this project, a considerable number have small skew soefficients (see Figure 2.2, Chapter 2). If the number of data points used in the partial series for flood frequency curve calculation was different to N for these stations, the differences between the predicted and observed flood magnitudes would be expected to be of similar magnitude to those shown in Figure B.1. This would obviously be unsatisfactory, as the sample of observed data represents the best means available for locating the flood frequency curve for the station.

The policy was therefore adopted that the number of data values to be used for calculating the LP3 distribution for frequency analysis of the partial series at a particular station should be N, the number of years of record at the station.

Table B1. Example of Variation of Flood Frequency Curve Statistics with Number n of Date Values in Partial Series - Station with a Series with Low Skewness

River Basin:- Tweed
Station:201005: Rous at Boat Harbour
Catchment Area: 111 km^2
Record Length: 17.3 years (N)

Number of Largest Floods Used (n)	Mean of Logarighms of Floods($\bar{x}$)	Standard Deviation of Logs (S)	Skewness Coefficient of Logs (γ)	Kurtosis Coefficient of Logs(Ck)	Parameters of fitted LP3 distbn. Shift (m)	Scale (a)	Shape (b)
10	2.859	0.057	0.302	3.14	2.479	0.0087	42.8
20	2.700	0.211	-0.949	4.35	3.146	-0.1003	3.44
30	2.512	0.323	-0.127	3.02	7.593	-0.0205	247
40	2.344	0.409	0.019	3.00	-40.95	0.0039	11188
50	2.202	0.465	0.222	3.07	-1.994	0.0516	80.3
60	2.080	0.506	0.369	3.20	-0.668	0.093	28.4
70	1.975	0.535	0.484	3.35	-0.239	0.129	16.1
80	1.868	0.576	0.483	3.35	-0.516	0.139	16.1

Table B2. Example of Variation of Flood Frequency Curve Statistics with Number n of Data Values in Partial Series - Station with a Series with High Skewness

River Basin: Richmond
Station: 203013: Wilsons at Federal
Catchment Area: 54 km^2
Record Length: 17.5 years (N)

Number of Largest Floods Used (n)	Mean of Logarithms of Floods ($\bar{x}$)	Standard Deviation of Logs (S)	Skewness Coefficient of Logs (γ)	Kurtosis Coefficient of Logs (Ck)	Parameters of fitted LP3 Dist. Shift (m)	Scale (a)	Shape (b)
10	2.659	0.199	1.021	4.565	2.270	0.102	2.834
20	2.463	0.251	0.831	4.036	1.858	0.104	4.790
30	2.333	0.277	0.955	4.368	1.754	0.132	3.385
40	2.231	0.299	0.954	4.366	1.604	0.143	3.392
50	2.135	0.330	0.834	4.044	1.343	0.138	4.746

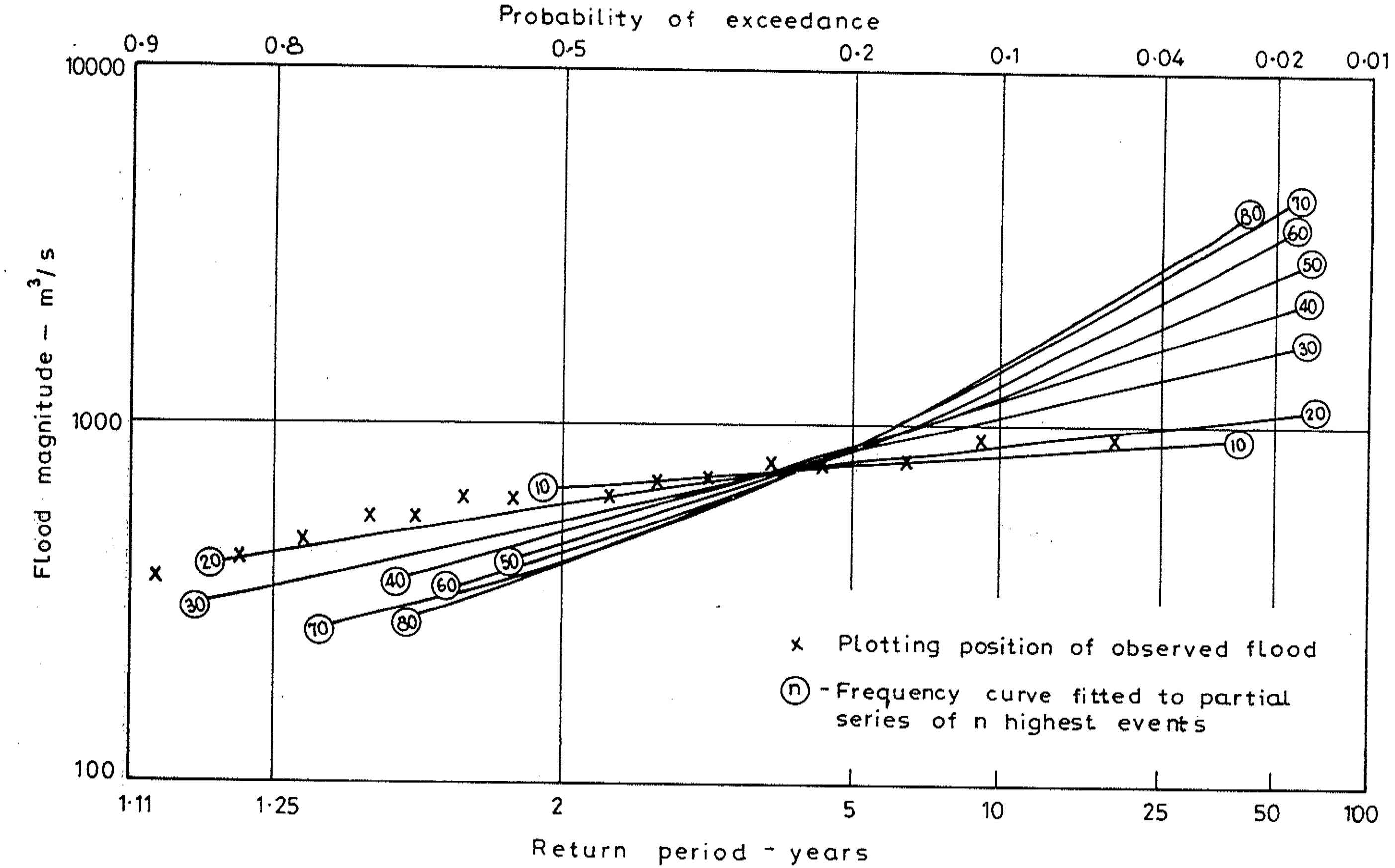

Figure B.1 Flood frequency curves fitted to partial series consisting of different numbers of the highest events for station 201005 with low skew. Effective length of record of 17.3 years.

record of 17.3 years.

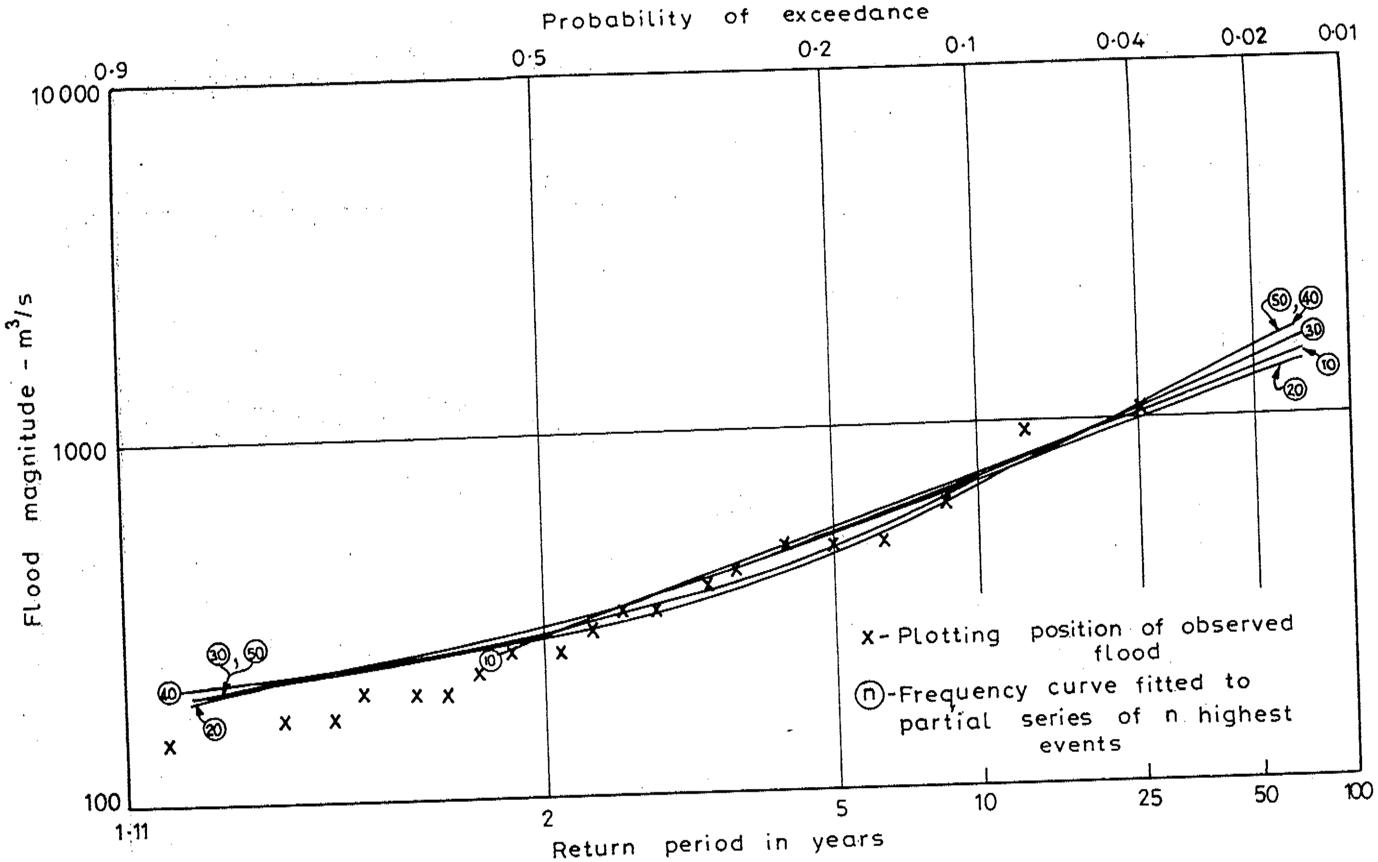

Figure B.2 Flood frequency curves fitted to partial series consisting of different numbers of the highest events for station 203013 with high skew. Effective length of record of 17.5 years.

APPENDIX C. TREATMENT OF MISSING FLOOD DATA

C.1 NEED FOR THE INVESTIGATION

Examination and processing of the stream flow data from recorder stations used in this project revealed the presence of an appreciable amount of missing and anomalous data. The former refers to the number of months where maximum monthly peak flow rates were not recorded or obtainable, and anomalous data refers to the number of months where maximum monthly peak flow rates were given but were anomalous in some way. The most common type of anomalous data was where the peak flow rate given for an event was the same as the daily average flow rate for the same event. The frequency distribution of the percentage of missing data for a sample of 206 stations is shown in Figure C.1. The 206 catchments represent those used in the project with their entire period of flows recorded on a continuous basis. The amount of missing data was of sufficient magnitude to warrant careful consideration of the procedure suggested in ARR (page 108), as following this practice would have caused the elimination of an appreciable amount of the data base. As described in Section 2.2.1, as much as possible of the missing and anomalous data was checked from the original recorder charts. The amount of data still missing after these checks was considerable, so that a decision was made to carry out an investigation to determine the best policy for treatment of missing data with respect to the effect of this treatment on the derived flood frequency curves.

C.2 APPROACH

Based on observation and discussion with experienced hydrographic personnel, the main causes of missing data were identified as:-

(a) those peculiar to the instrument (e.g. build up of condensation in the pressure line of the Bristol water level recorder).

(b) general - mechanical faults
- natural hazards
- man-related hazards

Of these the most common cause by far was clock stoppage. This is not usually related to flood event size (except when the instrument is submerged), but occurs as a random process dependent on the clock, the instrument and gauging personnel.

This random nature of the occurrence of missing data was used as a base from which to design an experiment to test the effect of the method of treating missing data employed in the derivation of the flood frequency curve for a station. Two types of investigations were carried out, one using observed data from stations with good records, and the other using synthetically generated data. The first approach will be discussed here, while the second is described later in Section C.5. In the first approach, the concept was to take a complete record of maximum monthly flows with no missing data and calculate the flood frequency curve for this record, calling it the 'true' curve. 'Holes' or 'gaps' were then made in this original series in a random manner to obtain a 'gapped' series. Each of several methods of treating missing data (described below) were then applied to the gapped series and flood frequency curves calculated for each. These curves were then plotted with the original 'true' flood frequency curve, and compared with it and each other for goodness of fit. A diagrammatic summary of this approach concept is given in Figure C.2.

Three separate cases were identified for investigation:-

Case 1 - Data missing from a station for which a nearby station record is available that reflects the magnitude of events as well as their time of occurrence.

Case 2 - Data missing from a station for which a nearby station record is available which reflects the time of occurrence of events but not their magnitudes. Daily read records, where peaks are known to have occurred but not their magnitudes, also fit into this category.

Case 3 - Data missing from a station for which no nearby station record is available.

C.3 METHODS FOR TREATMENT OF MISSING DATA FOR CASES 1 and 2

If a perfect correlation existed between two station records of flood peaks, then obviously the best method for estimating missing data in one station record would be to calculate the form of the relationship between the stations and to use this to fill in the missing periods of the record of the other station. However, in the real world, such perfect correlations do not exist and it is possible that on two adjacent catchments, a low ranking flood on one coincides with the highest ranking flood on the other, especially for small catchments.

Most gauging stations which are close spatially do exhibit a relationship of flood peaks. The records from one station can be used to indicate when a sizeable peak has occurred on the other station. However, the degree of correlation between event magnitudes is commonly quite poor. After study of catchment records, the 3Nth ranking peak was adopted to indicate the lower limit of flood magnitude on one catchment which could be accompanied by a peak larger than the Nth ranking peak occurring at the same time on a nearby catchment, where N is the number of years of record.

In a gapped series in this investigation, the original data series was taken as representing a nearby station record in that a 'flood event' was considered to have occurred in a gap if the original series value in that period exceeded the 3Nth ranking flood of the original series. Where there was no flood greater than the 3Nth ranking peak at a nearby station or in the original data series, the period of missing data was simply considered as part of the period of record.

The policies used by each method when the magnitude of an event in a gap in the original series exceeded the 3Nth ranking flood were:-

<u>Method A - ARR</u>
Eliminate the year and subtract one year from the record length

<u>Method B - Proportion</u>
Depending on the number of ranking events in the year subtract a proportional amount of the year from the record length and neglect the flood event.

e.g. If the missing peak is:

the only one in the year	-	subtract 1 year
one of two in the year	-	subtract $\frac{1}{2}$ year
one of three in the year	-	subtract 1/3 year

Methods C, D and E - Replacement by Synthetic Data
Data for each of the 12 months in the year were considered separately to account for seasonality. For the month in which the gap occurred, say January, all the ranking (above 3Nth) January flows in the entire record were listed. If there were less than four values, floods from the months either side (December and February) were included until at least four were obtained. Based on this small series of values, a frequency distribution of floods was estimated for the particular month, and the gap filled with a value drawn randomly from the distribution. The three types of distributions used are listed below:

Method C - Uniform Distribution
One of the series of values for the month was selected at random (with replacement) and placed in the gap.

Method D - Normal Distribution
Using the mean and standard deviation of the series of values for the month, a normally distributed random number was generated with these statistics and placed in the gap.

Method E - Log-Normal Distribution
As for method D, but using the logarithms of the values.

Method F - Nearby Station
This method applies only to Case 1 where data are available at a nearby station, and a relationship can be determined between the magnitudes of the floods at that station and at the station being analysed. It was found that more reliable relations could be found between daily flood volumes or average flows than between peak flows. The relation was used to estimate the average daily flow for a missing flood, and this was then converted to a peak flow from an average ratio of peak to average daily flows derived from recorded floods on the catchment being analysed.

Method G - Ignoring Missing Data
The fact that gaps were present was ignored, and the flood frequency calculation was based on the highest N flood peaks from the gapped series, where N was the record length of the original series including the gaps.

C.4 RESULTS OF INVESTIGATION USING OBSERVED DATA FOR CASES 1 AND 2

Nine station records were used to sample different regions, as summarised below:-

- four on the North Coast
- two on the South Coast
- three on the Western Slopes

The periods of record ranged from 9 to 15 years. Three gapped series were created randomly for each station record, with a range of missing data of 8 to 25% of the record. Methods A to E were applied to these series, as described in Sections C.2 and C.3. Method F (nearby station) was applied to the six stations which had a nearby station with concurrent records and of similar catchment size. Method G (ignore missing data) which was included in the study at the conclusion of this part of the investigation, was applied only to the gapped series for one station.

Frequency curves were calculated for each station for the series of the highest N floods made up of the events left in the record after the gaps were created, and the floods estimated by each of the methods for treating missing data. For each station, these curves were plotted with the frequency curve for the original ungapped record and its 5% and 95% confidence intervals. A typical example is shown in Figure C.3. The frequency curves for methods A to E were compared by eye with the curves for the original record and assigned ranks of 1 to 5, rank 1 being the best fit to the curve for the original record. The results are listed in Table C1, where the catchments are grouped by region. Equivalent ranks on the scale 1 to 5 for methods F and G where applicable are also listed in Table C1.

Methods A to E are applicable to both case 1 and case 2. For the results of these methods for all regions, the ARR method A was significantly worse than any of the other methods, while the Normal Distribution method D was slightly better than any of the other methods. All methods except ARR were inside the 5% to 95% confidence interval range, as illustrated in Figure C.3. This indicates that the ARR method is significantly worse than the other methods, while none of the other methods could be considered significantly better than the others. There was one gap creation trial however, which was an extreme exception to the above points. The gaps created excluded the first and second ranking floods from the series. Frequency curve plots for this trial fell outside the 95% confidence limit as shown in Figure C.4, so that none of the curves fitted the original series. This trial thus provides the critical situation regarding the effects of methods for treating missing data on the derived flood frequency curve.

The results of this investigation using observed data thus indicate that for case 2 where correlation with a nearby station is not possible, all methods except A are generally reasonable, and the replacement by the Normal Distribution in method D is marginally best. None of the methods are satisfactory in the extreme case discussed above.

For case 1 where correlation with a nearby station is possible, the results for method F on the six catchments are applicable. These results indicate a slightly better performance than method D. In particular, however, the nearby station method was the only method to fit within the 5% and 95% confidence intervals for the critical trial illustrated in Figure C.4. It fits the true curve well, in contrast to the other methods.

It can be concluded from the results that where it is possible to obtain a relation of flood flows with a nearby station, denoted here as case 1, method F is the best procedure, and is the only method likely to give satisfactory results in cases where all of the high floods are missing. The results also indicate that in all circumstances for cases 1 and 2, the method A recommended in ARR gives by far the worst results. These conclusions are dependent on the assumption discussed previously that the occurrence of periods of missing data is of a random nature and is unrelated to the occurrence of flood events.

C.5 INVESTIGATION USING GENERATED DATA

The second investigation was carried out to provide further information on case 2, to indicate a suitable procedure for case 3, and to verify the above conclusion regarding ARR method A. It was also desired to test method G where the missing data are ignored, as the application to the single catchment with observed data noted above had shown promising results.

The concept utilised here was to enlarge the sample of gap creation trials to test whether the conclusions drawn from the results for the observed data were merely due to chance. This was done by generating a

TABLE C1 Results of Investigation Using Observed Data for Cases 1 and 2

Region	Station Number	Trial Number	Ranks in Range 1 - 5 Assigned to Methods:-					Equivalent Rank for Method	
			A	B	C	D	E	F	G
North Coast	201002	1	2	5	2	4	2	<1	<1
	(Tweed)	2	4.5	3	4.5	1.5	1.5	<1	<1
		3	5	2.5	2.5	1	4	<1	<1
	210068	4	5	3	2	1	4	3	
	(Hunter)	5	5	1	2	3	4	1	
		6	3	3	3	3	3	3	
	204043	7	5	3.5	1.5	1.5	3.5	<1	
	(Clarence)	8	5	2	3	1	4	3	
		9	5	4	2.5	1	2.5	2.5	
	211005	10	5	1	4	3	2	<1	
	(Tuggerah)	11	5	3	4	1	2	3	
		12	5	4	2	1	3	1	
	Total		54.5	35	33	22	35.5	16.5	
	Av. Rank		4.5	2.9	2.8	1.8	3.0	1.4	
South Coast	218006	13	5	1	3	2	4	<1	
	(Tuross)	14	5	2	4	2	2	1	
		15	5	3	4	1.5	1.5	<1	
	220002	16	5	4	2	2	2		
	(Towamba)	17	5	4	1	2	3		
		18	5	2.5	2.5	2.5	2.5		
	Total		30	16.5	16.5	12	15		
	Av. Rank		5.0	2.8	2.8	2.0	2.5		
Western Slopes	410066	19	5	2	4	2	2	4	
	(M'Bidgee)	20	5	3	1.5	4	1.5	2	
		21	5	1	4	3	2	2	
	421051	22	3.5	1.5	1.5	5	3.5		
	(Bogan)	23	5	3	4	1.5	1.5		
		24	5	2	3.5	1	3.5		
	418025	25	4.5	1.5	3	1.5	4.5		
	(Gwydir)	26	5	1	2	4	3		
		27	5	3	3	1	3		
	Total		43	18	26.5	23	24.5		
	Av. Rank		4.8	2.0	2.9	2.6	2.7		
Overall Av. Rank			4.7	2.6	2.8	2.1	2.8		

large number of series of maximum monthly peaks using a suitable probability distribution, then carrying out the procedure illustrated in Figure C.2 and described in Section C.2. The distribution used was the three parameter log Normal distribution. This was chosen because it is well known and relatively easy to use. The input values used to define the distribution were the three moments given by the mean, standard deviation and skew of the maximum monthly flows, in real space. The actual parameters of the log Normal distribution were calculated from these moments of the maximum monthly flows by means of the relationships given by Moshman (1953), following the procedure described by Doran (1975). A study of a large number of observed station records was used to establish suitable value ranges for the moments of the maximum monthly flows, as below:-

Mean	(μ);	2 -100 m^3/s
Standard Deviation	(δ);	μ -3μ m^3/s
Skew Coefficient	(γ)	3 -6

To generate each series, statistical parameters were selected using a uniformly distributed random number generator in the appropriate ranges above. These distribution parameters were then used to generate from a three parameter log Normal Population an N year series of maximum monthly peaks, which represented the original series. The simulation was then designed to allow for variations in the following aspects:-

(a) Percentage of missing data or number of gaps made
(b) Record length
(c) Seasonality
& (d) Comparison weighting

Aspects (c) and (d) are simple two-way choices in that a particular series can be generated as seasonal or non-seasonal, and the comparison between the resulting frequency curves can be carried out by either giving higher weighting to differences at higher return periods or by giving an equal weighting to differences for all return periods.

The population statistics used to generate the original series were not used to calculate the 'true' flood frequency curve. This was done by calculating the mean, standard deviation and skew from the generated series of values and using these to calculate the flood frequency curve. Similarly, flood frequency curves resulting from application of the methods for treating missing data in the gapped series were calculated using the particular statistics for the sample.

C.6 RESULTS OF INVESTIGATION USING GENERATED DATA FOR CASE 2

The same methods of treating missing data were used in the generated data trials as were used with the observed data, except that the nearby station Method F was not used as relations of floods of a nearby station were not available. The original (generated) series was again taken as representative of a nearby station record in that a flood event was considered to have occurred in a gap if the original series value there exceeded the 3Nth ranked flood of the original series. This procedure was identical with that used for cases 1 and 2 with observed data.

A summary of the results is given in Table C2 for record lengths of 10,20 and 50 years with percentages of missing data ranging from 10 to 30%.

TABLE C2. Results of t Test Values for Generated Data for Case 2.

RECORD LENGTH	NUMBER OF CYCLES	% MISSING	SEASONAL	WEIGHTED	't' statistic values for differences of method shown to best (*)								't' values for ARR method A compared with next worst
					A	B	C	D	E	DI	EI	G	
10 years	500	10	-	-	28	14	7.6	7.9	7.4	8.3	7.4	*	17
		10	-	-	21	17	7.4	6.4	6.0	10	9.2	*	9.1 (highest 2N floods only)
		10	-	yes	29	15	7.8	8.3	7.7	8.2	7.6	*	19
		10	yes	yes	29	14	9.8	9.1	8.1	8.0	5.8	*	19
		20	-	-	25	2.2	6.2	7.4	5.0	2.8	4.1	*	19
		20	-	yes	27	4.2	7.9	9.1	6.5	4.2	5.6	*	20
		20	yes	yes	24	2.6	7.2	6.3	4.7	2.4	3.2	*	19
		30	-	-	25	*	6.8	5.2	5.8	2.1	2.5	4.4	21
		30	yes	-	21	*	6.4	3.4	3.8	1.4	3.4	5.4	17
20 years	200	10	-	-	16	2.9	3.8	4.3	4.0	3.4	3.1	*	12
		10	-	yes	18	4.4	4.5	5.1	4.8	4.0	3.8	*	13
		25	-	-	16	*	4.2	3.6	1.2	0.89	1.7	3.9	11
		25	-	yes	16	0.84	3.9	3.2	1.1	*	1.5	1.1	12
50 years	100	10	-	-	12	*	2.3	1.1	2.3	1.1	1.4	0.92	8.8
		10	yes	-	9.0	*	2.7	2.0	3.5	0.4	2.3	2.8	5.9

In this table record length (N) refers to the length of record generated to provide the original series used in the particular run. 'Number of cycles' refers to the number of series generated and 'percentage missing' is the ratio of number of gaps made in the original series to the length of the series, expressed as a percentage. Where seasonality was included, the generation of the original series involved the use of three sets of statistical parameters and hence three sets of data generation, one for each of three blocks of four successive months (e.g. November to February, March to June and July to October).

The additional methods used, D1 and E1, represent variations from methods D and E respectively in that replacement was made from the ranking floods in the sample throughout the whole year, rather than from only those in the particular month in which the gap occurred.

A numerical measure was devised to indicate the difference between the frequency curve estimated by each treatment method and the frequency curve for the original generated series before the gaps were created, and the values reported in Table C2 are the results of statistical tests on these numerical measures. For a given treatment method i, a sum of differences between the frequency curves was calculated as

$$\mathrm{SUM}(i) = \sum_{Y=2}^{\mathrm{YMAX}} \mathrm{DIFF}(Y) \times W(Y) \qquad \text{(C.1)}$$

where DIFF(Y) = the difference in the discharge values of the frequency curves for the estimated and original series at each value of Y,

Y = return periods in the series 2,5,10,20,50 and 100 years up to a maximum return period of YMAX, whose value depended on the length of each generated record,

and W(Y) = a weighting factor.

Two options were used for W(Y). In the first, equal weightings were given to each return period. For the second, higher weightings were used for higher return periods to reflect their greater practical importance, and this case is indicated by the 'weighted' column in Table C2. However, it should be noted that even with equal weightings, larger differences of the discharges are likely at the higher return periods of practical importance. For each generated series, a total sum of the differences for the eight methods for treating the missing data was then calculated as

$$\mathrm{TOTAL} = \sum_{i=1}^{8} \mathrm{SUM}(i) \qquad \text{(C.2)}$$

The relative difference of the frequency curve estimated by each treatment method (i) from the frequency curve of the original generated series was then given by

$$\mathrm{RELDIFF}(i) = \frac{\mathrm{SUM}(i)}{\mathrm{TOTAL}} \qquad \text{(C.3)}$$

The mean and standard deviation of this value were then calculated for each treatment method over each set of cycles of runs indicated in Table C2. An asterisk is shown in this table for the method that gave the best overall performance as indicated by the lowest mean value of RELDIFF. A t test was used to indicate whether the best method in each case was significantly better than the other methods, and whether the differences in performance are real. The t values were calculated from the formula (Hays 1970)

$$t = \frac{\bar{X}_1 - \bar{X}_2}{\sqrt{\left(\frac{N_1 S_1^2 + N_2 S_2^2}{N_1 + N_2 - 2}\right)\left(\frac{N_1 + N_2}{N_1 N_2}\right)}} \tag{C.4}$$

which for $N_1 = N_2 = N$ reduces to

$$t = (\bar{X}_1 - \bar{X}_2)\sqrt{\frac{N - 1}{S_1^2 + S_2^2}} \tag{C.5}$$

where $\bar{X}_1$ and S_1 are respectively the mean and standard deviation of relative differences RELDIFF(i) from the true flood frequency curve generated for the run, calculated for the particular method (i) considered;

$\bar{X}_2$ and S_2 are similarly the mean and standard deviation calculated for the method giving the best performance;

and N represents the number of series generated (number of cycles)

The values of t are shown in Table C2. For t values greater than 3.09 the probability that differences are due to chance is very small (smaller than 0.001. A table of t versus probability is given below in Table C3.

Table C3. One Tailed t Distribution

t value (one tailed)	Probability that differences are due to chance
.253	0.40
.674	0.25
1.282	0.10
1.645	0.05
1.960	0.025
2.326	0.01
2.576	0.005
3.090	0.001

The column in Table C2 headed 't values for ARR method A compared with next worst' gives the calculated t values resulting from the comparison of the results of the ARR method A with the results of the worst of the other methods for each set of runs.

Almost all of the t values in all columns of Table C2 are significant at the 0.001 probability level. Most are much higher than the value of 3.09 corresponding to this probability.

The effects of seasonality and comparison weighting changes on the results were negligible. The results in Table C2 support the conclusion from the investigation in Section C.4 that the ARR method eliminating the entire year with missing data is significantly the worst policy in all combinations of record length, percentage of missing data, seasonality and comparison weighting.

The method of ignoring missing data (G) is shown to be significantly better than any of the other methods tested for record lengths up to 20 years and percentages of missing data in the practical area of interest (Figure C.1). For longer record lengths the proportion method (B) or subtracting a proportional amount from the year for each missing peak was significantly better than the other methods (except the normal distribution replacement method D).

C.7 RECOMMENDED POLICY FOR CASE 2

Case 2 is where a gauging station is not completely isolated but the nearest concurrent station record only reflects whether or not a peak has occurred, and not its magnitude. This case also applies to a daily read record where the daily value shows that a peak has occurred, but its magnitude is unknown. For these situations, the two investigations using observed and generated data indicate that the best policy for treating missing data is:-

(i) For record lengths smaller than 20 years ignore any missing data
(ii) For record lengths larger than 20 years use the proportion method (B) of subtracting a proportional amount from the year for each missing peak.

C.8 RESULTS OF INVESTIGATION USING GENERATED DATA FOR CASE 3

For an isolated station no knowledge of occurrence or magnitude of events in missing periods could be assumed. In this situation ARR makes no recommendations, hence no method from this source could be tested. However, from examination of the case 2 results the obvious policy to recommend is to ignore the missing data. As this proved to be the best policy for case 2 where some knowledge of events in missing periods was available. It would necessarily be the best policy for case 3 where no information on events in missing periods is available.

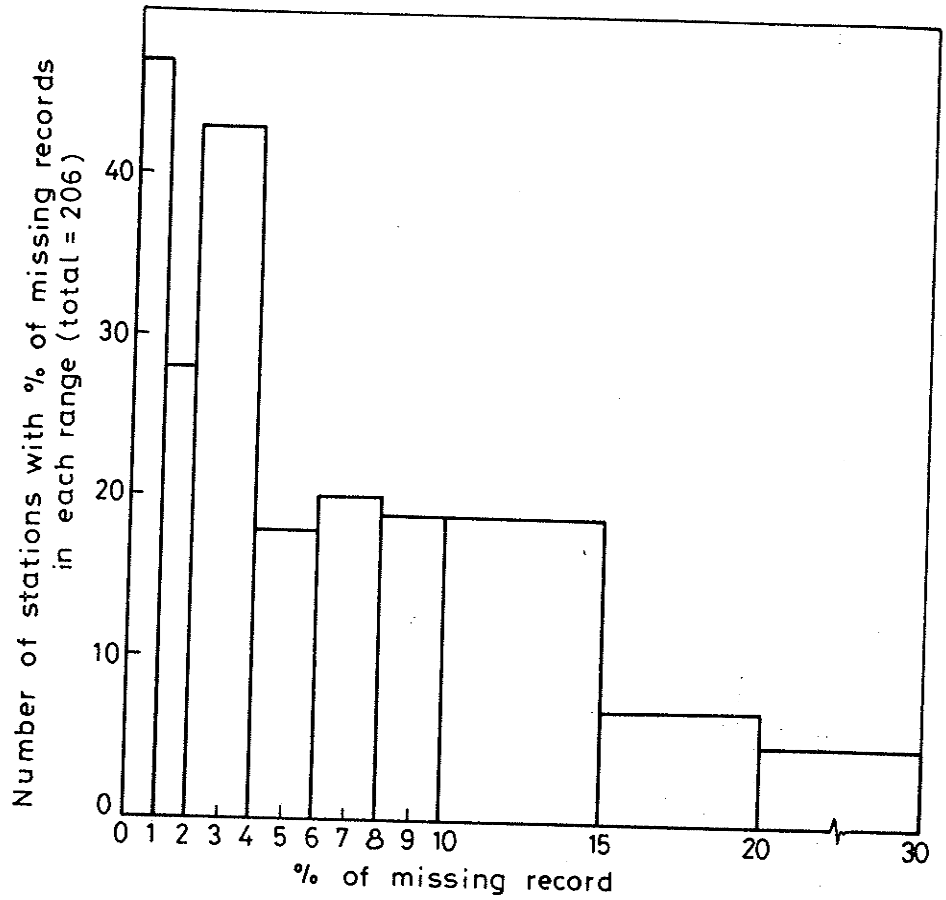

Figure C.1 Frequency distribution of percentages of missing data for 206 streamgauging stations with continuous recorders.

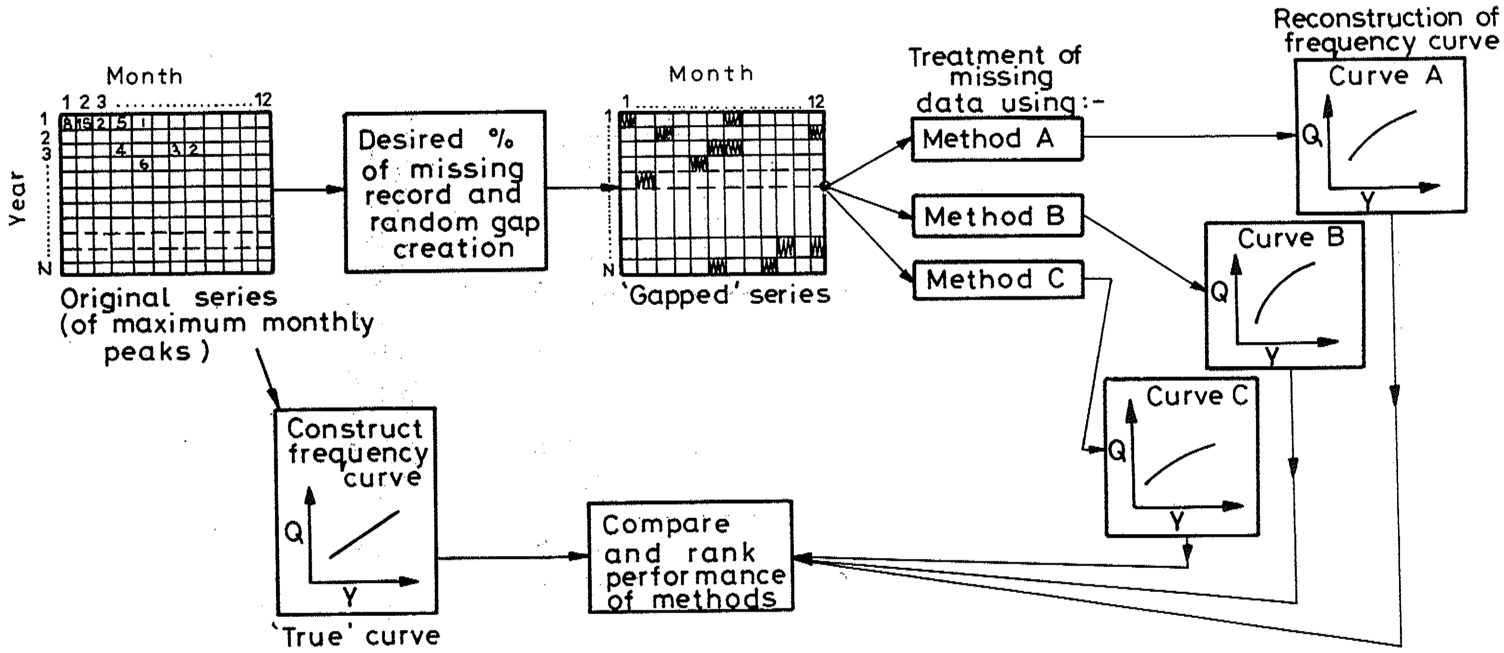

Figure C.2 Approach to the investigation of the treatment of missing data, using observed data.

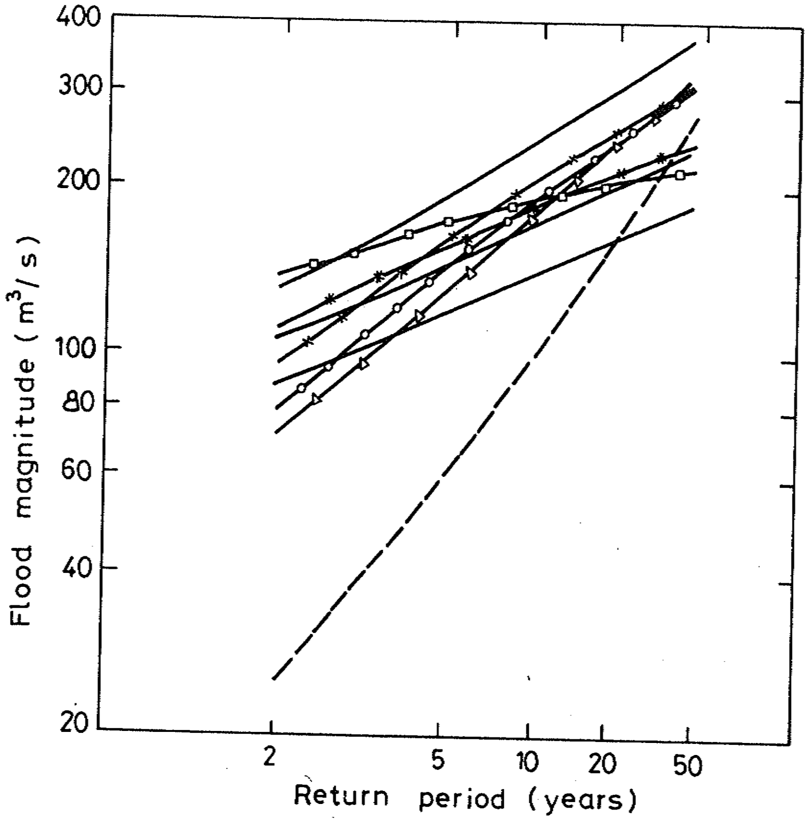

Figure C.3 Typical flood frequency curves estimated for observed data with gaps made in the record (for station 218006 - trial 14).

KEY:-

═══ True flood frequency curve from original series, with 5% and 95% confidence intervals plotted either side.

The other lines represent the flood frequency curves resulting from the various methods for treating missing data.

— — — Method A - ARR

—o—o— Method B - Proportion

—x—x— Method C - Uniform distribution

— — — Method D - Normal distribution

— — — Method E - Log-normal distribution

—*—*— Method F - Nearby station.

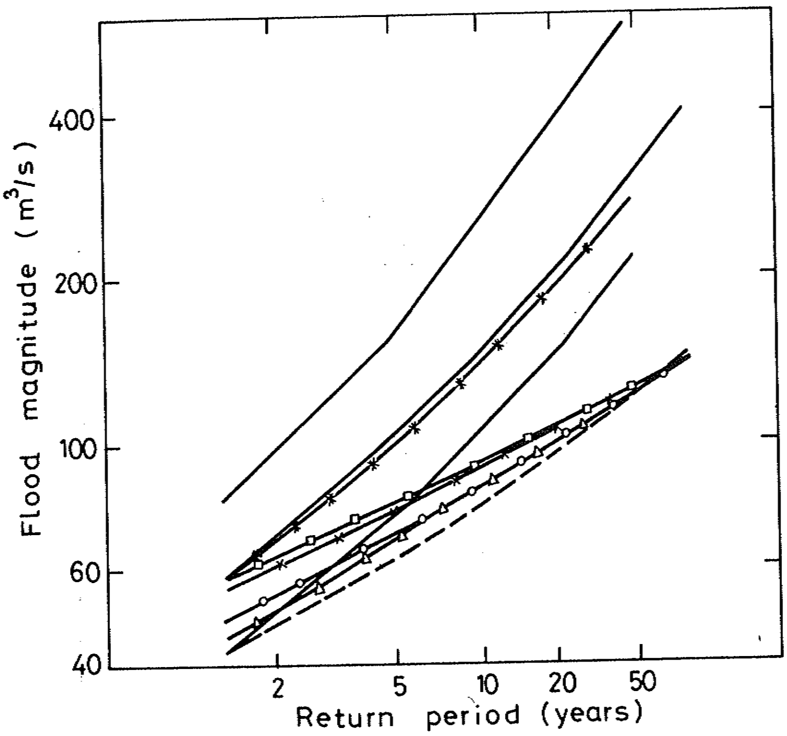

Figure C.4 Extreme case of frequency curves estimated for observed data with gaps made in the record (for station 204043 - trial 7).

KEY:-

——— True flood frequency curve from original series, with 5% and 95% confidence intervals plotted either side.

The other lines represent the flood frequency curves resulting from the various methods for treating missing data.

— — — Method A - ARR
—o—o— Method B - Proportion
—x—x— Method C - Uniform distribution
— — — Method D - Normal distribution
— — — Method E - Log-normal distribution
—*—*— Method F - Nearby station

APPENDIX D. RELIABILITY INDEX OF THE FLOOD FREQUENCY CURVE

In Section 2.2.3 the necessity for some form of reliability index for the flood frequency curves was described. Causes of errors were discussed and indicators that could be used to gauge their likely importance were presented. Any procedure used to calculate a comparative reliability index for the flood frequency curve for each station will involve considerable subjectivity due not only to the uncertainty of the size and effect of any particular cause of error, but also to the uncertainty of the effects of causes relative to each other.

For convenience, numerical values were assigned to various aspects of the unreliability of the flood frequency curve rather than to its reliability. These aspects were considered under two groupings. The first related to errors and uncertainties in the actual flood values that constitute the partial series of floods on a given catchment. The second grouping concerns the problem of whether the sample of values is representative of the true population of floods.

Aspects of unreliability and unsuitability identified as affecting the partial series of flood magnitudes are listed in Table D1. Also presented are their effect directions, indicators adopted for the size of their effects, relative ranking of the effects of the worst case of each factor, and assigned unreliability values for each worst case. The effect directions shown refer to the direction in which the partial series of flood peaks might be changed, compared with the partial series of flood peaks from a small rural catchment with no measurement or recording errors. These 'directions' are not considered further in this appendix, but are in Appendix K. The effect size indicators are self explanatory.

In Table D1 the 'worst case' refers to the plausible worst effect which the aspect can have on the partial series of flood peaks. The ranking assigned to the worst case of each aspect relative to the others' worst cases was obtained by subjective consideration of whether the aspect's effect was general or specific, and on the relative rarity of its occurrence. Unreliability values assigned to each aspect's worst case consequently followed the same order as in the ranking, with values beginning at 0.5 and ranging in steps of 0.5 up to 2.5. The aspect assigned the highest ranking and hence highest unreliability value was the reliability of the rating curve and its need for extension measured by the magnitude of the highest gauged discharge relative to the 2-year flood magnitude. Figure D.1 shows the frequency distribution of this measure.

Degrees of effect of each aspect were then described in relation to class ranges of the effect size indicator, as shown in Tables D2 to D7. For the missing records aspect (2a), a suitable unreliability value was considered to be equal to four times the percentage of missing record divided by 100. Unreliability value ranges for the forestry (3c) and cultivation (3d) aspects were such that a value of 0.5 was assigned where a large portion of a catchment was devoted to these activities, ranging to 0.1 where only a small portion was so devoted.

A relative reliability index value could then be calculated for the partial series of flood peaks for each station, such that:-

$$REL = (K - \sum_{i=1}^{4} \Delta_i)/K \qquad (D.1)$$

where REL - relative reliability index of partial series

Δ_i - unreliability value for the station, appropriate for aspect i.

K - constant of proportionality (defined later).

Sampling error affects all of the previously discussed aspects. Accordingly, sampling error was considered by means of an overall multiplying factor on 'REL' to obtain a reliability index for the flood frequency curve. Assuming a record length of 50 years to give no sampling error and a constant standard deviation of logarithms of flood peaks for all stations, a multiplying factor to account for sampling error was calculated as:-

$$MF = \sqrt{N/50}$$

where N = record length in years.

The overall form of relationship below could then be used to calculate a relative reliability index for the flood frequency curve:

$$TRR = MF \times REL \qquad \text{(D.3)}$$

where TRR = total relative reliability index for a station's flood frequency curve (a value of 1.0 being perfect)
MF = multiplying factor from equation (D.2)
REL = partial series relative reliability from equation (D.1)

Effectively then,

$$TRR = MF \times \left(\frac{(K - \Sigma\Delta_i)}{K}\right) \qquad \text{(D.4)}$$

A numerical value for K was then required to give a suitable range of values of TRR in the above relation. For example, the choice of a very large K value would make TRR approach MF, and K less than $\Sigma\Delta_i$ would make TRR negative. Of the $\Sigma\Delta_i$ terms obtained for each of 290 stations, the value of 3.0 was rarely exceeded. As a basis for the calculation of a suitable K value the following two cases were assumed to give equal total reliability indices (TRR):-

Case 1	Case 2
$MF = MF_1$	$MF = MF_2 = 2 \times MF_1$
& $\Sigma\Delta_i = 0$	$\Sigma\Delta_i = 3.0$

$\therefore$ $TRR_1 = TRR_2$

$$MF_1 \times REL_1 = MF_2 \times REL_2$$

i.e.

$$1 \times \left(\frac{K-0}{K}\right) = 2 \times \left(\frac{K-3}{K}\right)$$

or $K = 6$

This value of K (equal to 6) was then assumed to represent a reasonable balance between the weight given to the reliability of the partial series flood magnitudes and the weight given to sampling error in the expression for the reliability of the flood frequency curve (equation (D.4)). The total reliability (TRR) values could then be calculated for each station's flood frequency curve.

As the relative reliability indices have a subjective basis, their absolute values have little meaning and they cannot be considered to be of high accuracy. Accordingly, five classes of reliability were defined as listed in Table D8, and these classes were used in practical application. The reliability class for each of the catchments used in the study is listed in Appendix A. The class of each station is also shown on Map 1 of eastern New South Wales at the back of this report, and this was used in drawing the isopleths of design runoff coefficients, greater weight being given to the stations with higher reliabilities. The reliability classes were also used in examining outlier values in Appendix K.

TABLE D 1 Unreliability Aspects and Effects on Partial Series of Floods

Unreliability or Unsuitability Aspect	Partial Series of Flood Magnitudes-Effect Direction: Lower	Random	Higher	Effect Size Indicators	Worst Case of Effect:- Ranking	Unreliability Values Assigned to each worst case
1) Amount of Rating Curve Extension required		✓		$R = \frac{\text{highest gauged discharge}}{\text{2-year flood magnitude}}$	1	2.5
2a) Uncertainty due to missing large events - missing records	✓			% missing record	4	1.5
2b) Uncertainty due to missing large events-daily read records	✓			% daily read and catchment area	2	2.0
3a) Non Rural-Storage(s) on main stream	✓	✓		size of storage(s) and catchment area	4	1.5
3b) Non Rural-small storages on catchment	✓	✓		No. of storages and catchment area	6	1.0
3c) Non Rural-forestry activities		✓	✓	% of catchment under forest	8	0.5
3d) Non Rural-cultivation activities	✓	✓		% of catchment cultivated	8	0.5
3e) Non Rural-urban		✓	✓	% urbanised and catchment area	6	1.0
4) Large catchment area-greater than 250 km^2		✓		catchment area	2	2.0

TABLE D2 Unreliability Values Assigned to Ranges of Rating Curve Extension

Ratio R of highest gauged discharge to 2-year flood Magnitude.	Unreliability Value Assigned to Range.
< 0.04	2.5
0.04 - 0.06	2.5
0.06 - 0.15	2.0
0.15 - 0.25	1.5
0.25 - 0.50	1.0
0.50 - 1.0	0.5
> 1.0	0.2

TABLE D3 Unreliability Values Assigned for Proportion of Daily Read Record

Area Range (km^2)	Unreliability Values for Proportion of Daily Read:-		
	All	½	¼
0 - 50	2.0	1.0	0.5
50 - 150	1.5	0.75	0.25
> 150	1.0	0.5	0.10

TABLE D4 Unreliability Values Assigned for Effect of Storage(s) on Main Stream

Area Range (km^2)	Unreliability Values Assigned to Relative Storage Sizes:-		
	Large	Medium	Small
0 - 50	1.5	1.0	0.5
50 - 150	1.0	0.5	0.2
> 150	0.3	0.2	0

TABLE D5 Unreliability Values Assigned for Effect of Small Storages on Catchment

Area Range (km^2)	Unreliability Values Relative to Number of Small Storages:-		
	Large	Medium	Small
0 - 50	1.0	0.5	0.3
50 - 150	0.75	0.3	0.1
> 150	0.5	0.2	0

TABLE D6 Unreliability Values Assigned for Effect of Urbanization

Area Range (km^2)	Unreliability Values for Area of Catchment Urbanized:-		
	Large	Medium	Small
0 - 50	1.5	0.5	0.5
50 - 150	0.75	0.4	0.2
> 150	0.5	0.3	0.1

TABLE D7 Unreliability Values Assigned for Catchments with Areas Larger than 250 km^2

Area Range (km^2)	Unreliability Value
250 - 500	0.5
500 - 1000	1.0
> 1000	2.0

TABLE D8 Reliability Classes of Flood Frequency Curves

Class Number	Range of TRR (Equation D4)	Remarks
1	≥ 0.6	Most Reliable
2	0.5 - 0.6	-
3	0.4 - 0.5	-
4	0.3 - 0.4	-
5	< 0.3	Least Reliable

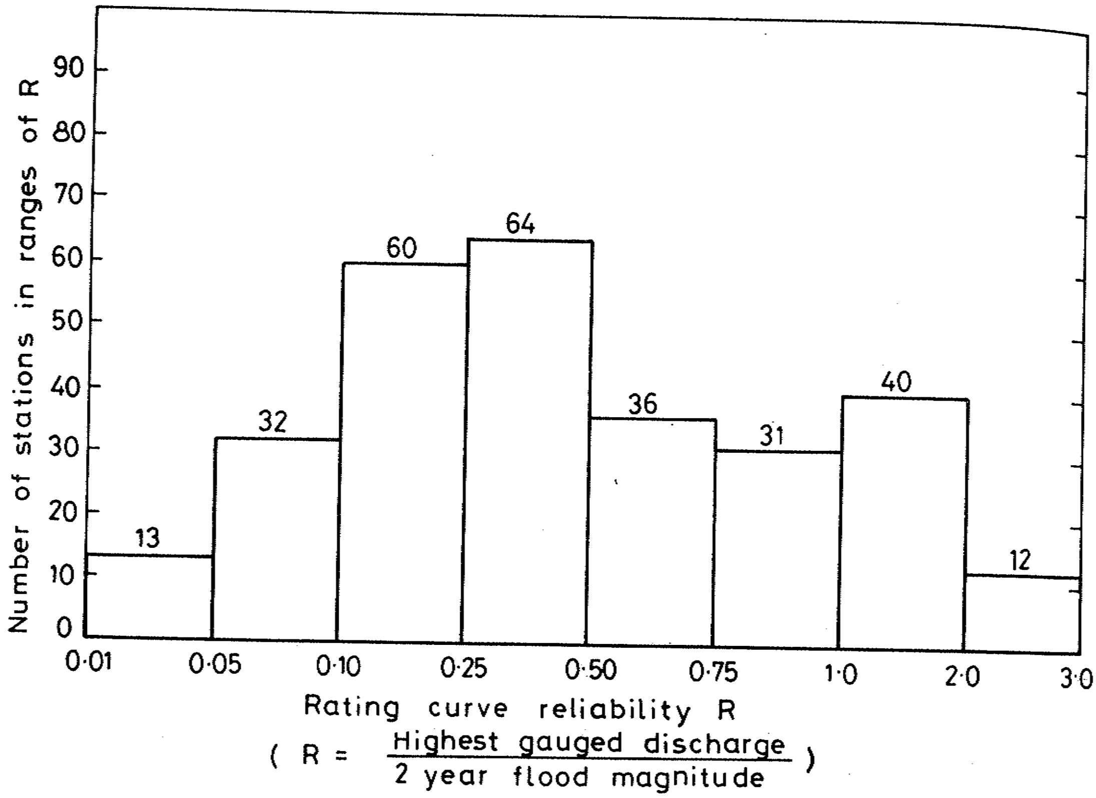

Figure D.1 Frequency distribution of reliabilities of gauging station rating curves.

APPENDIX E. SENSITIVITY OF RAINFALL INTENSITY TO DESIGN DURATION

Traditional formulae for design duration of rainfall have been tested against an observed measure of characteristic catchment response time by French et al. (1974), and large differences were found. The purpose of the investigation described in this Appendix was to demonstrate the magnitude of differences in calculated design rainfall intensities caused by the choice of the formula for design duration.

In order to achieve this purpose it was necessary to examine the methods for calculation of rainfall intensity for various durations and return periods. Three calculation procedures are used in ARR for the generalised rainfall data, one for each of the duration ranges $6 \leq t \leq 60$ mins, $1 \leq t \leq 12$ hours, and $12 < t \leq 72$ hours.

The use of these procedures to calculate rainfall intensities for a duration equal to 't' hours, of return period 'Y' years, employs the relations below:-

(a) $6 \leq t_{min} \leq 60$ minutes

$$I(t_{min},Y) = I(12,Y)x(AFACT+1)x(0.309+\frac{49.586}{t_{min}+11.767}) \qquad (E.1)$$

(b) $1 \leq t \leq 12$ hours

$$I(t,Y) = I(12,Y)\left[AFACT\ (\frac{1.798}{t+.576} - .143)+1\right] \qquad (E.2)$$

(c) $12 < t \leq 72$ hours

$$I(t,Y) = I(12,Y) - f(t)x(I(12,Y)-I(72,Y)) \qquad (E.3)$$

and $$f(t) = \frac{180.3 \log (t-5.8)-142.8}{161.9} \qquad (E.4)$$

for 't' in the range of interest for small catchments. In these equations,

$I(t,Y)$ = average rainfall intensity (mm/h) for duration t hours or t_{min} minutes and of return period Y years.

$I(12,Y)$= as above with t=12 hours

$I(72,Y)$= as above with t=72 hours

AFACT = zone factor from Table 2.3 of ARR, dependent on location. This is also shown in Figure 5.2 of this report.

Equation (E.4) is an empirical expression of the relationship over the section of interest of Figure 2.47 of ARR.

Thus once the site of interest has been located on the rainfall maps and the return period chosen, the only parameter affecting the calculation of rainfall intensity is the rainfall duration chosen for the catchment. Consider the case where one design duration formula recommends a value of t_1 for a catchment while another recommends t_2. The difference this causes in calculated rainfall intensities (and hence derived runoff coefficients) can be seen by referring to Figures E.1 and E.2.

In general form the curves shown on these figures can be expressed as:-

$$I_2 = I_1 \quad x \left[\frac{t_1}{t_2}\right]^n \qquad (E.5)$$

where I_2 = rainfall intensity (mm/hr) calculated for any particular return period for duration t_2

I_1 = as above for t_1

n = an exponent varying between 0.5 and 0.7 depending on rainfall zone and the value of t_1.

Reference to Figures E.1 and E.2 shows that, for example, a 50% over or under estimate of appropriate design duration can cause approximately a 25% underestimate or 50% overestimate of rainfall intensity respectively, This in turn would cause more than 30% overestimate or underestimate of runoff coefficient respectively, as the runoff coefficient derived by the approach described in Section 1.4.1 is a reciprocal function of rainfall intensity.

When considering very small catchments with short design durations of say 30 minutes, large percentage over and underestimates of duration are likely. The reason for this is that a 50% overestimate of design duration would be caused by an error of only 15 minutes or similar short time. The effects of errors in design duration on runoff coefficients are thus likely to be larger for smaller catchments.

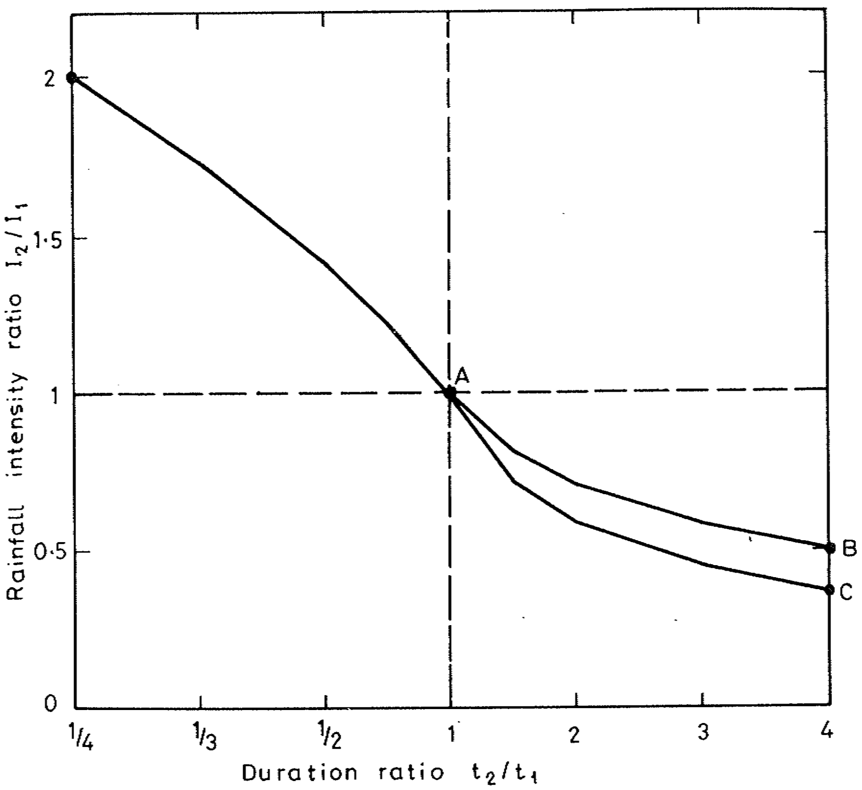

Figure E.1 Sensitivity of rainfall intensity to duration, for reference durations t_1, of 6-60 minutes.

AB and AC are the limits between which lie a family of curves depending on zone factor AFACT and duration t_1.

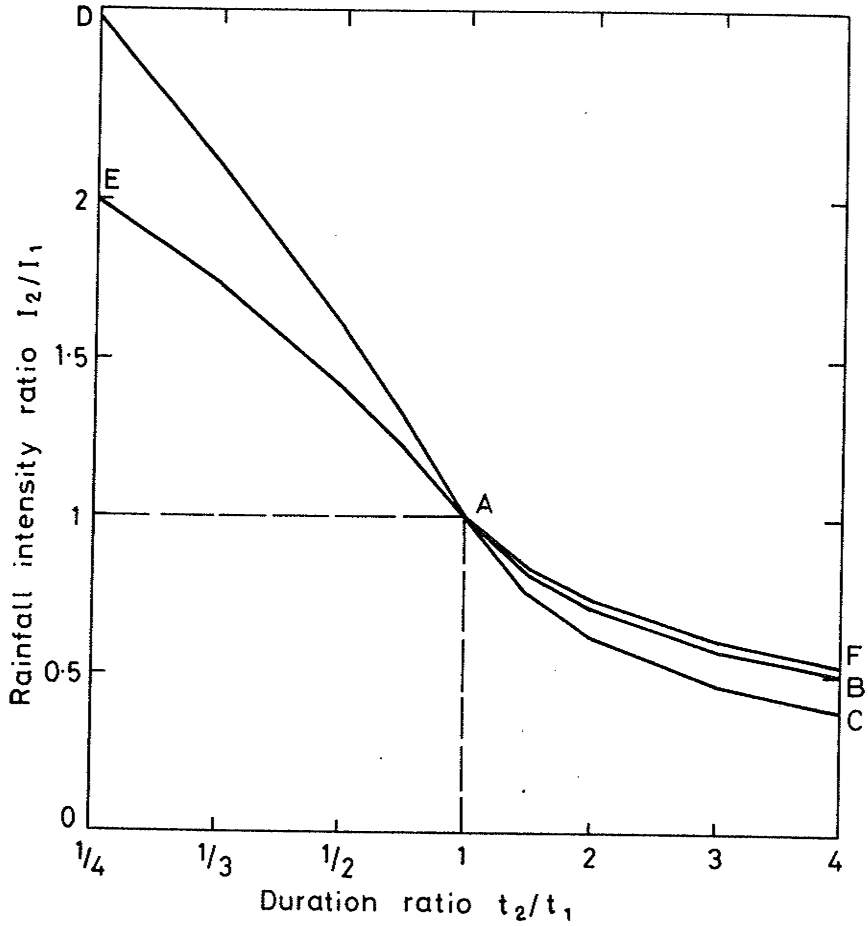

Figure E.2 Sensitivity of rainfall intensity to duration, for reference durations t_1 of 1-12 hours.

DAC and EAF are the limits between which lie a family of curves depending on zone factor AFACT and duration t_1; e.g. EAF is the limiting curve from zone A, while DAB is the limiting curve from zone D and AF is the limiting curve for t_2>12 hours.

APPENDIX F. CHARACTERISTIC MEASURES OF CATCHMENT RESPONSE TIME FOR DESIGN RAINFALL DURATION

As available formulae for time of concentration have been shown to give considerable scatter of values and errors, this investigation was carried out to develop a formula or formulae based on observed data. Values of time of concentration or response time were obtained for ninety-six catchments, as noted in Section 4.4.1. Data for a large number of catchments were available in three publications. However, some of the catchments were unsuitable as a result of features such as swamps or large storages, and some of the response times listed were obviously much longer than indicated by the remainder of the data. This problem of anomolously long times of concentration was also encountered by French et al. (1974), especially for the Snowy Mountains and Southwest Slopes regions. The values from suitable catchments that were adopted from the published studies were twenty-three minimum times of hydrograph rise from French et al. (1974), twenty-two values of the baselength C of the time area diagram of the Clark unit hydrograph model from ARR, and eight values of the Clark C from Baron et al. (1980). The remaining fortythree values were minimum times of rise extracted in this study. The response times and relevant data for the ninety-six catchments are presented in Table F1.

Measures of various catchment characteristics were extracted for use in the derivation of design formulae for catchment response time or time of concentration. The characteristics extracted for each catchment were:

a) Catchment area A
b) Main stream length L
c) Main stream average slope S_a
d) Main stream equal area slope S_e
e) Catchment compactness coefficient (shape factor)
f) Median annual rainfall
g) Soil type
and
h) Vegetation cover

Values of L and S were adjusted to correspond to the standard map scale of 1:25 000 (Section 3.6). General classifications only were obtained for soil type and vegetation cover as time did not permit more detail. To find some appropriate quantitative measure for soil type, infiltration indices were sought. These were only available for a very few soil types with detailed classifications. Soil type however was not expected to play a vital part in minimum rise time relations, and would be expected to be reflected in the values of the runoff coefficient rather than in the minimum rise times. The same would apply to vegetation cover and median annual rainfall. Also, in the event of a large flood with close to the minimum rise time, the soil would be likely to be saturated and hence the influence of soil type would be negligible.

An approximate hydraulic roughness index was assigned to various vegetation cover types ranging from roughest for grass cover to least rough for forest cover.

The ninety-six catchments were then separated into the three gross geographical regions of coastal (thirty-five catchments), Tablelands (twenty-three), and Snowy Mountains (thirty-eight). As some of the catchment characteristics were highly correlated with each other (see Section 3.7), it is not valid to consider each as an independent variable in regression studies. Various combinations of area, length and slope were used as single variables. The form of the physical relationships and the range and distribution of values of some variables indicated that multiplicative power function relations would be better than summation type relations. Regressions were thus derived using the logarithms of the values.

Initially the catchment characteristics of area and stream length were used in correlation studies in each region. These consisted of simple linear regressions of the logarithms of the response time values on the logarithms of the values of the catchment characteristics. The results are presented in Table F2. From this table and the plotted relationships in Figures F1 and F2, it was seen that there was very little difference between the relationships derived for each region and for all of the data. This indicated that the effect of catchment size on minimum rise time was largely independent of region. As catchment size is the major variable affecting minimum rise times, regional differences were considered to be negligible, with any residual regional effects to be accounted for by the characteristics (slope, vegetation cover and median annual rainfall) not yet considered.

Using the pooled data for all ninety-six catchments, the variations in response times not accounted for by the simple regressions on catchment area and slope were then investigated. Variations in terms of ratios rather than absolute values were appropriate as the regressions were calculated using logarithmic values. From the parameters of the simple regressions given in Table F2, these variations or deviations were calculated as

$$v_{ai} = t_i/0.76\ A_i^{0.38} \quad \text{(F.1)}$$

and

$$v_{\ell i} = t_i/0.54\ L_i^{0.66} \quad \text{(F.2)}$$

where v_{ai}, $v_{\ell i}$ = variation or deviation in response time not accounted for by the area or length relation respectively, for catchment i (h)

t_i = characteristic response time for catchment i (h)
A_i = area of catchment i (km^2)
L_i = mainstream length for catchment i (km)

The presence of any relation between v_{ai} and $v_{\ell i}$ and each of the characteristics of shape factor, median annual rainfall, soil type and vegetation cover roughness index was then investigated. The results are not presented here, but indicated that no relations existed. It was concluded that any attempt to improve the prediction relations for response time by the inclusion of any of these variables would not be warranted.

Although stream slope was not related to deviations of response time based on the area relation, it did show a slight correlation with those based on the length relation (equation F.2). A large set of regressions was then calculated of logarithms of characteristic response times on logarithmic values of the variable $(L/S_e^m)^n$ over a grid of values of m and n. This approach was used as L and S_e cannot be considered as independent variables. The combination of m and n yielding the highest correlation coefficient and lowest standard error of estimate of the logarithms was adopted as the best relation, yielding

$$t = 1.69 \left(\frac{L}{\sqrt{S_e}}\right)^{0.50} \quad \text{(F.3)}$$

where t = characteristic response time or time of concentration (hours)
L = mainstream length (km)
S_e = equal area slope (m/km)

This relationship had a correlation coefficient for the logarithms of 0.92 with a standard error of estimate slightly larger than that shown in Table F2 for the area relationship using data for all ninety-six stations.

As part of the above procedure the relationship given below with $(\frac{L}{S_e})$ as the independent variable was obtained:

$$t = 3.17 \left(\frac{L}{S_e}\right)^{0.38} \qquad (F.4)$$

This gave a correlation coefficient for the logarithms of 0.81 with a standard error of estimate 25% larger than that found for (F.3). Equation (F.4) yields almost identical time values with those given by the relation of Baron et al. (1980) for values of C in the Clark model for synthesis of unit hydrographs:

$$C = 3.01 \left(\frac{L}{S_e}\right)^{0.40} \qquad (F.5)$$

Both relations are shown in Figure F.3. Values of the Clark model C given by the relationship of Cordery and Webb (1974) are also only a few percent different. Although some of the data were used in deriving each of these three relationships, they are partly independent. The similarity of the relations provides considerable mutual confirmation, and reflects the fact that Clark C values are the same as minimum time of hydrograph rise as long as the rainfall excess is short and the time area diagram has its maximum ordinates at or near its end. To prevent proliferation of formulae giving virtually the same results, equation (F.5) of Baron et al. (1980) was adopted here rather than the derived equation (F.4).

From the work described above, three formulae were selected for further testing as a means of estimating the design rainfall duration. These formulae were:

$$t = 0.76\, A^{0.38} \qquad \text{(from Table F2)}$$

$$t = 1.69 \left(\frac{L}{\sqrt{S_e}}\right)^{0.50} \qquad (F.3)$$

$$t = 3.01 \left(\frac{L}{S_e}\right)^{0.40} \qquad (F.5)$$

The final testing of these and several other means of evaluating design rainfall duration was based primarily on the consistency of runoff coefficients derived using these procedures, and is described in Chapter 7.

TABLE F.1 List of Catchments Used in Response Time Studies

National Gauging Station Number	Responsible Authority	Characteristic Catchment Response Time (hrs.)	Source	Physical Catchment Characteristics: Catchment Area(km^2)	Stream Length (km)	Stream Slope (m/km) S_e	S_a
116014	IWSC (Qld)	9.0	ARR	585	65.8	4.64	7.53
119005	"	12.0	"	1092	72.8	5.27	7.54
122003	"	5.0	"	260	34.8	6.88	18.7
136107	"	11.0	"	375	77.0	1.38	7.66
136202	"	15.0	"	640	90.2	1.90	4.24
143103	"	4.0	"	225	27.9	16.6	38.0
143107	"	15.0	"	620	71.8	4.77	9.34
143108	"	15.0	"	920	92.1	3.19	12.9
145102	"	8.0	"	545	71.4	8.52	11.5
204020	WRC (NSW)	3.5	BPC	251	49.1	15.2	17.0
204021	"	4.6	"	70.0	25.5	11.0	19.5
204026	"	4.3	"	80.0	21.2	4.15	15.2
210017	"	4.0	FPL	98.0	20.0	28.1	51.9
210019	"	5.5	"	104	21.9	47.9	50.7
210042	"	6.0	"	205	41.2	5.62	13.2
210045	"	2.0	"	41.0	12.3	8.35	17.8
210046	"	5.0	"	153	22.2	7.06	13.2
Scone SCS	SCS (NSW)	0.40	"	0.18	0.72	117	219
212008	WRC (NSW)	4.0	"	199	23.8	6.84	14.7
212291	MWSDB (NSW)	2.6	BPC	650	58.1	9.82	17.6
212301	UNSW	0.25	NEW	0.06	0.33	201	200
212302	"	0.15	"	0.13	0.51	98.2	86.3
212303	"	0.15	"	0.06	0.43	128	128
212304	"	0.40	"	0.08	0.42	63	81
212305	"	0.31	BPC	0.06	0.37	66	66E
212306	"	0.20	NEW	0.11	0.40	128	110
212307	"	0.25	"	0.04	0.33	119	127
212309	"	0.44	BPC	0.23	0.71	38	38E
212320	"	6.0	FPL	89.6	20.3	1.88	4.58
212333	"	0.50	"	0.70	1.36	24.2	39.3
212340	"	4.0	"	24.9	9.25	5.64	10.4
213200	MWSDB	4.5	"	74.5	18.3	11.2	14.5
214310	UNSW	1.1	NEW	2.54	2.92	17.9	40.8
214314	"	1.0	"	8.03	6.10	29.0	40.6
214320	"	0.6	"	0.93	1.86	130	132
214324	"	1.0	"	22.5	11.6	14.0	25.2
214330	"	0.5	FPL	0.39	1.20	64.1	65.8
214333	"	1.25	NEW	2.64	2.10	47.0	65.0
214334	"	1.5	FPL	5.52	4.80	45.4	55.0
214340	"	2.5	"	40.2	17.5	8.4	18.1

215004	WRC	7.0	FPL	166	30.1	4.00	7.94
222400	SRWSC (VIC)	4.0	NEW	28.5	12.9	29.7	39.4
222401	"	1.5	"	20.7	6.41	29.6	31.2
222505	SMC	6.0	"	40.4	16.6	22.0	27.0
222508	"	4.0	"	37.6	13.3	29.4	36.2
222509	"	4.0	"	21.8	8.64	26.9	38.2
222516	"	4.0	"	12.2	7.73	30.6	36.2
222517	"	1.5	"	4.66	4.37	54.5	91.5
222519	"	3.0	"	28.2	12.5	64.6	63.8
222520	"	1.5	"	5.7	4.2	115	119
222524	"	4.5	"	20.7	10.5	32.4	34.8
304058	HEC (Tas)	4.0	ARR	216	25.2	4.88	4.88E
304108	"	2.0	"	19.4	7.01	47.5	41.0
309002	"	8.0	"	541	60.8	4.2	8.0
309001	"	10.0	"	449	52.8	4.2	8.0
309200	"	1.4	"	112	27.3	18.6	27.9
308003	"	6.0	"	757	62.3	8.05	12.0
310006	"	9.0	"	523	54.5	7.06	10.6
310007	"	6.0	"	756	76.5	5.44	10E
315002	"	10.0	"	713	82.5	5.05	10E
315003	"	3.0	"	158	29.1	17.0	25.6
315006	"	10.0	"	316	44.3	10.0	15.0
316003	"	8.0	"	78.0	15.6	7.92	11.9
318039	"	2.0	"	9.3	5.63	11.6	53E
401007	WRC	9.0	FPL	134	28.6	12.7	21.4
401009	"	3.0	"	220	36.3	19.2	23.0
401508	SMC	1.5	NEW	5.0	4.73	38.2	50.7
401512	"	4.0	"	24.9	15.5	17.8	24.1
401516	"	1.0	"	24.6	12.1	48.0	74.6
401519	"	1.5	"	3.68	3.52	36.2	73.9
401520	"	3.0	"	16.3	7.34	21.7	32.7
401521	SMC	4.0	NEW	27.5	15.4	27.9	31.5
401526	"	3.0	"	43.0	19.7	50.5	70.3
401527	"	1.75	"	18.6	10.9	67.7	91.1
401528	"	4.0	"	27.7	10.2	18.1	29.0
401536	"	1.75	"	8.29	6.5	63.8	51.4
401537	"	1.2	"	12.2	11.7	21.9	42.7
401542	"	5.3	"	55.4	18.2	13.4	25.2
401543	"	2.5	"	28.7	7.13	45.4	94.8
410009	WRC	4.0	FPL	134	32.5	29.9	35.2
410034	"	4.0	"	95.5	20.6	32.7	32.9
410059	"	2.5	"	233	49.3	14.5	21.1
410067	"	8.0	"	220	35.8	11.7	13.6
410508	SMC	1.6	NEW	9.07	7.53	47.7	43.8
410511	"	1.5	"	20.2	6.78	39.3	53.4
410513	"	2.5	"	19.4	6.93	44.2	57.7
410516	"	3.5	"	33.4	7.95	39.5	32.7
410527	"	3.0	"	12.7	6.35	63.6	109
410536	"	1.0	"	0.34	1.08	120E	120E
410537	"	0.75	"	0.44	0.94	120E	120E

410538	SMC	0.50	NEW	0.54	0.98	120E	120E
410539	"	0.40	"	0.28	0.87	133E	133E
421033	WRC	5.0	FPL	34.0	7.35	22.8	36.1
421034	"	5.0	"	15.5	6.20	21.6	28.6
421038	"	8.3	BPC	544	63.9	2.65	8.61
421079	"	16.4	"	1070	123	2.07	5.88

Note:- Key to 'Responsible Authority' abbreviations is as in Stream Gauging Information, 1974.

Key:- to Source

FPL - French, et al., 1974 (Minimim t_p)
BPC - Baron et al., 1980 (Clark C)
ARR - AR&R, 1977 (Clark C)
NEW - Data obtained from charts and from previous analyses (Minimum t_p)

TABLE F2 Results of Regressions of Characteristic Response Times on Catchment Area and Length of Main Stream

Region	Number of Catchments	Results of Regression Analyses							
		$t = kA^n$				$t = kL^n$			
		k	n	r^Δ	SEE*	k	n	r^Δ	SEE*
Coastal	35	.65	.40	.93	1.54	.45	.70	.91	1.55
Snowy	38	.92	.34	.82	1.53	.71	.57	.79	1.55
Tablelands	23	.72	.39	.96	1.56	.52	.70	.96	1.61
All	96	.76	.38	.92	1.54	.54	.66	.91	1.57

Δ Correlation coefficient of logarithms

* Standard error of estimate, expressed as a multiplying or dividing factor on time

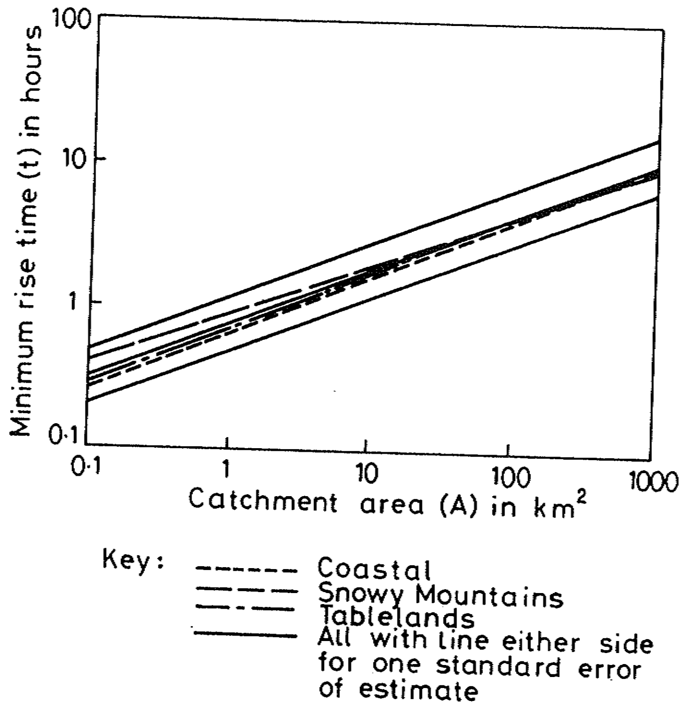

Figure F.1 Regional regression relations between response time and area.

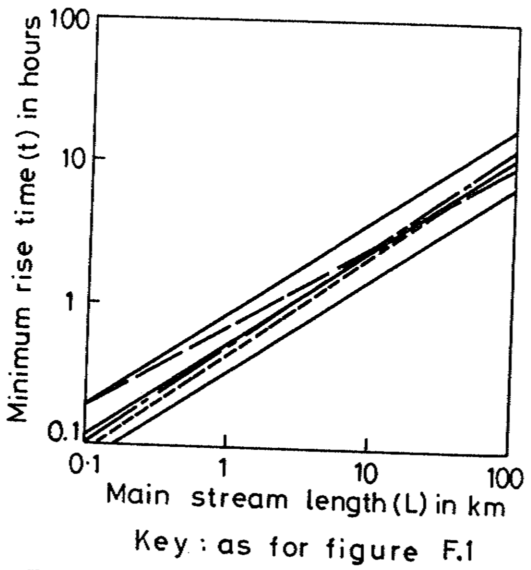

Figure F.2 Regional regression relations between response time and stream length.

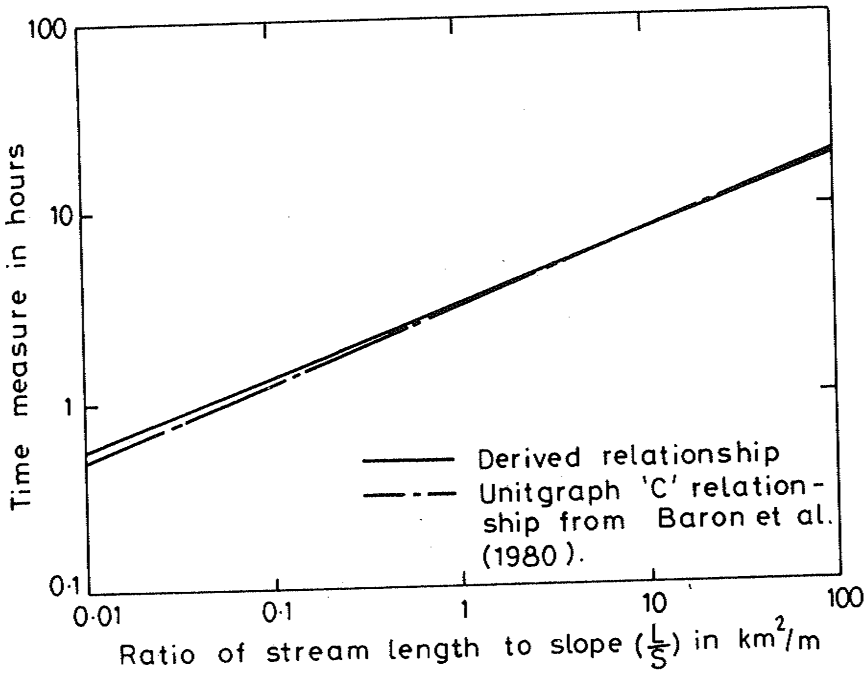

Figure F.3 Comparison of relations for unit hydrograph time measure C of Baron et al. (1980) and minimum times of hydrograph rise derived by regression.

APPENDIX G COMPARISON OF DESIGN RAINFALL INTENSITIES GIVEN BY PLUVIOGRAPH DATA AND GENERALISED PROCEDURES BASED ON DAILY DATA FROM "AUSTRALIAN RAINFALL AND RUNOFF"

In Section 5.2 reasons were given for choosing the generalised rainfall data presented in map form rather than the pluviograph data from ARR. The point was made that if the same rainfall data are used in derivation of runoff coefficients as are used in estimation of design floods, the observed floods of various frequencies should be reproduced. However, it is still very desirable that the best possible values of rainfall intensity be used in both derivation and application of runoff coefficients. Thus for completeness, a comparison was made between rainfall intensities calculated from pluviograph data at pluviograph sites and rainfall intensities extracted from rainfall maps based on daily data at these sites, in both cases using the design data in ARR. This was done primarily to show that there are differences between rainfall intensities calculated from different data sources, and that these differences are large enough to warrant the stipulation that rainfall intensities to be used with the runoff coefficients derived in this report must be calculated from the generalised procedure using the rainfall maps and based on daily data, rather than from the pluviograph data.

Rainfall intensity varies with both duration and return period, so that rainfall intensities were extracted from each data source for durations of 1 and 12 hours and return periods of 2 and 50 years. These data were extracted for eighteen pluviograph sites covering a range of topography from Coastal to Western Plains. The extracted values are listed in Table G.1 with the calculated ratios of rainfall map value to pluviograph data value. Some inferences from these ratios are:

(a) In general there is a fairly good agreement between intensities from each data source.

(b) Region, return period and duration have little overall effect on the magnitude of the ratio of the values from the two sources.

(c) Some differences should be expected, caused by the peculiarities of the particular pluviograph site as opposed to the larger areal spread of daily rainfall stations used to derive the rainfall maps in ARR.

(d) It is not possible to derive intensities for short durations for sites in the Western Plains region using the rainfall maps in ARR in a similar fashion to that used for Eastern NSW.

(e) Although the overall agreement of values from the two data sources is good, some individual differences are as large as 50%. For calculation of intensities for sites some distance from pluviograph sites, these differences will increase in many cases.

(f) Use of rainfall map data will yield rainfall intensity-frequency-duration estimates of comparable accuracy to pluviograph data for eastern NSW, based on the above inferences.

TABLE G.1 Comparison of Rainfall Intensities Derived from ARR Pluviograph Data and Generalised Map Data for 18 Pluviograph Sites

		Rainfall Intensities (mm/h) extracted from Pluviograph Data (Ip) and Generalised Map (Im) sources											
	Duration (h)	1						12					
	Return Period(yr)	2			50			2			50		
Region	Pluviograph Site	Ip	Im	Ip/Im	Ip	Im	Ip/Im	Ip	Im	Ip/Im	Ip	Im	Ip/Im
Coastal	Condong	44.0	39.5	1.1	74.1	86.9	0.9	11.7	10.0	1.2	26.4	22.0	1.2
	Bellbrook	36.4	31.6	1.2	79.8	64.8	1.2	8.1	8.0	1.0	19.2	16.4	1.2
	Williamtown	31.3	33.5	0.9	51.3	78.4	0.7	6.7	7.0	1.0	9.7	16.4	0.6
	Sydney	39.8	38.2	1.0	78.0	76.5	1.0	8.7	8.0	1.1	16.8	16.0	1.1
	Robertson	36.5	47.8	0.8	79.2	95.6	0.8	11.6	10.0	1.2	29.6	20.0	1.5
	Araluen	25.4	23.4	1.1	61.9	46.8	1.3	8.1	6.5	1.3	17.0	13.0	1.3
Table-lands	Tenterfield	30.8	33.6	0.9	55.7	57.0	1.0	4.5	5.9	0.8	10.4	10.0	1.0
	Walcha	27.8	21.3	1.3	45.7	43.5	1.1	4.6	5.4	0.8	7.0	11.0	0.6
	Kiandra	18.4	15.5	1.2	28.5	27.0	1.1	5.1	4.3	1.2	8.5	7.5	1.1
	Burrinjuck Dam	20.1	20.3	1.0	33.8	36.3	0.9	4.4	4.2	1.0	7.4	7.6	1.0
	Canberra	22.1	21.0	1.0	45.2	38.2	1.2	4.3	4.4	1.0	7.4	8.0	0.9
Western Slopes	Wellington	26.6	27.8	1.0	39.1	51.1	0.8	6.7	5.0	1.3	9.7	9.2	1.1
	Cowra	22.8	24.1	0.9	42.3	44.4	1.0	4.3	4.4	1.0	8.4	8.0	1.1
	Wagga Wagga	25.5	21.6	1.2	41.5	38.9	1.1	3.9	3.9	1.0	7.2	7.0	1.0
Western Plains	Deniliquin	18.6			36.6			3.6	3.4	1.0	6.5	6.5	1.0
	Mildura (Vic.)	16.4			44.0			3.1	3.0	1.0	7.0	6.2	1.1
	Woomera (SA)	13.7			35.7			2.6	2.8	1.0	5.3	7.0	0.8
	Windorah (Qld)	17.3			46.7			2.8	4.3	0.6	8.2	9.9	0.8

APPENDIX H. CALCULATION OF RAINFALL INTENSITIES WITH A PROGRAMMABLE CALCULATOR

This appendix presents the key sequence for the programming of a TI59 model calculator to calculate rainfall intensities for return periods of 1, 2, 5, 10, 20, 50 and 100 years. The program is listed in Table H.1. The intensities can be calculated for any design duration. The inputs required are catchment area to calculate design duration in the adopted formula (equation 4.2), and the three rainfall parameters zone, I(12,2) and I(12,50). These can be extracted from the reproductions of ARR maps presented here as Figures 5.2, 5.3 and 5.4.

Rainfall intensities using this program (Table H.1) can be calculated for catchments in the size range 0.005 to 1400 km^2 (i.e. 6 mins<t<12 hours from equation 4.2). However, the maximum size of catchment for which the design procedure in this report is recommended is 250 km^2. The method and its use are described in the following steps:

1. Locate Catchment on best available map scale.
2. Extract Latitude and Longitude of approximate catchment centroid.
3. Measure Catchment area from map.
4. Extract rainfall parameters based on location extracted in (2) above:

   Figure 5.2 - Zone Factor (AFACT)
   Figure 5.3 - I(12,2) mm/h
   Figure 5.4 - I(12,50) mm/h

5. Input the programme sequence shown in Table H1.
6. Input the necessary parameters as below:

   RST, R/S, Catchment Area (km^2), R/S, Zone Factor
   R/S, I(12,2) mm/h, R/S, I(12,50) mm/h, R/S.

7. Recall the rainfall intensities calculated for the particular design duration for any of the return periods of interest: 1, 2, 5, 10, 20, 50, and/or 100 years.

It would then be a simple matter to calculate flood peak estimates for any of the above return periods using the design runoff coefficients derived in this report.

Table H.1 Program Key Sequence

Description of Step	Key Sequence For Step (TI59)
1. Inputs Required (in order)	CLR, LRN,
- catchment area (km^2)	R/S, Sto03,
-zone factor (Figure 5.2)	R/S, Sto04,
-I(12,2)mm/hr(Figure 5.3)	R/S, Sto06,
-I(12,50)mm/hr (Figure 5.4)	R/S, Sto07,
2. Calculate Design Duration (t) and calculate appropriate multiplying factor for this duration (see Table 5.1 in main report); Memory08≡ design duration	1, x⇄t, RCL03, Y^x, 0.38, x,0.76,=,Sto08, 2nd, x> t,A RCL08,x,60,+,11.767, =, $\frac{1}{x}$,x,49.586,

Table H.1 Cont/d...

Memory 09≡ multiplying factor	+,0.309,=,x,(,cRCL04, +,1,),=,Sto09, GTO,B,2nd,Lbl,A, 1.798,-,(RCL08,+, 0.576,),-,0.143, =,x,RCL04,+,1, =Sto09,2nd, Lbl,B,
3. Calculate 2 and 50 year return period rainfall intensities and proportional increase factor for other return periods (Figure 2.46,ARR); Memory 02 - I(t,2) mm/h Memory 50 - I(t,50) mm/h Memory 11 - increase factor	RCL06,x,RCL09,=,Sto02, RCL07,x,RCL09,=,Sto50 RCL50,-,RCL02,=,÷, 129.5,=,Sto11,
4. Calculate Rainfall intensities for other return periods:- Memory Intensities (mm/h) 01 - I(t,1) 05 - I(t,5) 10 - I(t,10) 20 - I(t,20) 59 - I(t,100)	RCL02,+,RCL11,x, 40,=, Sto05, RCL02,+,RCL11,x, 61,=,Sto 10, RCL02+RCL11,x, 95,=,Sto20, RCL02,+,RCL11,x, 160,=,Sto59, 1.13,x,RCL50,÷,RCL02 =,lnx,x,0.1734,+, 1,=,$\frac{1}{x}$,x,0.885, x,RCL02,=,Sto01, RST,R/S, LRN

APPENDIX I. RESULTS OF REGRESSION AND CORRELATION STUDIES

This appendix contains the various regression and correlation studies referred to in Chapters 7 and 8 and Appendix L. They are presented in the form of fourteen summary tables. The requirements for and inferences from each study are not described here as the main text contains the relevant descriptions. The sections where each table is referred to are listed below:-

TABLE NUMBER	REFERENCE IN SECTIONS:-
I2	7.3
I3	7.3
I4	7.3
I5	7.3
I6	7.3,L.2,L.3
I7	8.1.5
I8	8.3.3, 8.4
I9	8.3.4
I10	8.3.4
I11	8.3.4
I12	L.2,L.4
I13	L.2
I14	L.4
I15	L.4

The regressions and correlations were computed using a standard IBM simple and multiple linear regression program. A description of regression analysis will not be given here as it is available in any standard statistical text. The regressions have been derived to obtain predictive relations between the various variables, and also to compare various forms of relations between the variables. The degree of association has generally been indicated by correlation coefficients or coefficients of multiple correlation. Both the regressions and correlation measures often apply to the logarithms of the variables rather than to the actual values of the variables. Standard errors of estimate have also been given in Table I6 in the form of multiplying and dividing factors for the actual values as the relations are of logarithmic form.

The choice between alternative forms of regression relations has been aided by the values of correlation coefficients. It has been recognised that correlation coefficients considered in isolation provide an inadequate basis for assessment and comparison of derived regression ralations. Consideration of the form of the relations and all relevant physical factors has also been used to assess and select the relations. Significance tests have also been carried out on the correlations, as the significance of the correlation coefficient depends to a major extent on the size of the sample. For simple regressions, the significance of the correlation coefficient has been examined by the standard t test with (N-2) degrees of freedom using the statistic

$$t = r\sqrt{(N-2)}\Big/\sqrt{(1-r^2)} \qquad \text{I.1}$$

where r = value of the correlation coefficient
and N = number of data points in sample.

For multiple linear regressions, the significance of the coefficient of multiple correlation has been examined by the F test using the statistic

$$F = \frac{r^2}{1-r^2}\ \frac{N-K}{K-1} \qquad \text{I.2}$$

where K = number of independent factors in the relation.

As noted above, where logarithms of values are used in the regression relations, the correlation coefficients and significance levels apply to the logarithmic values. In general, a probability level of 5% or lower has been adopted in this report to indicate a significant degree of correlation and that the apparent relation is unlikely to have occurred by chance.

The variables used in the regression and correlation studies reported in this appendix are listed below in Table I.1 in alphabetical order. Their symbols, meanings and units are given, together with the tables in which they are used.

Table I.1 Variables used in Regression and Correlation Studies

Symbol	Meaning	Units	Tables in which used
A	Catchment area	km^2	7
C(Y)	Y-year runoff coefficient	-	2, 3, 4, 5
C_D	10-year runoff coefficient derived from observed floods	-	7
C_P	10-year runoff coefficient predicted from design map at back of report	-	7
FF_y	Frequency factor by which to multiply C(10) to obtain C(Y) for a station	-	8, 9, 10,11
$\overline{FF}_y$	Average frequency factor for return period Y years		
	- for eastern NSW	-	8
	- for each of the three rainfall zones	-	9, 10, 11
FFR_y	Ratio of frequency factor FF_y for individual station to average $\overline{FF}_y$ for region.	-	8, 9, 10, 11
I(12,Y)	Rainfall intensity of 12-hour duration and Y-year return period	mm/h	2, 6, 8, 9, 10, 11, 12, 13, 14, 15

Table I.1 Cont/d...

MAR	Median annual rainfall for a catchment	mm inches	6, 12, 14, 15 3, 13
MF	Multiplying factor by which to multiply predicted C(10) to obtain derived C(10) -i.e. C_D/C_P	-	7
Q(Y)	Y-year flood peak	m^3/s	6
RL	Record length of observed floods used in derivation of partial series	years	2, 12, 14, 15
S_e	Equal area slope of main stream	m/km	7
S_a/S_e	Slope non-uniformity index - ie., ratio of average slope to equal area slope of main stream	-	5, 7, 8, 9, 10, 11 12, 13, 14, 15
SDEV	Standard deviation of the logarithms of the floods in the partial series at a station	$\log(m^3/s)$	12, 13, 14
SKEW	Skew coefficient of the logarithms of the floods in the partial series at a station	-	12, 13, 15
X	General symbol for a variable		

A point of interest not referred to elsewhere in the report is that record length (RL) is significantly correlated with catchment area (Table I.12). This reflects the fact that larger catchments have in general been of greater interest to gauging authorities than smaller catchments. Concern for the gauging of small catchments has only developed fairly recently, and the records available for analysis in this project indicate that there is still a deficiency of data for small catchments.

The remainder of this appendix presents the tables summarising the various regression and correlation studies.

Table I.2 Relation of runoff coefficients of various return periods to 12-hour rainfall intensity.

Return Period Y years	Number of Stations	Relation C(Y) = k + a.I(12,Y) k	a	Correlation Coefficient	Significance Level - %
1	284	-0.259	0.110	0.60	<0.2
2	284	-0.319	0.108	0.66	<0.2
5	284	-0.280	0.087	0.64	<0.2
10	284	-0.270	0.085	0.62	<0.2
20	214	-0.238	0.073	0.66	<0.2
50	105	-0.229	0.072	0.66	<0.2
100	73	-0.178	0.065	0.66	<0.2

Table I.3 Relation of runoff coefficients of various return periods to median annual rainfall (inches)

Return Period Y years	Number of stations	Relation C(Y) = k + a.MAR k	a	Correlation Coefficient	Significance Level-%
1	284	-0.076	0.0082	0.49	<0.2
2	284	-0.088	0.0102	0.52	<0.2
5	284	-0.067	0.0114	0.50	<0.2
10	284	-0.052	0.0127	0.48	<0.2
20	214	0.041	0.0110	0.44	<0.2
50	105	0.693	-0.00022	0.17	8
100	73	0.606	0.00027	0.05	60

Table I.4 Relation of runoff coefficients of various return periods to elevation above sea level

Relation $C(Y) = k + a.X$
where X = elevation above sea level (m)

Return Period Y years	Number of Stations	k	a	Correlation Coefficient	Significance Level-%
1	284	0.278	-7×10^{-5}	0.12	3.6
2	284	0.361	-1.1×10^{-4}	0.16	0.5
5	284	0.428	-1.1×10^{-4}	0.14	1.6
10	284	0.501	-1.3×10^{-4}	0.14	2.1
20	214	0.565	-2.0×10^{-4}	0.22	<0.2
50	105	0.693	-2.2×10^{-4}	0.17	7.6
100	73	0.663	-9×10^{-5}	0.07	56

Table I.5 Relation of runoff coefficients of various return periods to slope non-uniformity index of main stream

Return Period Y Years	Number of Stations	Relation $C(Y) = k + a.S_a/S_e$		Correlation Coefficient	Significance Level-%
		k	a		
1	284	0.206	0.017	0.10	11
2	284	0.244	0.030	0.14	1.4
5	284	0.322	0.024	0.10	9
10	284	0.398	0.020	0.07	22
20	214	0.394	0.034	0.11	9.5
50	105	0.593	-2.4×10^{-4}	0.04	68
100	73	0.999	-0.170	0.01	>80

Table I.6 Multiple regression relations for derived floods of various return periods

Return Period Y years	Number of Stations	$Q(Y) = k.A^a.\ [I(12, Y)]^b\ .MAR^c$				r of Logs	SEE -ratios
		k	a	b	c		
1	284	0.0005	0.69	2.27	0.69	0.89	2.0
2	284	0.0005	0.72	2.49	0.59	0.91	1.9
5	284	0.0007	0.74	2.40	0.53	0.92	1.9
10	284	0.0011	0.74	2.31	0.51	0.92	1.9
20	214	0.0020	0.75	2.37	0.37	0.92	1.9
50	105	0.0134	0.65	0.58	0.79	0.81	2.4
100	73	0.1470	0.66	2.83	-0.34	0.91	1.8

Table I.7 Relations for multiplying factor ($MF = C_D/C_P$) for obtaining the derived from the predicted 10-year runoff coefficient.

Relations tested for estimating $MF = C_D/C_P$	Parameters			Significance level - %
	k	a	b	
$MF = k + a\ (\log A)$	1.069	-0.022	-	25
$MF = k + a\ (\log S_e)$	0.996	0.031	-	35
$MF = k + a\ (S_a/S_e)$	1.041	-0.007	-	60
$MF = k + a\ (\log A) + b(S_a/S_e)$	1.076	-0.020	-0.004	>50

TABLE I.8 Relations of FFR_y to catchment characteristics for all of eastern NSW; where FFR_y is the ratio of the frequency factor FF_y for an individual station to the average frequency factor $\overline{FF}_y$ for the region - ie. $FFR_y = FF_y / \overline{FF}_y$

Return period Y years	Number of Stations	Overall average $\overline{FF}_y$	FFR_y = k + a (log A) + b.S_a/S_e + c log I(12,Y)				Correlation coefficient	Significant for probability < 5%
			k	a	b	c		
1	284	0.576	1.068	-0.108	0.038	0.075	0.21	YES
2	284	0.700	0.872	-0.956	0.046	0.184	0.27	YES
5	284	0.845	0.916	-0.017	0.016	0.098	0.23	YES
10	284	1.000	-	-	-	-	-	-
20	214	1.127	1.110	0.016	-0.013	-0.120	0.25	YES
50	105	1.396	1.286	0.045	-0.048	-0.265	0.32	YES
100	73	1.573	1.531	0.027	-0.065	0.424	0.33	YES

TABLE I.9 Relations of FFR_y to catchment characteristics for zone (A + B); where FFR_y is the ratio of the frequency factor FF_y for an individual station to the average frequency factor $\overline{FF}_y$ for the zone - ie. $FFR_y = FF_y/\overline{FF}_y$.

Return period Y years	Number of Stations	Zone average $\overline{FF}_y$	$FFR_y = k + a\,(\log A) + b.S_a/S_e + c\,I(12,Y)$				Correlation coefficient	Significant for probability < 5%
			k	a	b	c		
1	149	0.60	1.070	-0.097	0.032	0.010	0.21	NO
2	149	0.70	0.965	-0.057	0.028	0.019	0.24	YES
5	149	0.86	0.973	-0.020	0.008	0.005	0.21	NO
10	149	1.00	1.000	-	-	-	-	-
20	116	1.13	1.038	0.018	-0.015	-0.005	0.23	NO
50	45	1.36	0.996	0.065	-0.041	-0.004	0.30	NO
100	29	1.55	0.669	0.297	-0.106	-0.012	0.61	YES

Table I.10 Relations of FFR_y to catchment characteristics for zone C; where FFR_y is the ratio of the frequency factor FF_y for an individual station to the average frequency factor $\overline{FF}_y$ for the zone - ie. $FFR_y = FF_y/\overline{FF}_y$.

Return period Y years	Number of Stations	Zone average $\overline{FF}_y$	$FFR_y = k + a (\log A) + b.S_a/S_e + c\ I(12,Y)$				Correlation coefficient	Significant for probability < 5%
			k	a	b	c		
1	52	0.60	0.268	-0.008	-0.010	0.128	0.36	NO
2	52	0.73	0.413	0.082	0.023	0.052	0.45	YES
5	52	0.88	0.735	0.080	0.003	0.010	0.31	NO
10	52	1.00	1.000	-	-	-	-	-
20	38	1.08	1.289	-0.055	-0.001	-0.014	0.52	YES
50	26	1.22	1.100	0.058	-0.076	-0.001	0.58	YES
100	18	1.36	0.918	0.192	-0.086	-0.004	0.68	YES

Table I.11 Relations of FFR_y to catchment characteristics for zone (D + E); where FFR_y is the ratio of the frequency factor FF_y for an individual station to the average frequency factor $\overline{FF}_y$ for the zone i.e. $FFR_y = FF_y / \overline{FF}_y$.

Return period Y years	Number of Stations	Zone average $\overline{FF}_y$	$FFR = k + a\ (\log A) + b.S_a/S_e + c\ I(12,Y)$				Correlation coefficient	Significant for probability $<5\%$
			k	a	b	c		
1	83	0.49	1.399	-0.053	0.004	-0.074	0.14	NO
2	83	0.62	0.995	-0.016	0.040	-0.004	0.12	NO
5	83	0.82	0.910	-0.013	0.020	0.012	0.16	NO
10	83	1.00	1.000	–	–	–	–	–
20	60	1.18	0.773	-0.013	0.022	0.025	0.25	NO
50	33	1.48	0.479	-0.059	0.035	0.071	0.28	NO
100	26	1.75	0.692	-0.117	0.024	0.052	0.35	NO

Table I.12 Regressions and correlations between variables used in the study

Variable (X1)	Variable (X2)	(X1) = k + a (X2)		Correlation coefficient	Significance level-%
		k	a		
SKEW	SDEV	1.082	-1.078	0.06	32
S_a/S_e	log A	1.585	0.305	0.21	<0.2
log A	I(12, 2)	2.496	-0.084	0.16	0.6
log A	MAR	2.338	-0.00035	0.14	2.1
log A	RL	1.729	0.139	0.20	<0.2
S_a/S_e	I(12, 2)	0.604	0.306	0.21	<0.2
S_a/S_e	MAR	2.05	0.00035	0.05	38
S_a/S_e	RL	2.701	-0.016	0.08	16
I(12, 2)	MAR	3.141	0.0027	0.58	<<0.2
RL	I(12, 2)	21.80	-0.296	0.04	50
RL	MAR	19.68	0.00039	0.01	>80

Table I.13 Mean and standard deviation within each ARR rainfall zone of the variables used in the study

Source of values of variables	Variable X	Southern coast & tablelands - Zone A - 34 catchments		Central coast & tablelands - Zone B - 115 catchments		Northern Coast - Zone C - 52 catchments		Northern tablelands - Zone D - 49 catchments		Eastern Interior - Zone E - 34 catchments	
		$\overline{X}$	SDEV(X)	$\overline{X}$	SDEV(X)	$\overline{X}$	SDEV(X)	$\overline{X}$	SDEV(X)	$\overline{X}$	SDEV(X)
Observed flood data for station	RL	14.5	5.3	20.1	10.9	20.4	8.2	24.0	14.1	19.8	17.5
	SDEV	0.222	0.069	0.215	0.079	0.226	0.069	0.256	0.089	0.249	0.107
	SKEW	0.795	0.696	0.768	0.650	0.565	0.673	0.767	0.453	0.795	0.706
ARR and other rainfall data for catchment location	I(12, 2)	5.94	1.16	5.55	1.40	7.56	1.60	5.56	0.59	4.53	0.71
	I(12,50)/I(12,2)	2.002	0.212	1.895	0.191	2.086	0.248	1.816	0.124	1.855	0.125
	MAR (in)	36.4	8.3	39.0	11.7	54.3	11.0	34.1	5.1	25.0	3.6
Topographic maps	log A	1.976	0.582	1.755	0.970	1.981	0.438	2.375	0.632	2.402	0.795
	S_a/S_e	1.813	0.693	2.158	1.216	2.819	1.637	1.882	0.824	2.210	1.107

Table I.14 Relation of standard deviation of the logarithms of partial duration floods to catchment variables.

Variable X	Units	SDEV = k + a. X		Correlation coefficient	Significance level-%
		k	a		
log A	km^2	0.215	0.0081	0.08	18
S_a/S_e	-	0.243	-0.0054	0.08	18
I(12,2)	mm/h	0.242	-0.0022	0.04	48
MAR	mm	0.278	-0.0005	0.20	<0.2
I(12,50)/I(12,2)	-	0.222	0.0036	0.01	>80
RL	years	0.217	0.00062	0.09	14

Table I.15 Relation of skew coefficient of the logarithms of partial duration floods to catchment variables

Variable X	Units	SKEW = k + a. X		Correlation coefficient	Significance level-%
		k	a		
log A	km^2	0.624	0.040	0.06	29
S_a/S_e	-	0.889	-0.077	0.15	1.3
I(12,2)	mm/h	1.358	-0.090	0.09	12
MAR	mm	1.699	-0.00079	0.13	3.1
I(12,50)/I(12,2)	-	1.565	-0.429	0.15	1.1
RL	years	1.020	-0.009	0.07	22

APPENDIX J LIST OF RUNOFF COEFFICIENTS DERIVED FROM OBSERVED FLOOD DATA

The runoff coefficients derived by the adopted Method 2 are listed below for the 284 catchments in eastern New South Wales and the six catchments in the arid zone. Values are given for each of the various return periods used for each catchment. For catchments marked with an asterisk, small adjustments have been made to the values as a result of additional checking of the flood frequency data late in the project, and the values listed are slightly different to those shown on Map 2 at the back of this report. However, the difference in the coefficient values is generally not greater than 0.02, so that the practical effect is very small. The coefficients listed below are the correct values.

RIVER BASIN	STATION NUMBER	AREA (km^2)	C(1)	C(2)	C(5)	C(10)	C(20)	C(50)	C(100)	
TWEED(201)	001	213	.79	.87	.81	.82	.77	.77	.76	
	004	49	.53	.59	.59	.61	.59	.61		
	005&2	111	.73	1.15	1.14	1.16	1.07	1.02	.96	
BRUNSWICK(202)	001	34	.60	.52	.45	.44				
RICHMOND(203)	002	62	.72	.81	.86	.93	.94	1.00	1.03	*
	007	44	1.01	1.10	1.18	1.30	1.35	1.48		
	009	109	.37	.49	.69	.90	1.08	1.45		
	010	179	.52	.67	.86	1.02	1.15			*
	012	39	.75	.86	1.00	1.13	1.19	1.34	1.44	
	013	54	.70	.94	1.24	1.50	1.68	2.01		*
	014	223	.25	.21	.18	.17	.16	.15		
	015	109	.58	.66	.70	.77	.79			
	023	98	.21	.25	.30	.34				
CLARENCE (204)	006	127	.15	.24	.42	.57				
	008	31	.46	.52	.57	.68	.77			*
	011	10	.45	.52	.62	.73				
	014	881	.15	.33	.56	.81	1.07			
	015	2670	.15	.19	.22	.26	.29			*
	016	104	.40	.52	.60	.69	.74	.85	.93	
	017	82	.42	.63	.87	1.10	1.28	1.62	1.89	
	019	220	.24	.34	.49	.63	.75	.98	1.16	
	020	251	.23	.31	.41	,53	.62	.82	1.00	
	021	70	.47	.53	.58	.66	.69	.78	.85	
	022	39	.54	.55	.56	.59				
	023	26	.48	.47	.48	.51				

RIVER BASIN	STATION NUMBER	AREA (km^2)	C(1)	C(2)	C(5)	C(10)	C(20)	C(50)	C(100)	
	024	31	.85	.92	1.06	1.24	1.36	1.61	1.83	
	025	132	.80	.89	.86	.86	.80	.76		
	026	80	.76	.85	.95	1.07	1.13	1.28	1.38	*
	027	31	.81	.84	.88	.94	.94			
	030	200	.11	.15	.17	.20	.21	.24	.26	
	032	60	.11	.13	.19	.24				*
	033	985	.28	.35	.41	.47	.50			
	035	135	.20	.31	.41	.48				
	036	300	.21	.31	.40	.47	.51	.60		
	037	62	.35	.47	.58	.70				
	038	18	.21	.35	.49	.60				
	040	231	.37	.39	.43	.47	.49			
	043	47	.19	.26	.37	.47				
	044	41	.48	.53	.58	.63	.64			
BELLINGER (205)	002	433	.40	.63	.71	.77				*
	003	13	.72	1.03	1.67	2.39				
	004	166	.27	.51	.68	.77				
	006	539	.31	.61	.67	.67				
	007	11	.41	.61	.85	1.07	1.24			
MACLEAY (206)	001	163	.27	.42	.55	.65	.72	.87	.97	*
	004	407	.17	.29	.38	.45	.49	.57	.62	*
	005	202	.19	.36	.58	.77	.94			*
	008	383	.31	.44	.53	.60	.63			
	009	251	.12	.21	.32	.41	.48	.61	.70	
	010	70	.13	.18	.24	.29	.33	.40	.45	*
	013	47	.13	.18	.23	.28				
	014	376	.21	.32	.40	.46	.49	.55		
	015	205	.51	.53	.52	.56	.55	.59	.62	
	017	22	.32	.49	.51	.57	.58	.63	.67	
	018	894	.22	.34	.42	.47	.49			
	020	78	.31	.56	.61	.69	.69			*
	021	135	.19	.27	.30	.34	.35			
	023	130	.18	.24	.25	.26				

APPENDIX J Cont/d...

HASTINGS (207)	003	60	.50	.65	.74	.81	.82	.87		
	006	363	.25	.40	.44	.46	.44			
	008	60	.38	.42	.46	.51				
	009	181	.36	.34	.35	.37				
MANNING (208)	001	21	.22	.27	.32	.37	.39	.45	.48	
	002	52	.17	.18	.21	.24	.26			*
	007	218	.11	.16	.20	.25	.28	.34	.38	
	008	207	.40	.46	.50	.55	.58	.64	.68	*
	009	150	.12	.19	.23	.29	.34			
	015	96	.63	.60	.63	.74				
KARUAH (209)	001	202	.51	.69	.94	1.16	1.34			
	005	259	.43	.53	.61	.67				
HUNTER (210)	011	194	.49	.58	.65	.72	.74	.81	.86	
	017	98	.11	.13	.16	.18	.19	.22	.24	*
	019	104	.10	.12	.15	.18	.21	.26	.30	
	021	277	.30	.40	.52	.63	.72			*
	022	205	.36	.42	.45	.47	.47	.48	.48	
	025	168	.17	.21	.22	.24	.24			
	026	85	.41	.42	.39	.36	.34	.33		
	029	246	.23	.33	.44	.53	.58			
	034	733	.04	.07	.11	.17	.22			
	037	585	.19	.21	.25	.32	.39			
	040	658	.10	.12	.15	.20				
	042	205	.06	.07	.10	.13	.16	.23		
	043	78	.07	.10	.12	.13	.14			
	045	41	.14	.17	.22	.29	.36	.52		
	046	153	.03	.05	.08	.15	.23			
	049	9	.18	.16	.15	.15				*
	052	1050	.14	.19	.27	.33	.40			*
	053	83	.09	.14	.16	.18	.18			*
	054	95	.22	.28	.31	.36	.38			*
	059	71	.18	.24	.34	.45	.54			*
	063	14.8	.12	.15	.19	.24				
	067	7.8	.17	.23	.28	.34				

APPENDIX J Cont'd/...

RIVER BASIN	STATION NUMBER	AREA (km)	C(1)	C(2)	C(5)	C(10)	C(20)	C(50)	C(100)
(210) cont'd.	068	24.9	.14	.19	.23	.28			
	069	4.9	.06	.09	.11	.13			
	074	1.04	.16	.29	.42	.52			
	076	14.2	.41	.51	.46	.43			
	078	6.5	.20	.23	.24	.26			
	084	249	.21	.22	.28	.31			
(Assumed No.	099	0.18	.05	.06	.08	.11	.12	.16	
MACQUARIE - TUGGERAH (211)	001	18	.22	.21	.18	.18	.17		
	002	249	.10	.14	.22	.30			
	005	150	.19	.24	.28	.32			
	006	8.8	.79	.75	.69	.69			
	008	52	.58	.49	.40	.38			
HAWKESBURY (212)	003	25.9	.13	.19	.24	.29	.32	.38	.42
	008	199	.13	.16	.22	.28	.33	.41	.47
	011	404	.12	.14	.18	.23			
	012	622	.11	.15	.19	.25	.29		
	013	24.6	.44	.64	.92	1.15			
	014	18.9	.21	.22	.22	.22			*
	016	75	.22	.23	.28	.32			
	019	202	.35	.43	.51	.60			
	203	686	.16	.28	.40	.51	.59	.74	.85
	209	72.5	.28	.51	.78	1.01	1.18	1.48	1.70
	210	148	.24	.30	.41	.53	.63	.82	
	231	163	.21	.35	.49	.60	.67	.80	.89
	291	650	.49	.51	.56	.64	.69	.80	.89
	301	0.06	.04	.06	.09	.12			
	302	0.13	.12	.16	.21	.27			
	304	0.08	.14	.17	.18	.19			
	306	0.14	.15	.15	.16	.18			
	307	0.04	.20	.24	.27	.28			
	320	89.6	.26	.29	.31	.33	.34	.37	
	333	0.70	.33	.49	.58	.64	.64	.66	
	340	24.9	.42	.44	.43	.45	.43	.44	

APPENDIX J Cont/d...

SYDNEY COAST (213)	200	74.5	.44	.55	.69	.84	.95	1.17	1.36	
WOLLONGONG COAST (214)	003	31	.41	.55	.86	1.18	1.51	2.15		
	310	2.54	.79	.91	1.00	1.11				
	320	0.93	.59	.66	.68	.72	.70			
	330	0.39	.77	1.00	1.26	1.49				
	334	5.52	.49	.51	.60	.72	.82			
	340	40.2	.72	.78	.90	1.02	1.07			
SHOALHAVEN (215)	004	166	.55	.62	.71	.79	.82	.91	.97	
	006	130	.34	.46	.57	.66	.70			
	009	210	.86	.88	.91	.97	.97	1.03		
	010	241	.72	.81	.86	.92	.92			
	223	67.3	.90	.97	1.12	1.29				
	233	7.3	.18	.16	.14	.14				
CLYDE (216)	001	161	.88	1.00	1.18	1.36				
	002	880	.69	.96	1.27	1.53				
MORUYA (217)	001	891	.22	.30	.36	.41				
	003	129	.41	.62	.60	.58				
TUROSS (218)	001	93	.29	.27	.26	.27	.26			
	003	103	.34	.35	.32	,32	.31			
	006	64	.26	.45	.46	.46				
BEGA (219)	001	18.6	.21	.24	,30	.38	.45	.61		
	004	148	.37	.45	.50	.55	.57			
	006	88	.14	.29	.50	.75	1.04			
	008	18	.17	.19	.20	.21				
	009	7.8	.68	.65	.63	.66				
	010	3.6	.12	.15	.18	.21	.22			*
	015	25	.23	.28	.41	.58				*
	016	77	.37	.78	1.02	1.22				
	017	144	.38	.57	.78	1.03				
	018	38	.46	.46	.56	.69				
	020	20	.21	.24	.28	.32				
	021	124	.35	.41	.48	.57				

TOWAMBA (220)	002	72	.09	.13	.17	.20			
	003	129	.35	.43	.45	.49			
GENOA (221)	202	777	.08	.09	.10	.11			
SNOWY (222)	001	277	.18	.25	.35	.45	.53		
	004	621	.05	.06	.09	.13	.17	.25	.34
	007	556	.05	.09	.15	.20	.25		
	009	543	.32	.38	.45	.55	.64		
	012	160	.66	.67	.71	.76			
	400	28.5	.10	.10	.12	.14			
	404	69.9	.07	.09	.12	.14			
	513	12.4	.72	.71	.75	.79	.78		
	522	165	.26	.28	.33	.39	.43		
MITCHELL (224)	402	50	.08	.08	.07	.07			
UPPER MURRAY (401)	006	116	.07	.08	.10	.13	.15	.20	.24
	007	134	.06	.06	.07	.08	.08	.09	.09
	009	220	.07	.08	.10	.13	.17	.25	.34
	205	743	.05	.05	.06	.06	.07	.08	.08
	508	5	1.39	1.51	1.65	1.78			
	517	39	.10	.10	.11	.12			
KIEWA (402)	202	282	.08	.09	.11	.13	.14	.17	
	400	3.6	.17	.17	.18	.19	.19	.20	.21
OVENS (403)	206	302	.10	.11	.12	.12	.12	.13	.13
	213	223	.06	.07	.10	.12			
BROKEN (404)	207	448	.08	.08	.08	.09			
GOULBURN (405)	208	251	.09	.09	.08	.09	.09		
	229	109	.09	.12	.11	.11			
LODDON (407)	213	471	.07	.07	.08	.08	.08	.08	.08
	214	308	.07	.09	.12	.14	.15	.17	.19
	221	163	.14	.16	.20	.24	.27		
AVOCA (408)	200	2670	.05	.05	.05	.05	.05	.05	.05

MURRUMBIDGEE (410)	009	134	.11	.12	.14	.17	.19	.23	.27	
	010	98	.17	.18	.21	.24				
	029	30	.06	.06	.06	.06	.06	.06	.06	
	034	96	.10	.11	.11	.12	.12	.12	.12	
	059	233	.14	.18	.25	.31	.35	.44	.50	
	061	155	.17	.20	.23	.26	.28	.30	.32	
	063	220	.05	.08	.13	.19	.25	.36	.46	
	066	124	.20	.21	.24	.28	.31	.37		
	067	220	.53	.61	.65	.68	.68	.70	.70	
	070	67	.16	.20	.28	.35	.43			
	071	114	.11	.15	.21	.27				
	075	69	.35	.33	.31	.31	.30			
	076	217	.09	.17	.28	.37	.46	.82		*
	077	75	.24	.28	.32	.36	.38			
	080	708	.01	.01	.02	.02	.03			
	081	103	.10	.12	.16	.19				
	090	320	.17	.23	.29	.34				
	351	0.07	.15	.22	.33	.42	.49	.63	.74	
	506	227	.11	.12	.13	.15	.15			
	507	43.5	.09	.11	.14	.17	.19			
	514	116	.05	.06	.08	.09	.10			
	523	32	.06	.07	.09	.11	.12			
	524	104	.18	.19	.19	.20				*
	533	131	.27	.33	.41	.46				
	534	109	.16	.18	.24	.29				
	535	216	.14	.16	.22	.27				
LAKE GEORGE (411)	001	16	.32	.40	.47	.57	.64			
LACHLAN (412)	028	2670	.07	.11	.17	.24	.31	.44	.56	
	031	272	.08	.13	.19	.24	.29	.38	.45	
	063	570	.10	.11	.14	.18	.20			
	064	181	.18	.23	.30	.36				
	068	363	.04	.04	.05	.07				
	073	225	.06	.11	.18	.24				
	074	127	.15	.19	.25	.29				

LACHLAN (412) Cont/d...	075	350	.08	.09	.11	.13			
	077	233	.07	.09	.11	.13			
	090	272	.15	.14	.13	.14			
WIMMERA-AVON (415)	202	80	.02	.03	.05	.07	.09	.13	.18
	210	658	.03	.03	.03	.03	.03		
	217	36	.06	.06	.06	.06			
BORDER (416)	003	570	.10	.16	.26	.38	.52	.80	1.08
	004	1606	.07	.09	.13	.17	.21	.30	.37
	006	3160	.10	.12	.16	.20	.23	.29	.33
	008	907	.05	.08	.13	.20	.28	.45	
	010	2020	.09	.12	.15	.18	.20	.23	.26
	024	181	.09	.11	.12	.14			
	303	1036	.12	.16	.18	.20	.20	.21	.21
	304	70	.02	.02	.03	.04	.05	.06	
	401	3650	.05	.07	.10	.12	.13	.16	.18
	404	686	.07	.19	.34	.49	.62		
	407	1243	.05	.13	.23	.33	.42		
GWYDIR (418)	014	855	.18	.34	.65	1.01			
	025	156	.09	.10	.12	.14			
	027	220	.34	.39	.53	.75			
	030	67	.26	.27	.31	.34			
NAMOI (419)	004	310	.30	.38	.45	.52	.55	.62	.66
	015	1140	.10	.13	.17	.21	.23	.27	.31 *
	016	907	.11	.13	.18	.23	.28	.39	.49
	020	2020	.08	.13	.23	.37	.55	.97	
	024	2409	.10	.15	.22	.28	.32		
	027	3630	.02	.02	.02	.03			
	029	389	.03	.04	.07	.10			
	036	143	.02	.03	.06	.11			
	037	277	.10	.10	.11	.12			
	038	358	.12	.11	.12	.13			
	044	171	.10	.16	.28	.43			
CASTLEREAGH (420)	003	142	.10	.13	.20	.27	.36		
	004	3470	.05	.07	.10	.16	.23		
	009	331	.07	.24	.44	.61			

APPENDIX J Cont/d...

(420)	011	320	.01	.01	.02	.03				
	013	124	.15	.20	.30	.40	.50	.68	.85	
MACQUARIE-BOGAN (421)	007	2771	.06	.09	.12	.16	.20	.27	.34	
	010	543	.02	.04	.05	.06	.07	.09	.10	
	032	34	.06	.09	.14	.22				
	033	34	.06	.08	.14	.25	.42			
	034	16	.05	.10	.19	.32	.49	.52		
	035	570	.07	.08	.10	.14				
	036	104	.07	.09	.15	.23	.33	.34		
	038	544	.03	.06	.12	.18	.24			
	041	230	.19	.23	.26	.29	.29			
	050	365	.32	.30	.30	.31				
	051	31	.27	.25	.24	.25				
	053	202	.28	.30	.29	.29				
	056	225	.11	.17	.25	.32				
	066	119	.38	.43	.47	.51	.52			
	067	179	.35	.50	.63	.73			*	
CONDAMINE-CULGOA (422)	301	80	.10	.15	.20	.24	.28	.35	.40	
	302	21	.05	.08	.15	.20	.25	.33	.40	
	303	10	.03	.10	.23	.39	.57	.95	1.33	
	305	98	.06	.11	.21	.32	.45	.71		
	306	83	.13	.20	.29	.38	.46	.61	.73	
	307	332	.26	.35	.40	.44	.45	.47	.48	*
	313	140	.15	.25	.39	.53	.67	.94	1.18	
	317	466	.12	.14	.17	.19	.20			
	318	648	.20	.25	.36	.48	.61			
	319	254	.14	.24	.40	.58	.79	1.22		
	321	34	.13	.17	.23	.29				
DARLING (425)	016	15	.05	.08	.11	.13				
TODD (006)	003	5.2	.43	.48	.57	.62				
	009	450	.24	.35	.44	.48	.49	.52	.53	
	047	42	.18	.27	.45	.62	.78			
LAKE BANCANNIA (011)	001	20	.23	.34	.50	.64				
	315	.13	.23	.25						

APPENDIX K EXAMINATION OF DERIVED 10-YEAR RUNOFF COEFFICIENTS WITH LARGEST DEVIATIONS FROM CONTOURS OF DESIGN C(10) VALUES.

K.1 AIM OF INVESTIGATION

The contours of design values of the 10-year runoff coefficients for the design method developed in this report generally deviated to some ext t from the derived values. The frequency distribution of these deviations is listed in Tables 7.2 and 8.1 and illustrated in Figure 8.1(a). Although it has been shown in Section 8.1.4 that the deviations approximate closely those that would be expected from sampling errors inherent in the basic flood data, this appendix examines the factors that could lead to large magnitudes of these deviations. The 24 stations giving the twelve largest over and under estimates are reviewed on the basis of these factors.

K.2 POSSIBLE CAUSES OF DEVIATIONS

Three factors are involved in the derivation of the value of the 10-year runoff coefficient for a gauged catchment. These are:-

(i) the 10-year flood magnitude Q(10);

(ii) the 10-year rainfall intensity of duration t hours (derived from equation (4.2) $t = 0.76A^{0.38}$); and

(iii) catchment area A.

The main causes of error in each of these three factors are discussed below.

K.2.1 Flood magnitude

The main causes of error were identified as those resulting from the available sample of floods, extension of rating curves, and non-homogeneity of the record of flows.

a) Sampling error

Sampling error is a function of the number of the data points (years of record) used to construct the flood frequency curve, and the return period of the flood magnitude estimated from this curve (see ARR, Table 9.4). The longer the record length the more confidence can be placed in the flood frequency curve constructed from the observed flood sample.

With the large number of stations (284) used in deriving runoff coefficients, large errors should be expected for at least a few stations due to sampling error. For example, if there was a 5% probability that the true Q(10) value was twice or more times the sample value for a station with a record length of 10 years, then for 100 such stations a total of 5 stations should be expected to have Q(10) values twice or more times their true values. As noted above, an analysis of sampling error based on this type of consideration has been carried out (see Section 8.1.4) which indicated that all outliers, even extremes, could probably be attributed to sampling error. Novertheless the non-random causes of error are examined below.

b) Errors in extension of rating curves

To convert the record of river stages to discharges, a stage-discharge relation is required for the station. For small catchemnts the highest gauged discharge (see Figure D.1) is often considerably less than the once in 2-year discharge estimated from the record. Considerable extension of rating curves is thus often required. While several methods are commonly used for these extensions, all involve assumptions and appreciable errors are likely. These errors are systematic for a particular gauging station, but are probably random for many stations over a region, so that the resulting errors in estimating discharges and derived runoff coefficients could be under or over estimates. The magnitude of the probable error increases with the extension required.

c) Non-homogeneity

Several factors related to lack of homogeneity in the individual records or in the catchments analysed as representing small rural catchments can lead to errors in the estimated flood magnitudes.

Storages on the catchment:- Construction of storages on a catchment would generally lead to reduction of peak flows. The only exceptions are likely to be where gates on the spillway are operated in a manner to quickly release large volumes of water, or where the impounded water occupies much of the catchment. However, gated spillways are rare on dams on small catchments, and the area of impounded water is generally small relative to catchment area.

Urbanisation:- Increase in urban development is likely to cause an increase in peak discharges and in the speed of response of the catchment. However, the effect is only likely to be appreciable where a large portion of the catchment is urbanised. This was the case in only one of the catchments analysed, and no increase in flood magnitude could be detected.

Changes in land use:- Even on rural catchments, some changes in land use due to the activity of man are evident during the period of most records. In some cases large changes occurred. The effects of such changes are complex and often difficult to estimate (Boughton 1970). Increases or decreases can result from different changes, but in most cases the effects are fairly small. The effects are likely to be of greater importance on small catchments.

Daily read and continuous records:- Where different methods of recording stages are used at different stations, lack of homogeneity will be introduced into the data base. A change of recording method will also cause lack of homogeneity in the data at a particular station. As peaks are likely to be missed when the stage is read manually, flood estimates from daily read stations are likely to be lower than where stage is recorded continuously. This is more likely to occur with the rapid response of small catchments than on larger catchments with slower response. The frequency of intermediate readings in manual records also depends on the motivation and diligence of the gauge reader. With major floods equivalent to the 10-year event, it is likely that intermediate readings and peak heights would be recorded at most daily read stations. Errors in floods of this magnitude are thus likely to be relatively small, but the errors that do occur as a result of manual readings will be underestimates of the true values.

K.2.2. Rainfall intensity

For a given catchment, only two variables affect the design rainfall intensity with a return period of 10 years. These are the 12-hour duration, 10-year return period rainfall found directly from a map in ARR and adjusted by a standard procedure, and the design duration. Errors in design rainfall are therefore related to errors in these variables and in the procedures in which they are used.

a) Errors in I(12,10)

The maps in ARR from which I(12,10) and other intensities are determined have been discussed in Chapter 5. In general, the absolute accuracy of these maps is not of major importance in the design method, as long as the same rainfalls are used in design as were used in deriving the runoff coefficients. Of much greater importance is the situation where an individual catchment is subject to appreciably different rainfall intensities to neighbouring catchments as a result of localised rainfall variations, and these are not accounted for by the smoothed contours in the maps of either rainfall or design runoff coefficients. Localised variations in rainfall generally result from topographic features, and may be increases due to local uplift of inflowing air, or decreases due to local rain shadows. These effects are likely to be greater on small than on large catchments where greater averaging of conditions occurs.

b) Errors in Design Duration

As illustrated by the sensitivity study in Appendix E, variations in the design duration cause considerable variations in rainfall intensity. Errors in the design duration would thus lead to errors in the derived values of C(10). Equation 4.2 for design duration, viz.

$$t_c = 0.76\ A^{0.38}$$

was derived as a best estimate of the characteristic minimum time of hydrograph rise for 96 catchments. Variations of catchment characteristics from the averages for these catchments would be likely to cause errors in the estimated design duration. This is more likely to occur on larger catchments, which are made up by different combinations of smaller catchments. Greater variability is thus likely on larger catchments, leading to greater probable errors in design duration for these catchments. However, the form of the relation between rainfall intensity and duration results in a smaller effect on rainfall intensity for the longer durations with large catchments. This compensating effect indicates that rainfall errors of similar magnitude are likely for small and large catchments, and that the errors are random.

K.2.3 Catchment area

Except for very flat regions where the divide may be difficult to define, neglibible errors are likely to result from the actual measurement of catchment area. However, several other types of errors discussed in Sections K.2.1 and K.2.2 above are likely to have more severe effects for either very small or large catchments compared with catchments of intermediate size. The general trends of the various types of error in the derived values of C(10) are summarised in Table K1.

K.3 EXAMINATION OF THE 24 STATIONS WITH EXTREME DEVIATIONS OF C(10) VALUES

K.3.1 Selection of stations

Stations were selected for examination where the derived 10-year runoff coefficient C_D differed considerably from the general trend of values in the vicinity of the catchment as indicated by the value of C_P predicted from the design map at the back of this report. The stations selected were in two groups. The first consisted on the twelve stations with the highest derived coefficients as measured by the ratio C_D/C_P. These are listed in Table K.2. The other twelve stations were those that had the lowest derived coefficients as indicated by high values of the ratio C_P/C_D. These stations are listed in Table K.3. Inverse ratios have been used for the high and low derived coefficients so that values of the ratios in both cases are greater than unity. A similar approach was used in Chapters 7 to 9 of the main body of the report, except that in that case deviations of the predicted coefficients relative to the derived values were under consideration. The deviations discussed in this appendix are thus in the opposite direction to those in Chapters 7 to 9.

Various factors that could account for the relatively high or low values of the derived coefficients are discussed in subsequent sections, and details are listed in Tables K.2 and K.3

K.3.2 Catchment area

Examination of Tables K.2 and K.3 indicates that there is no consistent trend of the extreme derived C(10) values with catchment size. In fact, the frequency distributions of the areas of the catchments with the twelve high and twelve low C(10) values are very similar. However, comparison of their areas with the frequency distribution of the areas of all of the catchments used in the study (Figure 1.1) shows that most of them lie at either end of the range, with only six of the twenty four within the most common size range of 50 to 250 km^2.

In the column for explanatory factors of deviations in Tables K.2 and K.3, the term "AREA" has been listed for those catchments with areas less than $1km^2$ and greater than 250 km^2, indicating a possible cause of the large deviations. The latter area was selected as the desirable upper limit of size of catchments to be used in the study and has also been nominated as the upper limit for application of the derived design method. As noted in Chapter 1, quite a few catchments above this limit were included in the study to indicate flood potential in regions where no other data were available.

K.3.3 Sampling error

As noted earlier, the analysis of sampling errors described in Section 8.1.4 showed that these could account for almost all of the deviations between the derived C(10) values and those predicted by the design method. All of the relatively low extreme values of the derived floods and coefficients could be accounted for in this way, but not all of the relatively high extreme values could be explained as sampling errors. However, this could result from the non-central t distribution assumed to apply to the errors. The justification for this assymetric distribution is rather obscure, as discussed in Section 8.1.4.

Sampling errors increase as record length decreases. In the column for explanatory factors of deviations in Tables K.2 and K.3, sampling error denoted by "SE" has been listed against those catchments with lengths of record less than 20 years. For record lengths above 10 years, this would constitute a rather weak source of explanation of the extreme deviations, but would be a major explanation with record lengths less than 10 years.

Table K.1 Cause and Direction of Errors in Derived 10-year Runoff Coefficients

Cause of error	Symbol used in Tables K.2 & K.3	Likely magnitude of maximum effect	Likely direction of error	Likely magnitude of error compared with that for catchment of intermediate size	
				very small catchment	large catchment
Sampling error	SE	Large	Random	Same	Same
Rating extension	RE	Large	Random	Same	Same
Storages	STOR	Medium	Low	Higher	Lower
Urbanisation	URB	Medium	High	Higher	Lower
Land use changes	LU	Small	High & low	Higher	Lower
Daily read stages	DR	Medium	Low to nil	Higher	Same
Rainfall intensity	RINT	Medium	High, random & low	Higher	Higher
Design duration	DRTN	Medium	Random	Same	Same

Table K.2 Characteristics of twelve Stations with highest derived values of C(10) relative to regional trends.

STATION NUMBER	RATIO OF DERIVED TO PREDICTED C(10) C_D/C_P	A (km^2)	N years	SDEV of logs	SKEW of logs	RATING RELIABILITY (Q_{10}/GGD)	DEVIATION - EXPLANATORY FACTORS (see Table K.1 for key)	DEGREE OF EXPLANATION
211006	2.9	8.8	10	.092	.31	3.7	SE; STOR; RINT; tidal backwater; high hydrograph peakiness-questionable rating.	strong
410067	2.1	220	26	.146	.29	16	RE; LU; RINT.	moderate
420009	2.0	331	11	.432	-.26	13	AREA; SE; RE; DR; possibly affected by Castlereagh backwater; high SDEV	strong
205003	2.0	13	10	.359	.92	15	SE; high SDEV; RINT; RE	moderate
410351	2.0	0.7	28	.284	.63	(Flume)	AREA; Possibly some grazing activity; RINT	weak
421053	1.8	202	9	.114	-.30	10	SE; RE; STOR; 20% of flood record missing	strong
210059	1.7	70	18	.300	.77	32	SE; RE; STOR; high hydrograph peakiness - questionable rating; dam construction 1968	strong
212333	1.5	0.70	24	.221	-.30	1.0	AREA; STOR; RINT	weak
422303	1.5	10	47	.529	.38	3.9	RINT; high SDEV;	moderate
214330	1.5	0.39	8.5	.247	.47	(Model)	AREA; SE; RINT	moderate
401508	1.5	5.0	10.8	.135	.42	25	SE; RE; RINT; Alpine; 21% of flood record missing	-strong
419016	1.4	907	32	.262	1.30	5.1	AREA	weak

Table K.3 Characteristics of twelve stations with lowest derived values of C(10) relative to regional trends.

STATION NUMBER	RATIO OF PREDICTED TO DERIVED C(10) C_P/C_D	A (km^2)	N years	SDEV of logs	SKEW of logs	RATING RELIABILITY (Q_{10}/GGD)	DEVIATION - EXPLANATORY FACTORS (see Table K.1 for symbol key)	DEGREE OF EXPLANATION
422317	2.6	466	20	.173	.57	1.4	AREA; DR; RINT	moderate
210069	2.1	4.9	13	.286	.03	5.2	SE; LU; RINT; high hydrograph peakiness	moderate
212301	2.1	0.06	14	.302	1.22	(weir)	AREA; SE; LU; RINT	moderate
419036	1.7	143	9	.473	2.26	8.4	SE; RE; high SDEV and SKEW	moderate -strong
217001	1.7	891	14	.249	.20	29	AREA; SE; RE; DR; RINT	strong
219008	1.7	18	9	.158	.27	18	SE; RE; RINT	strong
419029	1.6	389	11	.357	1.24	8.4	AREA; SE; RE; RINT; high hydrograph peakiness; high SDEV	moderate
210053	1.6	83	18	.279	.06	2.2	SE; STOR; URB; LU; 7.1% of flood record missing	moderate
214320	1.5	0.93	17	.144	.36	(model)	AREA; SE; RINT;	weak
214334	1.4	5.52	18	.203	1.39	(model)	SE; RINT	weak
410534	1.4	109	12	.223	.76	1.3	SE; STOR	weak
212306	1.4	0.14	14	.145	1.72	(Weir)	AREA; SE; LU; RINT	moderate - weak

K.3.4 Errors due to rating curve extension

Extension of the rating curve was necessary at most of the stations analysed, as noted earlier, and illustrated by Figure D.1, and is a major source of error. This has been noted as an explanatory factor in Tables K.2 and K.3 for those catchments with ratios of Q(10) to the greatest gauged discharge (G.G.D.) of greater than 5. Even those stations that were model rated involve some extension errors, as the model rating constitutes an extension of field calibrations, although involving much greater accuracy than conventional extension.

K.3.5 Effects of land uses

Various types of land use have been discussed earlier in this Appendix. Their effects on runoff and peak floods are of uncertain magnitude and of different directions. However, they may cause differences in the flood characteristics of the catchments affected relative to those of other catchments in the region, especially if the affected catchments are of small size. Those catchments which have that are not typical of their region are noted in the 'explanatory factors' columns of Tables K.2 and K.3.

K.3.6 Localised variations in rainfall intensity

For small catchments in areas of high relief, local variations in rainfall intensity that are not represented on the generalised maps in ARR may cause variations in flood characteristics. The greater hydrological variability of small catchments is discussed by Baron et al. (1980). Little definite information is available on this source of variation, but small catchments where some effect of this type could be present are noted in Tables K.2 and K.3.

K.3.7 Standard deviation and skewness

The standard deviation (SDEV) and the skew coefficient (SKEW) of the logarithms of the N highest floods, together with the average value, are used to fit the LP3 flood frequency distribution for each station. Very high or low values of these statistics will thus lead to high or low values of Q(10) and C(10). Catchments with extreme values of SDEV and SKEW are noted in Tables K.2 and K.3. However, the fact that these values are high or low does not really provide an explanation for the extreme values of Q(10) and C(10).

K.3.8 Summary

The possible explanations for the extreme deviations of C(10) values for the twenty-four catchments are summarised in Tables K.2 and K.3. The subjectively assessed degree of explanation is listed in the last columns of the tables. The reliability class of each station (Appendix D) was considered in making these assessments. For only six of the catchments, the degree of explanation is strong, while the deviations are only weakly explained for six catchments. Overall, it does not seem that the extreme deviations can be adequately explained in terms of physical characteristics. Two factors seem to be of greatest importance. The first is the extension of rating curves, which was required at almost all stations. The large extensions at many stations would be a major source of errors. Sampling errors in the flood series, the second factor, are probably the dominant factor. The previous analysis of sampling errors discussed in Section 8.1.4 indicated that most of the deviations could be accounted for in this way.

The lack of strong explanation for most of the extreme deviations seems to confirm the fact that sampling errors are a major source of the deviations.

The extreme deviations are thus to be expected in the large sample of catchments analysed, and there is no good reason to reject them. Reduction of the deviations could only be achieved by a substantial increase in the lengths of the records analysed.

APPENDIX L EXAMINATION OF THE DEPENDENCE OF RUNOFF COEFFICIENTS ON LOCATION

In Chapters 6 and 8, the concept was adopted that both the 10-year and Y-year runoff coefficients depend only on geographical location. This was not a fundamental premise adopted at the start of the study, but rather resulted from analysis of the large amount of recorded data. The effect of scatter and sampling error in the original flood data was such that significant relationships between the runoff coefficients and other physical variables could not be found.

In this appendix, the validity of the dependence of runoff coefficient values on location is re-examined by analysis of the data from a different approach to that used in Chapters 6 and 8. In that this analysis is a re-working of the original data, it is not an independent check on the original analysis. However, the results provide support for the original conclusions and the approach adopted for presentation of the design data.

L.1 RESTATEMENT OF FORMULA FOR DERIVATION OF RUNOFF COEFFICIENTS

The runoff coefficients derived in this project were calculated from equation (1.2) with the appropriate factor of proportionality for the units used. This equation is

$$C(Y) = Q(Y) / 0.278 \times A \times I(t, Y) \qquad \text{(L.1)}$$

where the symbols are defined in Sections 1.2 and 1.4.1. To examine the validity of the concept that runoff coefficients are primarily dependent on location, this equation will be restated in several different forms based on relationships developed from the basic data used in the study.

Peak flood flows Q(Y) of return period Y years were calculated from the log-Pearson Type III distribution, which is stated in the form of equation (2.2) for application. As the distribution was fitted to the logarithms to base 10 of the flood flows, the term X(Y) of equation (2.2) can be replaced by log Q(Y) and the equation rewritten as

$$\log Q(Y) = M + SDEV.\ K(G,Y) \qquad \text{(L.2)}$$

where the terms are defined in Section 2.1.4.

Taking anti-logarithms,

$$Q(Y) = 10^{M+SDEV.\ K(G,Y)} \qquad \text{(L.3)}$$

The rainfall intensity - frequency - duration data were calculated using procedures in ARR for the rainfall maps such that:

$$I(t,Y) = I(12,Y) \times MF(t,Zone) \qquad \text{(L.4)}$$

where

I = intensity (mm/h)
Y = return period (years)
t = duration (hours)
MF = multiplying factor
Zone = ARR rainfall zone (Figure 2.17 of ARR)

This applies to durations of 6 minutes to 12 hours, which from equation (4.2) correspond to catchment areas of 0.005 to 1400 km^2. In the above equation the I(12,Y) term is calculated using the rainfall parameters extracted from the maps in ARR Figures 2.18 and 2.19 for return periods of 2 and 50 years, and these are entered in Figure 2.46 to determine values for other return periods. These extracted parameters are functions of geographical location (latitude and longitude). Figure 2.46 in ARR can be used to calculate I(12,Y) in the form below:

$$I(12,Y) = I(12,2) + \alpha \times (I(12,50) - I(12,2)) \qquad (L.5)$$

where

I(12,2)and I(12,50) are the intensities extracted from ARR rainfall maps and α is a function of return period (see Table L.1 below).

Table L.1 Multiplying Factors Representing ARR Figure 2.46

Return Period Y (Years)	2	5	10	20	50	100
α	0	0.309	0.471	0.734	1	1.236

The rainfall intensity of one year return period and 12 hour duration is also a function of I(12,2) and I(12,50) (see Appendix H). Thus for any particular return period I(12,Y) is purely a function of geographical location.

The multiplying factor (α) used in equation (L.5) above is a function of duration and rainfall zone. As duration, in the design method here developed, is related uniquely to catchment area by equation (4.2), this multiplying factor is effectively a function of catchment area and rainfall zone. By plotting this multiplying factor against catchment area for each zone, it was found that with a maximum error of 5%, the relationship for each zone could be approximated by a power function of the form

$$MF = k\,A^n \qquad (L.6)$$

where K and n are parameters. The values of these parameters for the various rainfall zones are presented below in Table L.2.

Table L.2 Parameters of Power Function Relations Between Rainfall Intensity Multiplying Factors and Catchment Area

(a) Duration t = 6 to 60 minutes

ARR Rainfall Zone	k	n
A	4.30	-0.19
B	5.71	-0.19
C	4.72	-0.19
D	6.82	-0.19
E	6.64	-0.19

(b) Duration t = 1 to 12 hours

ARR Rainfall Zone	k	n
A	4.09	-0.20
B	5.59	-0.24
C	4.47	-0.21
D	7.00	-0.27
E	6.64	-0.26

From equation (4.2), rainfall durations of 6 minutes, 1 and 12 hours correspond to catchment areas of 0.005, 2 and 1400 km^2 respectively, so that (a) and (b) in Table L.2 correspond to these ranges of area.

By substituting the relations of equations (L.4) and (L.6), equation (L.1) can then be rewritten for any particular return period Y as

$$C(Y) = CON(Y) \times \frac{Q(Y)}{A^{(1+n)}} \qquad (L.7)$$

where $CON(Y) = (0.278 \times I(12,Y) \times k)^{-1}$ and is a constant for the location and return period.

By replacing Q(Y) by equation (L.3), this can also be expressed as

$$C(Y) = CON(Y) \times \frac{10^{(M+KS)}}{A^{(1+n)}}$$

or

$$C(Y) = \frac{\beta \times 10^{(M+KS)}}{I(12,Y) \times A^{(1+n)}} \qquad (L.8)$$

where $\beta = (0.278 \times k)^{-1}$ and is a constant for a zone.

L.2 IMPLICATIONS FOR DEPENDENCE OF C ON LOCATION

If the concept that the value of the runoff coefficient depends solely on location is true, then for two adjacent catchments (called U and V), their runoff coefficients for any particular return period should be the same.

That is,

$$C_U = C_V$$

where $$C_U = CON_U \times \frac{Q_U}{A_U^{(1 + n_U)}}$$

and $$C_V = CON_V \times \frac{Q_V}{A_V^{(1 + n)}}$$

As U and V are adjacent,

$$CON_U = CON_V$$

Further, if both A_U and A_V are between 2 and 1400 km^2 or between 0.005 and 2 km^2, then

$$n_U = n_V = n$$

and $$\frac{C_U}{C_V} = 1 = \frac{Q_U}{Q_V} \times \left(\frac{A_V}{A_U}\right)^{(1 + n)}$$

or $$Q \propto A^{(1+ n)} \qquad \text{(L.9)}$$

The case where A_U is above and A_V is below $2km^2$ is considered later. The majority of the gauged catchments used in this study had catchment areas between 2 and 1400 km^2. Only eleven were smaller than $2km^2$, and twelve were larger than 1400 km^2. To disprove the adopted concept that the value of the runoff coefficient depends primarily on location therefore can be viewed as disproving equation (L.9).

To investigate the validity of this equation, regression and correlation studies were carried out of flood magnitudes of each return period and various physical variables. The best relationships were obtained with logarithmic multiple regressions of Q(Y) on the three variables, area A, a representative rainfall intensity (I(12,Y)), and median annual rainfall (MAR). The parameters and other statistics of these best relations are listed in Table I.6, Appendix I. As I(12,Y) and MAR are location dependent, these relationships for a given location can be expressed as

$$Q(Y) = KON(Y) \cdot A^a \qquad \text{(L.10)}$$

where KON(Y) = a constant depending on the location, and is a function of I(12,Y) and MAR (see Table I.6, Appendix I)

A = catchment area - km^2

a = an exponent (see Table I.6, Appendix I)

The inclusion of other physical variables in the regression caused very little or no improvement in the relation. This may result from the fact that several variables that would not be expected to depend on location do in fact seem to be correlated with location dependent variables (see Tables I.12 and I.13, Appendix I), possibly due to the areal spread of the basic data or some bias in the small catchments that are gauged.

Equation (L.10) provides a means of empirically checking the validity of equation (L.9).

L.3 10-YEAR RUNOFF COEFFICIENTS

The 10-year return period is of particular interest as these coefficients are mapped as the basic design data of the procedure. From equation (L.10) and the value of a in Table I.6, Appendix I.

$$Q(10) = KON(10) \ . \ A^{0.74} \qquad (L.11)$$

For the two adjacent catchments U and V, KON(10) would have the same value, and thus

$$Q(10) \propto A^{0.74}$$

For catchments with areas between 2 and 1400 km^2, this exponent of 0.74 agrees closely with those of equation (L.9) given by the values of n in Table L.2, considering the expected magnitude of the sampling error (Table 8.1). The exponents in equation (L.9) are 0.80, 0.76, 0.79, 0.73 and 0.74 for zones A to E respectively. This agreement thus supports the concept of location dependence of 10-year runoff coefficients for catchments larger than 2 km^2.

If the adjacent catchments U and V are both smaller than 2 km^2, the values of n in Table L.2 give the exponent in equation (L.9) as 0.81 for all zones. This is significantly higher than the value of 0.74 from equation (L.11). However, the exponent of power function relations between flood magnitude and catchment area has generally been found to increase as catchment area decreases. Probably the best-known examples are the relations of Creager et al. (1939). As the exponent of 0.74 in equation (L.11) was derived from catchments that were generally much larger than 2 km^2, the difference in the two exponent values is consistent with accepted relationships. This again confirms the location dependence of runoff coefficients for areas below 2 km^2.

The remaining case is where one catchment U is above 2 km^2, and the other V is below this limit. In this case the equations leading to equation (1.9) must be modified as n_U and n_V are not equal, resulting in

$$\frac{Q_U}{Q_V} = \frac{k_U \ A_U^{(1 + n_U)}}{k_V \ A_V^{(1 + n_V)}} \qquad (L.12)$$

To obtain some comparison of this relation with equation (L.11), values of 100 and 0.1 km^2 were assumed for A_U and A_V. These are about the middle of the area ranges above and below 2km^2. Using the parameter values from Table L.2, the ratios Q_U/Q_V were calculated for each of the five zones. The exponent of a simple power relation between Q and A was then found that would give the same value of Q_U/Q_V. For the five zones, this exponent was in the range 0.76 - 0.80, which as expected is between the values for both catchments above and below the limit of 2 km^2. Considering the small value of A_V and the expected sampling errors, this range is again in good agreement with the exponent of 0.74 in equation (L.11).

Thus for all ranges of catchment size of interest, this investigation supports the concept of 10-year runoff coefficient values depending primarily on location. Lack of other trends reflects the fact that no significant relations could be found for the deviations of derived C(10) values from the mapped contours with any physical variables.

L.4 RETURN PERIODS OTHER THAN 10 YEARS

As described in Section 8.3, the procedure adopted for specifying design values of the frequency factor C(Y)/C(10) was to derive average values for each of three zones. These design frequency factors thus also depend on location, and the validity of this assumption was also investigated further.

From equation (L.8), for a given catchment,

$$\frac{C(Y)}{C(10)} = \frac{\beta \times 10^{M + K' SDEV}}{A^{(1+n)} x I(12,Y)} \Bigg/ \frac{\beta \times 10^{M + K SDEV}}{A^{(1+n)} x I(12,10)}$$

where M, SDEV, K and K' are statistics of the logarithms of the highest 'N' flood peaks observed and

N = record length in years
M = mean of the logarithms
SDEV = standard deviation of the logarithms
K = a function of skewness coefficient, for the 10-year return period
K' = a function of skewness coefficient, for the Y-year return period
β = $(0.278 \times k)^{-1}$ where k is given in Table L.2
n = a constant for the zone, see Table L.2.

In simplified form,

$$\frac{C(Y)}{C(10)} = 10^{(K' - K)SDEV} \times \frac{I(12,10)}{I(12,Y)} \qquad (L.13)$$

This provides a basis for examining the factors affecting C(Y)/C(10). For a given return period Y, the ratio I(12,10)/I(12,Y) is dependent on location only as its terms are extracted from maps in ARR. SDEV, K and K' however are dependent on the observed flow data for the station. For a particular return period Y, the variation in the term $10^{(K' - K)SDEV}$ between stations is dependent on the variation in the statistical parameters standard deviation (SDEV) and skewness coefficient (G) of the logarithms of observed flood event magnitudes between stations. Due to sampling error these statistical parameters are subject to considerable errors in estimation (see ARR, Chapter 9), so that attempts to relate values of these parameters with other variables should result in low correlation coefficients and relationships of low statistical significance. A regression and correlation study was carried out however which revealed that of the factors tested, standard deviation was significantly related to MAR, the median annual rainfall (Table I.14, Appendix I). The correlation coefficient was quite low, accounting only for 4% of the between station variation in SDEV. Standard deviation thus would seem to be either location dependent (as MAR is location dependent) or a constant average value.

With a lesser degree of significance than for the correlation of SDEV and MAR, each of three factors was found to be significantly correlated with skewness coefficient G with a probability below 5% that the relationship was due to chance (see Table I.15, Appendix I). These factors were median annual rainfall (MAR), the ratio of the 12-hour, 50-year to the 12-hour, 2-year rainfall intensity (IR), and the ratio of average to equal area stream slope (S_a/S_e) as an index of slope non-uniformity. Both MAR and IR are location dependent variables. The variable S_a/S_e however would seem to be location independent. In fact this variable was found to be significantly related (Table I.12, Appendix I) to both rainfall intensity and catchment area, the former of which was considered location dependent and the latter location independent.

The above discussion reflects the problem encountered in Section 8.3 in selecting the design approach to be adopted. The frequency factors C(Y)/C(10) are largely dependent on location, but are also dependent to some extent on other physical variables. This resulted in the equally acceptable alternative approaches of average values within zones, or relationships with physical variables. The analysis in this appendix confirms these results. It also supports the approach adopted that location can be considered the primary variable affecting the frequency factors.

L.5 SUMMARY OF DEPENDENCE OF C ON LOCATION

The investigations described in this appendix have examined the design approach adopted in this project that the value of the runoff coefficient can be specified in terms of location. The investigation really constitutes a reworking of the data on which the original approach was based, and is thus not an independent check. However, the results provide support for the adopted approach that the values of both C(10) and C(Y)/C(10) depend primarily on location, and that with the available data, this provides a satisfactory means of specifying the design data.

APPENDIX M A SIMPLE RELATION BETWEEN RETURN PERIOD AND FLOOD MAGNITUDE

In examining the economic significance of the design procedure in Chapter 9, costs were expressed as a function of return period. Cost differences resulting from differences in the estimated flood magnitude could therefore be determined from the differences in the return period corresponding to these flood estimates. For example, an estimate of the Y-year flood using the generalised design data might be Q_e, while the flood data observed at the site might indicate that the actual value should be Q_a. From the observed frequency curve for the site, Q_e might correspond to the Y'-year flood rather than the Y-year flood. If the value of Y' could be obtained, the difference in costs associated with the difference between Q_e and Q_a could be determined from the relation between costs and return period. To facilitate this, a simple empirical relation between flood magnitude and return period is developed in this appendix.

The desired form of the simple relationship was linear such that:

$$Q(Y) = m \times f(Y) + b \qquad (M.1)$$

where $Q(Y)$ = flood magnitude in m^3/s

y = return period in years

$f(Y)$ = some function of Y

and m and b are parameters

A power function relationship was selected as suitable for f(Y) such that:

$$f(Y) = Y^n \qquad (M.2)$$

The exponent n was assigned a value by trial and error fit to the derived flood frequency curve for a given station. Equation (M.1) then became (for each station):

$$Q(Y) = m(Y^n) + b \qquad (M.3)$$

with n known for each station.

The slope (m) of this straight line relation could then be calculated as

$$m = \left(\frac{Q_{YM} - Q_1}{Y_M^n - 1}\right) \qquad (M.4)$$

where Y_M = maximum return period to which the station flood frequency curve was extended.

The value of m depends on the magnitude of floods recorded at the particular station. To make m comparable between stations a standardized value m' was obtained by dividing m by the flood magnitude Q_2 of return period of 2 years for that station. That is,

$$m' = m/Q_2 \qquad (M.5)$$

The exponent n was then calculated by trial and error for the flood frequency curves from each of 172 stations selected as a suitable sample in the time available of all of the study catchments. Similarly the parameter m' was calculated for each station. This latter parameter was found to vary only a small amount between stations such that its overall statistics were:

$$\bar{m}' = 0.714 \quad \text{(mean)}$$

and $$\sigma = 0.075 \quad \text{(standard deviation)}$$

Larger variations were observed between stations for the exponent n such that

$$\bar{n} = 0.443 \quad \text{(mean)}$$

and $$\sigma = 0.151 \quad \text{(standard deviation)}$$

The mean value of m' (0.714) was then adopted as a constant. Given the observed flood frequency curve for a station, the exponent n could then be calculated as below by rearrangement of equations (M.4) and (M.5):

$$n = \log\left[\left(\frac{Q_{Y_M} - Q_1}{Q_2 \times 0.714}\right) + 1\right] \Big/ \log(Y_M) \qquad \text{(M.6)}$$

By substituting equation (M.5) into (M.3), using the mean value of m' of 0.714, and by considering a return period Y of 2 years with flood peak of Q_2, the value of b in equation (M.3) can be evaluated as

$$b = Q_2 - 0.714\, Q_2\, (2^n) \qquad \text{(M.7)}$$

Substituting this and equation (M.5) into (M.3) gives the relation between Q(Y) and Y as

$$Q(Y) = Q_2 \left[0.714\, Y^n + 1 - 0.714\, (2^n)\right] \qquad \text{(M.8)}$$

Considering the estimated flood peak Q_e as the value of Q(Y) and its corresponding actual return period Y' years, equation (M.8) can be re-arranged to give the relation between Y' and Q_e for a particular station as

$$Y' = \left[2^n + 1.4\left(\frac{Q_e}{Q_2} - 1\right)\right]^{\frac{1}{n}} \qquad \text{(M.9)}$$

where Y' = the actual return period from the observed flood frequency curve appropriate for the estimated Y-year return period flood magnitude (in years)

Q_e = estimated Y-year return period flood magnitude calculated from a flood estimation method for the station (in m^3/s)

Q_2 = 2-year return period flood magnitude from the observed flood frequency curve (in m^3/s)

n = exponent as calculated using equation (L.6) from the observed flood frequency curve for the particular station. As noted, values varied from about 0.3 to 0.6 with a mean of approximately 0.45 for the 172 stations analysed.

APPENDIX N CALCULATION OF APPROXIMATE COSTS RESULTING FROM INACCURACIES IN DESIGN FLOOD ESTIMATES

N.1 EFFECT OF INACCURACIES

The general form of the relationship between total costs and design return period for a given structure is illustrated in Figure 9.3. If the costs (both capital and damages) were known for each discharge, and the true relation between discharge and return period was known, the most economic return period and design discharge could be selected for the structure. However, Section 9.1 demonstrates that discharges of a given return period estimated from design methods often involve considerable errors, especially where the design method is not based on observed data.

The apparent costs of the structure will reflect the erroneous discharge estimates. For a given return period, over and underestimates of discharge will cause over and underestimates of the apparent values of both capital costs and expected damages. The apparent optimum return period may not be greatly different from the true value.

Despite the above discussion of apparent costs, both over and underestimates of the true optimum discharge will result in increases in real costs compared with the optimum. This shown by Figure 9.3, which implies that total costs increase for discharges above or below the optimum. The effect is likely to be greater for underestimates. In this appendix, a method is developed for estimating these increases in real costs for a structure resulting from inaccuracy of estimates of the design flood. For convenience, it is assumed that the true optimum return period is 10 years, which is an approximate average design value currently used in Australia for minor bridges and culverts (Pilgrim and Cordery 1980). The method is applied to the majority of the catchments used in this report. Estimates are made of increases in costs resulting from the inaccuracies in the design method in ARR and in the method derived in this study. It is assumed in the analysis that the flood frequency curve derived from the observed floods on each catchment gives the true discharges. The results indicate the potential savings of the design method presented in this report.

As discussed below, many assumptions were required in the analysis. Consequently, the results can only be considered as approximate. However, they do indicate the magnitude of the costs involved and enable comparison of the design methods.

N.2 DATA ON COSTS

A major problem in economic analyses of structures on small waterways is the obtaining of data on both capital costs and expected damages. Data on capital costs of small bridges and culverts was supplied by the Department of Main Roads of N.S.W. Costs were given of seventeen typical structures with design discharges ranging from 50 to 1000 m^3/s. A plot of capital costs against design discharge showed considerable scatter, as many variables affect the cost of a bridge and its capacity to pass flood flows. These include the allowable velocities and afflux at the bridge, the stream cross-section, the overall size of the bridge, its skew, the type of superstructure and the types of piers and abutments. Despite the scatter in the values, the relationship gives the order of capital costs, and is considered to be valid for use in the semi-quantitative analysis described here. The relationship was approximately linear over most of the range of values. A linear regression gave the relation

$$CC\$ = 279\ Q(Y) + 118\ 000 \qquad (N.1)$$

where CC\$ = capital cost in dollars at December 1979
and $Q(Y)$ = design discharge (m^3/s) of return period Y years.

The correlation coefficient was 0.69 and the standard error of estimate was \$80 000. The correlation is significant at the 0.2% probability level. This linear relation was inadequate at low discharges below about 100 m^3/s, and it is obvious that the actual relation must pass through the origin. A second relation for low discharges was derived using the following three sources of information;-

(i) costs of the small bridges from the information supplied by the Department of Main Roads and discussed above;

(ii) cost estimates of box culverts and pipe drains given by Polin (1978); and

(iii) cost estimates for various types and combinations of box culverts using cost information from the Department of Main Roads and discharges calculated by a formula given by Henderson (1966, p. 263).

An approximate straight line of best fit to the data from these sources and passing through the origin intersected the relation of equation (N.1) at a discharge of 100 m^3/s. This relation, adopted for the discharge range 0 - 100 m^3/s, is

$$CC\$ = 1460\ Q(Y) \qquad (N.2)$$

Equation (N.1) was adopted for discharges above 100 m^3/s.

Less information is available on costs of expected damages. The only relevant information that could be found was two case studies described by Polin (1978) and summarised by Polin and Cordery (1979). Logarithmic plots of damages against return period for Polin's two examples gave straight lines with the same slopes. A linear relationship of the logarithms was thus adopted for damage costs of the form

$$\log (D\$) = K \log Y + b \qquad (N.3)$$

where D\$ = present worth of expected damage costs in dollars at December 1979 for a structure designed to pass the Y-year flood.
Y = return period of design flood (years)
K = a constant, the slope of the linear relationship of the logarithms
b = a parameter with value dependent on the particular site.

For Polin's (1978) two examples, the value of K was – 0.96. This was rounded to –1.0, and taking antilogs of equation (N.3) than gave

$$D\$ = \frac{B}{Y} \qquad (N.4)$$

where B = a parameter equal to 10^b

N.3 VARIATION OF COSTS WITH DISCHARGE ESTIMATES

Combination of equation (N.4) with either (N.1) or (N.2) provides a means of estimating the total cost of a structure for any selected return period. As described below, the parameters can then be selected so that the optimum or minimum cost occurs at a return period of 10 years, which was arbitrarily adopted in this study as discussed earlier. The relation between cost and return period then allows evaluation of the cost of a structure designed for a discharge different to the optimum value. If the true return period of an inaccurate estimate of the 1-year discharge is known, the above relation allows evaluation of the additional costs resulting from the inaccurate discharge estimate. A simple relationship for determining the true return period of estimated discharge was derived in Appendix M and is given as equation (M.9).

The relation between total costs and return period will be considered separately for discharges greater than 100 m^3/s, and from 0 - 100 m^3/s.

Case 1 - Discharges greater than 100 m^3/s

Combination of equations (N.1) and (N.4) gives the total cost TC\$ of a structure in December 1979 dollars as

$$TC\$ = 279\ Q(Y) + 118\ 000 + \frac{B}{Y} \tag{N.5}$$

A minimum total cost will occur when

$$dTC\$/dY = 0$$

that is, $279\ \frac{dQ(Y)}{dY} - BY^{-2} = 0$ (N.6)

A relation between Q(Y) and Y was derived in Appendix M as equation (M.8), viz.

$$Q(Y) = Q_2\ \left[0.714\ Y_n + 1 - 0.714(2^n)\right]$$

where n = an exponent derived from the flood frequency curve for a given station. Differentiating this and substituting into (N.6),

$$279\ Q_2(0.714\ n\ Y^{n-1} - BY^{-2} = 0$$

Hence $B = 200\ n\ Q_2\ Y^{n+1}$

Thus the total cost TC\$ will be a minimum at the particular value of Y when B has this value. For the adopted optimum return period of 10 years,

$$B = 200\ n\ Q_2\ .\ 10^{n+1}$$

Substitution into equation (N.5) gives

$$TC\$ = 279\ Q(Y) + 118000 + 200\ n\ Q_2\ .\ 10^{n+1}/Y \tag{N.7}$$

This relation enables the estimation of the total costs of structures designed for discharges corresponding to actual return periods differing from the optimum value of 10 years.

Case 2 - Discharges 0 - 100 m^3/s

Combination of equations (N.2) and (N.4) and analysis similar to that for Case 1 lead to the following expression for total costs for this range of discharges:-

$$TC\$ = 1460\ Q(Y) + 1040\ n\ Q_2\ 10^{n+1}/Y \tag{N.8}$$

N.4 COMPARISON OF COSTS WITH DIFFERENT DESIGN METHODS

Estimates of 10-year floods were carried out in Section 9.1 for 271 catchments by means of the design method in ARR and by the method developed in this report. For each method, these estimates generally deviated from the true value of the 10-year flood adopted here as being the discharge estimated from the frequency curve of observed floods for the particular catchment. The true value of the return period Y' corresponding to the estimated value Q_e of the 10-year flood was calculated for each flood estimate and each catchment by equation (M.9) of Appendix M.

$$Y' = \left[2^n + 1.4\left(\frac{Q_e}{Q_2} - 1\right)\right]^{\frac{1}{n}}$$

where Q_2 = 2-year return period flood magnitude from the observed flood frequency curve (m^3/s)

and n = exponent calculated using equation (M.6) from the observed flood freqeuncy curve for the particular station.

This value of Y' was then used as the value of Y in either equation (N.7) or (N.8) depending on the value of discharge, and the total cost of the structure to pass the estimated discharge Q_e was calculated. These costs were calculated for the 10-year flood derived from the observed flood frequency curve, here considered as the true optimum value, and for the 10-year floods estimated by the ARR procedure and by the method developed in this project.

The complete estimates were only made for those catchments where all three discharge estimates were either below or above 100 m^3/s. This simplified calculations and provided data for 174 of the total 271 catchments. The 174 catchments constituted a sufficiently large sample to indicate the comparative costs of the different design methods.

Based on the true values of the optimum 10-year floods estimated from the frequency curves of observed floods, the sum of the total costs of the structures for the 174 catchments was approximately $34 million. Using the design data in ARR, the total additional costs resulting from the inaccuracies of the estimated discharges were $15 million. The corresponding figure for the design procedure developed in this project was $2 million. Table N.1 expresses these basic and additional costs in terms of average values per km^2 of catchment area and per m^3/s of the observed flood frequency discharge. While such unit measures may be oversimplified in that costs are not linear functions of catchment size or discharge, these measures provide useful and convenient indicators.

Table N.1 Average costs of structures sized by different design methods

Design Approach	Cost per km^2 of catchment area	Cost per m^3/s of observed flood frequency discharge
Basic cost of structure sized by true 10-year flood	$1500	$790
Additional costs-design by ARR	660	350
Additional costs-design procedure of this project.	80	40

As discussed in Chapter 9, the results are biased to some extent in that the costs were estimated for some of the catchments used for the derivation of the design data for the procedure developed in this project. The results for this procedure should thus be better than would be expected if catchments not used in the development of the design data had been used. In addition, the entire analysis is only of an approximate nature, as a result of the assumptions that were necessarily involved. Despite these difficulties, the general orders or magnitude of the costs in Table N.1 indicate that the design procedure developed in this project constitutes a major advance on the procedure in ARR. The additional costs resulting from the inaccuracy of the ARR procedure are approximately half of the optimum costs, and demonstrate that the procedure leads to a large wastage of public funds. The procedure of this report that is based on observed data not only is of much greater accuracy, but also involves only small costs above the optimum values. Its use in eastern New South Wales in place of the design method in ARR would yield large financial savings.

APPENDIX O RUNOFF COEFFICIENTS FOR THE CANBERRA REGION

After completion of the project and the final preparation of the body of this report, derived runoff coefficient values became available for twenty four catchments in the Canberra region. These were derived by Mr. J.F. Neal of the Department of Housing and Construction in a project as part of a Master of Engineering Science degree at the University of New South Wales supervised by the second author. Fifteen of the catchments are gauged by the Department of Housing and Construction, and the remaining nine are gauged by the Division of Forest Research, C.S.I.R.O. Data for these Canberra catchments were not available for the study described in this report. The derived runoff coefficient values are reported and described by Neal (1981).

Very little information was available in the vicinity of Canberra in the project carried out by the authors, and the isopleths of C(10) had to be estimated in this region with considerable uncertainty. The values derived by Neal (1981) were rather different to the estimated isopleths. The isopleths on Maps 2 and 4 at the back of this report have therefore been amended to take account of Neal's derived C(10) values in the Canberra region. Interim versions of the map of C(10) values prepared before this late amendment will thus give incorrect values in the Canberra region.

There is a pocket of very low C(10) values in the Cotter River catchment to the south-west of Canberra with several derived values less than 0.1. Some of the gauged catchments in this region are in exotic pine plantations with a deep litter cover on the ground. The high losses and delaying effect on runoff that would be expected under these conditions could account for the low values of C(10). However, other low values apply to catchments with eucalypt forest cover, and the low values are not really inconsistent with the general trends in the Canberra region. Despite this, values of C(10) below 0.1 have not been recommended for use in design on Map 4.

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