ADDENDUM SHEET

p2 4th paragraph: "frequency of floods" should be "magnitude of floods"

p6 6th paragraph: "lead to" should be "led to"

p11 1st paragraph: "lead to" should be "led to"

p11 2nd paragraph: "increase infinitum" should be "increase ad infinitum"

p15 3rd paragraph: "lead to" should be "led to"

p18 9th paragraph: "a AEP" should be "an AEP"

p19 7th paragraph: "Although is" should be "Although it"

p19 7th paragraph: "greater that": should be "greater than"

p19 8th paragraph: "highest AEP" should be "lowest AEP"

p20 3rd paragraph: "higher the AEP" should be "lower the AEP"

p26 5th paragraph: "a uncertain" should be "an uncertain"

p27 4th paragraph: "the risks" should be "the risk"

p27 5th paragraph: "the risk which is willing to be accepted by the community" should be "the risk which the community is willing to accept"

p47 7th paragraph: "PMF" should be "PMP"

p81 4th paragraph: "Section 6.1.2.3." should be "Section 6.1.2."

p94 3rd paragraph: "Equation 5.5" should be " Equation 6.5"

p97 3rd paragraph: "a optimal" should be "an optimal"

p106 6th paragraph: "probability of the design life" should be "probability for the design life"

p109 1st paragraph: "considered to robust" should be "considered to be robust"

p116 4th paragraph: "it is believed that this accounts for the April 1889 event" should be "this is acceptable as it better accounts for the 1889 event"

p133 1st paragraph: "the best fit of the two" should be "the better fit of the two"

p134 2nd paragraph: "provided the best fit" should be "provided the better fit"

p143 2nd paragraph: paragraph should be "The Bureau of Meteorology was unable to provide estimates of longer duration PMP using the GSAM within the time frame of the research."

p150 4th paragraph: "BoM (1991)" should be "BoM (1991a)"

p153 3rd paragraph: "BoM (1991)" should be "BoM (1991a)"

p168 4th paragraph: "the critical location" should be "the critical duration"

C-6 "Table C.4 Peak Flows at Clarendon Weir" should be "Table C.5 Peak Flows at Old Noarlunga"

G-7 2nd paragraph: "a input" should be "an input"

G-8 "Figure G.1" should be "Figure G.2a"

G-8 "Figure G.2" should be "Figure G.2b"

Extreme Flood Estimation for the Onkaparinga River Catchment

by

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Abstract

The methods used for determining extreme floods were critically examined in an attempt to reduce the large uncertainties associated with their estimation. The Onkaparinga River Catchment and the Mt Bold Dam in South Australia were used in the analysis.

The sensitivity of the probable maximum flood (PMF) estimate to the choice of various parameters was tested. It was found that the PMF estimate was particularly sensitive to the model non-linearity and the choice of uniform or spatially varying rainfall.

The RORB model was used to determine the PMF from an estimate of the probable maximum precipitation (PMP). The model was calibrated using recorded events by the method of sensitivity fitting. This method considers the errors associated with a choice of model parameters for a particular size flood. The optimal choice of model parameters was used in the calculation of the PMF.

The inflow frequency curve for Mt Bold Reservoir was determined from an extended data set derived from modified downstream records prior to the construction of the dam. The outflow frequency curve was calculated considering the joint probability of inflow and initial reservoir level. In order to rout the floods through the reservoir, a new spillway rating curve for Mt Bold was developed.

The best estimate of the outflow PMF for the Mt Bold Reservoir was 9,300 m^3/sec from a 4 hour duration PMP. The inflow PMF was determined to be 10,200 m^3/sec from a 3 hour duration PMP.

Both high and low bounds of a reasonable estimate of the PMF were calculated. The high and low bounds of the inflow PMF were 12,400 m^3/sec and 5,600 m^3/sec respectively. The high and low bounds of the outflow PMF were 10,800 m^3/sec and 5,200 m^3/sec respectively.

Statement of Originality

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

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

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Chapter 1

Introduction

1.1. Need for Research

The 1980s witnessed an increasing awareness of extreme floods. Greater understanding of the physical processes involved in the production of extreme storms and the recording of some extreme storm events led to increases in the estimates of probable maximum precipitation (PMP). New modelling techniques such as non-linear runoff routing were also developed throughout this period.

The adoption of new modelling techniques and the increases in the PMP led to increases in the probable maximum flood (PMF). The increases in the estimates of the PMF had an important effect on the perceived safety of existing dams. In some cases expensive remedial works were undertaken to increase the spillway capacities of existing dams.

Although there has been large expenditure on spillway upgrades, the methods used to derive extreme floods are still subjective and are somewhat arbitrary. This has resulted in inconsistencies in the estimates of extreme floods. There was a need to examine the procedures used to derive extreme flood estimates, in order to remove the observed inconsistencies.

In South Australia, there was an inconsistency between the PMF estimates for Mt Bold and Kangaroo Creek catchments (Doherty, 1992, pers. comm.). These adjoining catchments have similar areas and are in similar orographic situations. Mt Bold Dam has a catchment area of 388 km^2 and the PMF was estimated to be approximately 2,800 m^3/sec (Kotwicki, 1984). Kangaroo Creek Dam has a catchment area of 342 km^2 and the PMF was estimated to be approximately 5,000 m^3/sec (Water Resources Branch, 1981).

1.2. Study Objectives

The objective of the research was to critically examine the methods used to estimate extreme floods and to determine the best estimate of extreme floods, including the PMF, for the Onkaparinga River in South Australia.

The sensitivity of the PMF estimate was to be determined for the location of the isohyetal pattern, the addition of baseflow, the initial storage level, the model non-linearity and the choice of losses.

1.3. Methodology

Extreme floods were estimated for the Onkaparinga Catchment in South Australia. Two different catchments were considered; the catchment to Mt Bold Reservoir and the catchment to Old Noarlunga. The research concentrated on the catchment to Mt Bold Reservoir.

The PMF was calculated from an estimate of the PMP. In order to determine the magnitudes of floods less than the PMF it was also necessary to determine the frequency of floods up to the 1 in 100 annual exceedance probability flood event. The frequency distribution was then extended up to the PMF using the procedures contained in Australian Rainfall and Runoff (IEAust, 1987).

The non-linear runoff routing package RORB was used to rout the design rainfalls obtained from IEAust (1987). The sensitivity of the model to the choice of model parameters was analysed.

The recorded peak flows upstream of the Mt Bold Reservoir were used to fit a theoretical distribution. The data set was extended by using the recorded peak flows downstream of the reservoir prior to the dam's construction.

In order to determine the outflows from Mt Bold Reservoir it was necessary to calculate the spillway rating. The rating curve was extended so that the PMF could be routed through the storage.

The frequency of flows in the Onkaparinga River below Mt Bold Reservoir was calculated using the joint probability of inflows and the initial storage level. The effect of the choice of storage distribution was analysed.

Estimates of PMP up to 4 hours duration were calculated using the procedures contained in Bulletin 51 (Bureau of Meteorology, 1985). There was no funding or sufficient time for the Bureau of Meteorology to undertake a full PMP study for the Onkaparinga Catchment. The longer duration PMPs were therefore estimated by extrapolating the results from Bulletin 51 and comparing these with other generalised PMP studies undertaken for similar locations in south-eastern Australia.

The sensitivity of the estimated PMF was analysed. The effect of the choice of model parameters and the PMP on the calculated PMF was examined. Reasonable upper and lower bounds were calculated for both the inflow and the outflow PMF.

Chapter 2

Extreme Flood Estimation

2.1. Introduction

The probable maximum flood (PMF) is used in the design or evaluation of existing structures, where failure would result in severe damage and loss of life. The PMF is also used to determine the probability of floods which have a lesser magnitude than the PMF. Although there is a very small probability of the PMF occurring during the design life of a structure, the PMF is used as the design flood where a very high level of safety is warranted.

The PMF is determined from the probable maximum precipitation (PMP). A hydrological model is then used to convert the PMP to a PMF.

2.2. Probable Maximum Precipitation

IEAust (1987) states that all estimates of the probable maximum flood (PMF) should be based on the probable maximum precipitation (PMP). The PMP can be defined as,

> *"the theoretically greatest depth of precipitation for a given duration that is physically possible over a given size storm area at a particular geographical location at a certain time of year"* (World Meteorological Organisation, 1986).

Different organisations give slightly different definitions of the PMP (for example Riedel, 1977; Harlin, 1992). The definitions all contain the concepts of *'greatest depth'*, and *'physically possible'* rainfall.

Although the concept of the PMP is widely used, there still exists a need for greater understanding of the principles behind the derivation of the PMP.

> *"The bulk of the engineers concerned with dam spillway design are still far from appreciating the detail, or even the main principles, of PMP estimation."* (Laurenson, 1988)

There have been two basic approaches to the estimation of the PMF. These are:

1. the statistical approach; and
2. the deterministic or hydrometeorological approach.

The early methods of estimating the PMF were not based upon the PMP but were statistical. This involved fitting a theoretical distribution to a series of recorded flood peaks and extrapolating this to the PMF. This followed the approach used in Europe where there exists very long periods of records and historical flood marks. This method is also effective in Europe because the storms are generally of a homogeneous data set.

The statistical methods did not prove to be effective in Australian conditions. This was because (Pearce and Kennedy, 1993):

1. the generally short periods of records in Australia;
2. the lack of homogeneity in the storm population;
3. the lack of historical flood marks; and
4. the uncertainty of the actual distribution of flood peaks.

The problems with the statistical methods of estimating the PMF lead to the development of deterministic methods. These methods depended on the estimation of the PMP and the use of hydrological models to calculate the PMF.

2.2.1. Estimation of the PMP

The estimation of the PMP initially involved storm maximisation. This involved both the *in situ* maximisation of storms and the transposition of storms. Generalised methods have now been developed which allow the PMP to be calculated for catchments in Australia.

2.2.1.1. Maximisation of Storms

Traditionally PMP estimates were based on recorded storms which occurred in, or very near to, the catchment of interest. The recorded rainfall depths were then adjusted upwards based upon the moisture content of the air. This process is termed 'moisture maximisation'.

Moisture maximisation estimates the total moisture content of the air at a given location, temperature and humidity. It is an upper estimate of the possible depth of rainfall. This is because in a storm not all of the moisture from the atmosphere is precipitated (Raudkivi, 1979).

The measured rainfall depth in a storm is adjusted by the ratio of the highest atmospheric moisture content possible in the catchment to that observed in the storm. Although moisture maximisation is widely used, Kennedy et al. (1988) stated that it is difficult to prove analytically that storm rainfall can be maximised using the extreme moisture index.

The concept of maximising a storm led to the development of the term 'maximum possible precipitation' and then 'probable maximum precipitation'.

> *"The term 'maximum possible precipitation' was subject to severe criticism on account of the limited data available on extremes and the limited understanding of the processes leading to such an event. As a compromise, the more controversial and contradictory term 'the probable maximum precipitation' was introduced."* (Raudkivi, 1979)

In 1958, a conference was held by the Bureau of Meteorology on the estimation of extreme precipitation in Australia. It was suggested that the confusing terms 'maximum possible' and 'maximum probable' precipitation should be abandoned in favour of the expression 'extreme precipitation' coupled with an estimation of the probability of occurrence (Bureau of Meteorology, 1958).

2.2.1.2. Transposition of Storms

Initially only those storms which occurred in the catchment of interest were maximised. This was based on the philosophy that, *"the physical characteristics of the catchment played the dominant role in the production of rainfall from storms"* (Kennedy et al., 1988).

The concept of storm maximisation relied on extreme storms being recorded. The PMP estimates were therefore highly dependent on the occurrence of an outlier in the record. This resulted in inconsistencies between PMP estimates for different catchments. The concept of storm transposition was introduced to increase the sample of storms.

The concept of storm transposition (or spatial maximisation) deals with the translation of storms which have occurred near to the catchment of interest. It is based on the assumption that, *"within certain climatic boundaries the location of storms is determined by chance alone"* (Raudkivi, 1979). The transition of the isohyetal pattern from a storm is only justified in regions with a small difference in topographic influence.

Storm transposition was introduced in the late 1960s and early 1970s (Pearce and Kennedy, 1993). The increased sample of storms resulted in there being greater consistency between rainfall estimates. The choice of storms suitable for transposition introduced a certain degree of subjectivity into the procedure.

As the processes which produce extreme rainfall have been more fully understood, even wider transposition has occurred. For example, in New Zealand there are only 3 areas for transposition (Tomlinson and Thompson, 1991). The greater transposition has resulted in higher estimates of the PMP.

2.2.2. Generalised Methods

Generalised methods of estimating the PMP were introduced in Australia from about the mid 1970s. These methods make use of increased transposition. The use of generalised methods has resulted in increased estimates of the PMP. These PMP estimates have a lower annual exceedance probability and have the advantage of being regionally consistent.

These methods increase the useable transposition area by using a deterministic approach to adjust for topographic and moisture effects. This is based on the premise that, *"an equivalent optimum storm mechanism could occur anywhere in the transposition area, the frequency of occurrence is not important"* (Pearce and Kennedy, 1993).

2.2.2.1. Generalised Tropical Storm Method

The generalised tropical storm method (GTSM) was developed during the late 1970s and was finalised in 1985 (Pearce and Kennedy, 1993). The method is applicable to areas in Australia that are subjected to tropical storms and is described in Kennedy (1982) and Kennedy and Hart (1984).

The method is based upon that used by the National Weather Service in the United States of America to estimate the PMP for the Tennessee River Valley. The depth duration area curves are adjusted for the moisture content corresponding to a surface dew point of 28 °C.

The PMP estimates derived using the GTSM are larger than those obtained by the transposition of recorded storms. This is because the method effectively allows the transposition of storms over a very large area (Kennedy et al., 1988).

2.2.2.2. General South-Eastern Australia Method

The general south-eastern Australia method (GSAM) was derived to cover those areas of Australia not covered by the GTSM. The method has recently been completed by the Bureau of Meteorology. The need for such a method arose from the PMF study undertaken for the Warragamba Dam Catchment (Pearce and Kennedy, 1993).

The topographical effects on the rainfall in south-eastern Australia are more marked than in the tropics. The GSAM therefore separates the PMP into two components;

1. The convergence component of the rainfall. This is assumed to be due solely to atmospheric processes and it is therefore possible to transpose this over a compatible area.
2. The orographic enhancement of precipitation. This component is assumed to be a result of the topography of the particular catchment and is therefore not transferable.

The development of the GSAM involved the analysis of about 65 storms. The isohyets of these storms were drawn by hand and then digitised. The isohyets were fitted using Laplacian smoothing splines following the method described in Canterford et al. (1985a). The method also made use of the Computerised Design IFD Rainfall System which was developed as part of the production of IEAust (1987) (Kennedy et al., 1988).

The GSAM is applicable to catchments which have areas ranging from 10 to 40,000 km^2. The applicable durations are between 24 and 96 hours in New South Wales and between 24 and 72 hours in Victoria and Tasmania (Pearce and Kennedy, 1993).

2.2.2.3. Short Duration PMP Using Bulletin 51

Throughout Australia there is only a sparse network of pluviographs and these have been supplemented by a more concentrated network of daily read raingauges. The records of short duration storms is therefore not as complete as the records of longer duration storms. Accurate records of short duration storms have only been made when they have occurred over areas which are well instrumented.

In order to supplement the Australian database of short duration storms, the data collected in the United States of America has been used. This has made use of the longer period of record in the USA.

Extreme short duration storms are generally produced by thunderstorms.

> *"The most intense precipitation on drainage areas up to a few hundred km^2 are from thunderstorms"* (Raudkivi, 1979).

Walpole (1958) described generalised procedures for the estimation of the maximum possible rainfall for catchment areas from 10 to 500 square miles and for durations from 1 to 24 hours. It was assumed that the maximum possible rainfall intensity resulted from a thunderstorm in an air mass with the maximum possible surface dew point.

The current guidelines for calculating short duration PMPs for Australia are included in Bulletin 51 of the Bureau of Meteorology (BoM, 1985). This is based on both US and Australian thunderstorm data. The determination of short duration PMP is discussed in Kennedy (1982) and Pierrehumbert and Kennedy (1982).

The GTSM and GSAM are not applicable for short durations and therefore must be used in conjunction with the guidelines contained in Bulletin 51.

Initially the thunderstorm model was limited to use for catchments up to 500 km^2 in a narrow strip along the tropical and subtropical coast. The guidelines in Bulletin 51 are now applicable for catchments with areas up to 1,000 km^2 and durations up to six hours along the tropical and subtropical coastal areas, and up to three hours in inland and southern Australia.

The occurrence of a severe storm over Dapto near Wollongong in 1984 resulted in the procedures for the determination of short duration PMPs being changed. The depth-duration-area curves for rough terrain were modified which resulted in increases of the PMP for rough terrain by about 20 percent (Pearce and Kennedy, 1993; Kennedy et al., 1988). The US procedures were also reviewed and increased as a result of storms in the Tennessee Valley (Kennedy et al., 1988).

2.2.3. Increases in PMP Estimates

The different methods used to calculate the PMP have resulted in large increases in PMP estimates. This has had an important effect on the perceived safety of dams and other hydrological structures. Many existing spillways were found to have insufficient capacity to pass the newly calculated PMFs.

The increases in the PMP have resulted from (Laurenson, 1988; and Brown, 1988):

1. Changes in the methods used to estimate the PMP. Transposition has replaced *in situ* maximisation and now generalised methods allow even greater transposition.
2. The recording of more large storms.
3. Increased volume of knowledge on the mechanisms governing the occurrence of extreme rainfalls.

The large increases in estimates of the PMP have lead to possible credibility problems (Taylor and McDonald, 1988). This is because the new estimates of the PMP have so far exceeded previous estimates.

The increases in estimates have been a result of the effectively greater transposition employed by the generalised methods. It is therefore not expected that the estimates of the PMP will continue to increase infinitum.

> *"Once this change has been made, no further significant increases in PMP level is possible with the present storm data and the present methods of using the data."* (Kennedy et al., 1988)

As more extreme storms are recorded there may be some small alterations to the estimation of the PMP. These possible modifications are not however expected to dramatically affect the PMP estimates.

2.2.4. Greenhouse Effect

2.2.4.1. Introduction

The following definition of the PMP was given by Tomlinson and Thompson (1991);

> *"the greatest depth of precipitation for a given duration that is physically possible over a given size storm at a particular location under present climatic conditions."*

This definition introduced the concept of climate change. Present design methods are generally based upon the assumption that the climate is stable (Brown, 1988). The use of flood frequency analysis in particular assumes that the climatic conditions are constant. The current design philosophy needs to be changed to account for climate change (Robinson, 1987; Gorden and Tainsh, 1990).

Chow et al. (1988) stated that no allowance is made in the estimation of the PMP for climate change. In light of the large amounts of evidence which indicate a possible climate change, it is important that the effects of any climate change are considered.

The greenhouse effect refers to the warming of the earth by the greenhouse gases absorbing the radiated heat from the earth's surface. Greenhouse gases include water vapour, carbon dioxide and methane. The greenhouse effect maintains the earth at a temperature suitable for human habitation. The concern is not associated with the existence of the greenhouse effect but rather with the rate of increase in greenhouse gases which will increase the effects of greenhouse.

The greenhouse effect is a well established scientific theory. There is however great debate as to the likely effects of greenhouse. There is great uncertainty regarding the timing and magnitude of these effects, particularly on a local scale.

It has been possible through the use of global climate models to make predictions as to the possible changes in global climate. The resolution of these models is generally coarse and many simplifications are made (National Greenhouse Advisory Committee, 1992). Although they provide useful indications of possible changes in the global climate, these models do not allow accurate predictions of climate changes on a regional scale.

There is a need to consider the effects of greenhouse in current design procedures because the time taken to plan and construct water resource systems is of the same order of magnitude as the time over which significant climatic changes caused by greenhouse are expected to occur (Nathan et al., 1987).

It is therefore important that the issues associated with the effect of greenhouse are addressed by hydrological design. Daniell (1987) stated that, *"It seems strange that hydrologists in developing design procedures should divorce themselves from prognostications of other scientists"*.

2.2.4.2. Possible Climate Changes

The greenhouse effect will have serious repercussions on three climatic variables:

1. Temperature. It is generally accepted the greenhouse effect will result in an increase in the average global temperature of between 1.5 and 4.5 °C. The warming in Australia will be greatest in the southern winter and least in the north (Nathan et al., 1987).
2. Rainfall. It has been estimated that in Australia the winter rainfall will generally decrease whereas the summer rainfall will increase. For Adelaide the decline in winter rainfall may be about 20 percent and the increase in the summer rainfall could be between 20 and 30 percent (McIntosh and Fisher, 1989).
3. Sea Level. It is estimated that by the year 2030 the global average sea level would have increased by between 10 and 30 cm (National Greenhouse Advisory Committee, 1992). The increase is expected to be due to the expansion of the water column in the oceans and to the melting of the polar ice caps.

The changes in the rainfall are particular important to hydrological issues. The decrease in winter rainfall is especially important for the southern winter rainfall dominant states of Australia. The increase in summer rainfall will probably not result in large increases in streamflows because of the higher losses in summer.

2.2.4.3. Effect on the PMP and PMF

It is generally concluded that the greenhouse effect will result in extreme storms becoming more frequent (Robinson, 1987). Gorden and Tainsh (1990) noted that the greenhouse effect will result in, *"increased storminess"*. This will probably lead to increases in PMP and PMF estimates (Deen, 1987; Jakeman, 1990; and Robinson, 1987).

> *"Dependent upon the rainfall intensity-duration characteristics associated with any increase in the PMP, the result could directly influence the magnitude of extreme flood events and impact upon dam design and the safety of existing structures."* (McIntosh and Fisher, 1989)

The possible increases in the estimates of the PMP in southern Australia are thought to due to two main phenomena:

1. Southerly shift of weather. This would mean that those winter rainfall regions of southern Australia would be subjected to a greater number of tropical storms and hence more extreme storms. It has been suggested that this will result in increases in PMP estimates. Kennedy et al. (1988) noted however that, *"As regards an extension of the cyclone belt southwards, this in itself would not affect our current PMP estimates"*. This is because the database for south-eastern Australia already contains some storms of tropical origin.

2. Increase in average temperatures. The increase in temperature resulting from the greenhouse effect will result in an increase in the maximum persisting dew point. This is likely to result in increased estimates of the PMP (McIntosh and Fisher, 1989). The maximum precipitable water can be estimated to increase by about 8 percent for every degree Celsius rise in temperature (Deen, 1987).

2.3. Probable Maximum Flood

2.3.1. What is a Probable Maximum Flood?

The concept of the probable maximum flood (PMF) is widely used in extreme flood estimation. A PMF may be required for direct use in design situations of high risk, or as an interim step in determining the magnitude of other extreme flows which are less than the PMF.

The US Army Corps of Engineers (1979) defined the PMF as,

> *"the flood that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the region."*

The PMF is important in the estimation of extreme flood events, and in determining the safety of hydraulic structures. The Guidelines on Design Floods for Dams published by the Australian National Committee on Large Dams (ANCOLD, 1986) provide recommendations for calculating dam safety and the adequacy of spillways. The guidelines require an estimate of the PMF. The methods outlined in Australian Rainfall and Runoff (IEAust, 1987) in Chapter 13 are suitable for use with the design guidelines in ANCOLD (1986).

Estimates of extreme floods are also required by the National Association of Australian State Road Authorities (NAASRA) in their standard for the design of major bridges. The 1 in 2,000 annual exceedance probability (AEP) flood is required, and this flood is derived from an estimate of the PMF.

2.3.2. History

In considering the concept of a PMF it is useful to look at the development of the terminology used to describe extreme floods. A number of different terms have been used to describe extreme floods.

2.3.2.1. United States of America

Prior to 1900, the design flood for a hydraulic structure was determined using the judgement and experience of the engineer. The estimate was based on the historical information and eye-witness reports as to maximum observed floods.

As stream discharge data became available, statistical methods were developed. An example of such a statistical formula was calculated by Fuller (1914) and shown in Equation 2.1 (Riedel, 1977).

$$Q_T = \overline{Q}(1 + 0.8 \log T) \qquad (2.1)$$

Q_T = flood of return period T years
$\overline{Q}$ = mean maximum annual flood

Riedel (1977) stated that the term 'probable maximum flood' has been used for many years as it was mentioned in Fuller (1914).

The next development in the determination of design floods was the development of relationships between the peak discharge and the catchment area. Regional regression equations are still being used today and provide a quick and simple method of determining approximate values of design flows based on catchment characteristics.

The term 'maximum probable flood' was generally used up to the mid 1950s. This was based upon similar assumptions as those used to derive the PMF. After this time, the concept of the PMF was slowly adopted.

> *"It is unfortunate that the wording has changed since the basic concepts of PMF have not, nor has the overall method of its determination changed."* Riedel (1977)

2.3.2.2. United Kingdom

In the United Kingdom after 1933, estimates of extreme floods were based on envelope curves on plots of runoff intensities against catchment area (Shaw, 1989). This lead to the concept of a 'normal maximum flood'. This was used to determine an estimate of an extreme flood for a particular catchment area. This was not however the greatest possible flood as it was suggested that, *"due regard be paid to a possible catastrophic flood which could have extreme rates of runoff at least twice of those of the normal maximum flood"* (Shaw, 1989).

The concept of 'normal maximum flood' has now been replaced with the PMF.

2.3.2.3. Sweden

The Swedish guidelines use a deterministic approach which is similar to the concept of the PMF. The PMF is not calculated from an estimate of the PMP but rather the maximum areal rainfall is combined with extreme snowmelt. A trial and error approach is used to determine the critical combination of rainfall and snowmelt which produces the largest flood for a specific catchment (Harlin, 1992).

2.3.2.4. Australia

The first use of the PMP in Australia was by Lovett in 1954 for the Warragamba Dam site in New South Wales (Deen et al., 1989). Two different extreme floods were identified; the maximum possible flood and the maximum probable flood. The two floods were distinguished by their likelihood of occurrence. The maximum probable flood was considered to be a reasonable estimate of the design flood for the dam. The maximum possible flood was considered to be too extreme to be used for the spillway design. The maximum possible flood was based on similar assumptions as those used to derive the PMF.

Reports on the history of the estimation of the PMF for particular Australian dams provide a useful insight into the development of extreme flood estimation methods in

Australia. Two examples are for Warragamba Dam in Deen et al. (1989) and for Burrinjuck Dam in Green (1988). Both of these papers state that flood extremes are greater in Australia than those observed overseas, especially in Europe.

2.3.3. Definition of the PMF

There are many different definitions of the PMF and each of these indicate the different philosophies that are held by different groups.

The guidelines produced by the Australian National Committee on Large Dams defined the PMF as,

> *"The flood hydrograph resulting from PMP and, where applicable, snowmelt, coupled with the worst flood-producing catchment conditions that can be realistically expected in the prevailing meteorological conditions."* (ANCOLD, 1986)

The philosophy expressed in IEAust (1987) and discussed in Pilgrim (1986a) is that,

> *"the PMF should constitute a limiting value of floods that could reasonably be expected to occur. Superimposing risks of very low probabilities is not considered to be reasonable".*

The philosophy of the Bureau of Reclamation is,

> *"Determination of maximum probable flood is based on rational consideration of the chances of simultaneous occurrence of the maximum of the several elements or conditions which contribute to the flood. Such a flood is the largest that reasonably can be expected."* (Riedel, 1977)

The definitions of the PMF all introduce the concept of a 'reasonable' estimate of the PMF. Although the Swedish PMF is not based on the PMP, the definition given in Harlin (1992) also introduces the concept of 'reasonableness'.

2.3.4. A 'Reasonable' Estimate of the PMF

The concept of a 'reasonable' estimate of the PMF needs some clarification. Little guidance is given as to what is deemed 'reasonable', with the choice and consequent responsibility being left with the designer.

Initially the desire of extreme flood estimation was to obtain a maximum flood which was an 'upper limit' and this was reflected by the terminology of "maximum possible flood". This term was superseded by the PMF, but the meaning remained the same. The intention was that the PMF was an upper limit of the flood estimates.

In order to produce such an estimate of the PMF, the PMP was used with severe catchment conditions. Shaw (1989) recommended that,

> *"The PMF can be determined from the PMP by taking the worst catchment conditions, minimising rainfall losses and using high runoff coefficients."*

This is confirmed by Brown (1988) which suggested that conservatism should not only be considered but also that it should depend on the potential consequence of underestimating the PMF.

The concept of a 'reasonable' estimate is now widely accepted as testified by the definitions of the PMF shown above. The intention of estimating the PMF is now to produce a 'reasonable' estimate, which is not the absolute maximum.

> *"The philosophy in ARR is that, while there is still a need for a reasonable degree of conservatism in extreme flood estimation, what the designer should be aiming for is a reasonable probable maximum, rather than a maximum possible flood."* (Wright, 1988).

The desire of the current guidelines is to produce a PMF which has a similar probability as the PMP. This implies that average values are used for catchment characteristics and model parameters.

> *"At each step of the analysis, the most likely set of circumstances which could occur should be assumed."* (Wright, 1988)

The intention of hydrologic modelling for small flood design is to select losses, temporal patterns and other model parameters to ensure that the probability of the resultant flow is the same as that of the design rainfall. The concept of choosing model parameters to ensure that the probability of the resulting peak flow is consistent with that of the rainfall is a relatively new concept however for the estimation of the PMF.

This approach avoids the estimation of a physical upper limit although the PMP is still considered to be the maximum possible rainfall (Laurenson, 1988).

Although the intention of the current procedures are designed to produce a 'reasonable' estimate of the PMF, there is still the belief that the PMF represents an upper limit. This is inconsistent with its present derivation.

> *"The concept of a physical upper limit has been discarded even though most of the discussions of the PMF persist in using this concept."* (Laurenson, 1988)

A PMF which has a probability of exceedance (however small) and which is not an upper limit should perhaps be renamed to indicate the change in philosophy. The introduction of another term would have the advantage of making a distinction between a PMF calculated as an upper limit and a PMF calculated as a reasonable estimate. The introduction of yet another term however, may only lead to further confusion.

It is evident that the inclusion of an excessive degree of conservatism in the estimation of the floods is undesirable. This is because the inclusion of safety factors or overly conservative design procedures will effect the AEP of the resulting flood. The tendency for extreme conservatism due to the severity of failure should be resisted (Cantwell and Murley, 1986). The design procedures outlined by NAASRA and ANCOLD (1986) already include safety factors and therefore require an accurate estimate of the PMF, *"further arbitrary safety factors are unduly conservative and undesirable"* (Pilgrim, 1986a).

Although it is important that the AEP of extreme floods are not obscured by over-conservatism, it is also vitally important that the PMF is not under-estimated, and that the PMF is indeed a 'maximum'.

As already stated, the differentiation between conservatism and the need for a reasonable estimate is left to the designer. Wright (1988) stated that, *"conservatism in extreme flood estimation is difficult to define precisely."*

In striving to derive a reasonable estimate of the PMF it is therefore important that reasonable estimates of all parameters are made. This is pertinent to the choice of variables such as catchment antecedent conditions, assumed reservoir storage, treatment of baseflow, rainfall losses, temporal patterns and model parameters. Daniell (1987) referred to the desire to achieve the *'aurea mediocritas'* (the golden mean) in estimating the PMF.

In cases such as extreme flood estimation, where a risk is ultimately thrust upon the general public, the hydrologist's primary responsibility is to ensure that any estimate is not unconservative. It is therefore vitally important that the PMF estimates do possess some 'reasonable' degree of conservatism.

2.3.5. Assigning a Probability to the PMF

The PMF is a deterministic, rather than a probability based, concept. (Faulkner, 1988). In order to determine the magnitude of flows which are less than the PMF and for the purpose of economic evaluation, a probability needs to be assigned to the PMF. Harlin (1992) noted that a committee on Water Data in the USA concluded that,

> *"no procedure to date is capable of assigning an exceedance probability to the PMF or to near-PMF floods in a reliable, consistent, and credible manner."*

Chow et al. (1988) also concluded that the frequency of the PMF could not be determined.

Lave et al. (1990) considered the probability of the PMF by analysing the records of the dams in the United States. It was suggested that if the PMF is assumed to have a AEP of 1 in 10^4, there should be a PMF recorded in the United States once every 10 years. No

PMF has been recorded. The conclusion reached was that the PMF has a probability of less than 1 in 10^4.

2.3.5.1. Probability of a PMF Estimate Representing a Limiting Flood

If the PMF is considered to represent an absolute maximum limit of floods, the probability of exceedance by definition should be zero. A PMF is however assigned a probability of finite value for the purpose of estimating extreme floods less than the PMF (Daniell, 1987).

Such a philosophy was suggested by Deen (1987) when it was stated of the PMF;

> *"This flood is the largest conceivable flood that could affect the dam and is usually derived from the PMP. This precipitation is calculated to yield an upper limiting value so that the level of risk need not be considered."*

2.3.5.2. Probability of a Reasonable PMF Estimate

Because the philosophy and the methods used to derive the PMF have changed, the probability assigned to the PMF needs to be modified. The assigned probability should reflect the degree of 'reasonableness' incorporated in the estimate.

The philosophy of a 'reasonable' PMF means that the PMF has an absolute probability of exceedance. This means that the possibility of floods greater than the PMF must be accepted.

> *"The increasing references to the AEP of the PMP logically demand that the definition of PMP be changed; rather than being regarded as a maximum possible rainfall, it must be regarded simply as one point on the upper tail of a probability distribution that extends to infinity."* (Laurenson, 1988)

Although is may take some time before it is readily accepted, the concept of probabilities of floods and rainfalls greater that the probable maximum events is correct and the terms should be embraced by the profession.

2.3.5.3. Guidelines in IEAust (1987) for Assigning an AEP to the PMF

IEAust (1987) introduces the concept of a probable maximum event (PME). The procedures outlined are applicable to rainfalls, peak flood magnitudes and flood volumes. Two different criteria are used in selecting the AEP of the PME. Consistent with the desire for a reasonable degree of conservatism, the highest AEP is selected.

The two different methods of calculating the AEP of the PME are:

1. Meteorological considerations. This procedure is based on Kennedy and Hart (1984) and relates the ratio of the catchment area to the transposition area used to derive the PMP. An AEP of 1 in 10^3 is assigned for PMP estimates derived from *in situ* maximisation. PMP estimates derived from adjusted United States data are assigned a probability of 1 in 10^8 for catchments with areas of approximately 100 km^2.
2. Shape of frequency curves. This procedure was described by Rowbottom et al. (1986b). The AEP of the PME is determined from the relative magnitudes of the PME and the 1 in 100 and 1 in 50 AEP events. Australia is divided into two zones with zone B consisting of southern Western Australia, South Australia and western Tasmania. For zone B the AEP of the PME based on the shape of the frequency curve varies from an AEP of 1 in 10^4 to 1 in 10^7.

A flood frequency curve is then fitted between the PMF and the 1 in 100 AEP flood. The higher the AEP assigned to the PMF, the more conservative will be the estimates of intermediate floods.

The frequency curve between the 1 in 100 AEP event and the PME is constructed using curve fitting procedures. These procedures depend on the relative magnitudes of the PME and the 1 in 100 and 1 in 50 AEP events.

2.3.5.4. AWRAC Project

Kennedy et al. (1989) described the findings and conclusions of the investigations carried out under the Australian Water Research Advisory Council Partnership Project; Probability of Occurrence of Extreme Rainfall and Floods from April 1987 to June 1989.

Five basic approaches were considered (Kennedy et al., 1989):

- extrapolation of the flood frequency curve;
- regional analyses;
- joint probability analyses;
- palaeohydrologic analyses; and
- Bayesian analyses.

These methods were not considered to be mutually exclusive but rather many of the approaches rely on at least one of the other approaches.

Two different methods of assigning the AEP to the PMF and PMP were analysed;

1. District Extremes/Point-to-Area Approach. This method is an extension of the storm transposition theory. The method determines the AEP of rainfall at a specific point

and the equivalent probability of the same rainfall occurring anywhere in the rainfall district considered. An areal reduction factor is required to convert the probabilities of the point extremes to the probabilities of district rainfalls. It is believed that this method gives improved results over those in Kennedy and Hart (1984).

2. Joint Probability Approach. This method is based on the theory that the probability of the extreme event is the probability that the extreme values of all the stochastically varying factors occur simultaneously (Kennedy et al., 1989). The method outlined in Laurenson (1974) is used.

Faulkner (1988) described a similar procedure of using joint probability to determine the probability of the PMF. The probability of the PMF was dependent upon the probability of the rainfall depth and location and the probability of the antecedent moisture condition. In a case study for Caldron Falls Dam in the United States, the PMF was calculated to have a recurrence interval of 30 million years.

The AWRAC study concluded that both methods examined for determining the probability of the PMF were feasible. Further work needs to be undertaken to develop design procedures.

2.3.6. Unnatural PMF

The discussion to date has dealt with natural PMFs; PMFs that result from extreme rainfall. There is however a second class of PMFs which need to be considered. Unnatural PMFs can result from upstream dam failures. This is particularly important for catchments with more than one large storage.

There are two different philosophies which can be adopted in this case (Daniell, 1987):

1. All of the upstream dams should be designed for the same probability. This means that the safety of the downstream dam is not compromised by the dams upstream.
2. The downstream dam can be designed for the unnatural PMF.

2.3.7. Runoff Routing Model

A runoff routing model or unit hydrograph is required to convert the rainfall excess into a hydrograph of direct runoff. This requires estimates of losses, temporal patterns, antecedent conditions and other model parameters. Where possible, the model should be calibrated with records of floods which have occurred in the particular catchment.

2.4. Sensitivity of the PMF

It is important that the possible errors and sensitivity of the estimated values of the PMF are understood. Because of the large extrapolations involved with the estimation of the PMF it is important that sensitivity analyses are undertaken.

Daniell (1987) discussed the sensitivity of the estimation of the PMF to the choice of various parameters. These results were based on the analysis of the dams within the ACT. Special attention was paid to the sensitivity of the PMF to the location of the isohyetal pattern.

Deen et al. (1989) discussed the sensitivity of the PMF estimate for Warragamba Dam Catchment. The study analysed the effect of the model parameters and the PMP. The assumption of level pool routing was also checked by analysing the effect of wedge storage.

The results of the sensitivity of the PMF from these two studies are shown in Table 2.1.

Table 2.1 Sensitivity of the PMF from other Studies

Assumption	Daniell (1987)	Deen et al. (1989)
Temporal Pattern	-	High
Spatial Pattern	-15% to +30% of PMF	High
Depth of Precipitation	15 - 20% of PMF	High
Model Parameters	-25% to +30% of PMF	Moderate
Loss Parameters	< 2% of PMF	Moderate
Antecedent Floods	-	Low
Greenhouse Effect	-	High
Addition of Baseflow	< 2% of PMF	-
Wedge Storage	-	Low
Initial Storage Level	5% - 50%	-

2.5. Dam Safety

2.5.1. Introduction

The field of dam safety is a complex and developing area as public attitudes change. The following short discussion on dam safety presents some of the issues associated with the topic.

The failure of a dam can result in catastrophic loss of life and property. The cause of a dam failure is difficult to define precisely as it is usually a combination of more than one of the following (National Water and Soil Conservation Authority of New Zealand, 1986; and Cantwell and Murley, 1987):

1. misinformation on local hydrology and geology;
2. foundation failures;
3. design faults (including spillway capacity);
4. faulty construction;
5. overtopping;
6. neglect; and
7. unexpected natural events.

A safe dam depends upon good design and construction, and ensuring that the dam remains fit for its purpose throughout its design life. About a half of all dam failures occur during the first 5 years after completion (Baecher et al., 1980). Once a dam has survived the initial filling stage, the most likely cause of a dam failing is a large flood (Murley, 1985).

Dam safety has greatly improved over the years. Before 1899 approximately 15 percent of all dams failed. By 1960 the fraction of dam failures had dropped to 0.26 percent (National Water and Soil Conservation Authority of New Zealand, 1986). The improved safety record has been a result of better design methods based upon longer records of streamflow and rainfall.

There is a much greater chance of failure of small dams than of larger dams. This is because the consequences of the failure of smaller dams are less catastrophic and hence a lower level of safety is accounted for in the design.

It is difficult to estimate the probability of dam failures. This is because dams can fail through an essentially infinite number of mechanisms which cannot be fully enumerated; *"most failures occur due to accident, inadequate construction control, or poorly understood physical processes"* (Baecher et al., 1980). Once the dam has failed, most of the direct evidence of the mechanism of failure is destroyed.

2.5.2. ANCOLD Guidelines

There is general consensus as to the importance of dam safety. Controversy arises when attempts are made to establish guidelines and assign responsibility for safety standards. The International Committee on Large Dams (ICOLD) has found that it cannot produce a single internationally uniform code for dam safety (National Water and Soil Conservation Authority of New Zealand, 1986).

A national survey of design flood practices for Australian dams was conducted in 1978. This highlighted the wide variations between different authorities in the procedures that were used to determine design floods for dams. Following the workshop on spillway design held at Monash University in 1981, an interim document was produced for comment in 1984. This document was finalised and published in 1986 (ANCOLD, 1986).

The intention of the guidelines was to provide a national philosophy for the determination of spillway capacity and the review of the adequacy of existing spillway capacities. The guidelines were also designed to assist in assessing in an orderly and consistent way the relative priorities for dam rehabilitation (Cantwell and Murley, 1987).

It was not the intention of the guidelines to provide a design manual for determining spillway capacity that could be implemented by a person with inadequate hydrological experience. The guidelines were not written to be mandatory such as the structural codes but rather to provide a flexible approach. It was however intended that owners would have to justify the adoption of standards outside of the guidelines (Murley, 1992). The philosophy expressed by the guidelines is, *"A dam owner will always retain the primary legal and moral responsibly for the safety of his dam"* (Cantwell and Murley, 1986).

The guidelines apply to 'referable' dams. Referable dams are considered to be those which are at least 5 metres high and have a capacity greater than 50 x 10^3 m^3 or are at least 10 metres high and have a capacity greater than 20 x 10^3 m^3. Referable dams apply to, *"any artificial barrier, temporary or permanent, including appurtenant works, which does or could impound, divert or control water, other liquids, silt, debris or other liquid-borne material"* (ANCOLD, 1986).

2.5.2.1. Incremental Flood Hazard Category

ANCOLD (1986) distinguishes between the risk and the hazard. The risk is taken to be a measure of the chance that the dam will fail as a result of a shortfall in some aspects of the dam. The hazard is a measure of the consequences of the failure of the dam, with no regard to the probability of failure. The hazard needs to be examined periodically because the hazard may increase as more development occurs downstream of the dam. ANCOLD (1986) uses the concept of the incremental flood hazard. The incremental flood hazard is the,

> *"incremental loss of life, property and services, which is directly attributable to the failure of the dam, that is, the incremental loss over and above the loss which would occur if the dam did not fail."* (Cantwell and Murley, 1987)

Following the procedures in the United States, there are three different incremental flood hazard categories. These are high, significant and low. These are assigned depending on the threat to both life and property.

The recommended design flood is determined directly from the incremental flood hazard category. In cases where there is not a threat to life, ANCOLD (1986) allows the recommended design flood to be determined from an economic risk analysis.

The guidelines apply for both proposed and existing structures. The procedures recommended for determining the adequacy of a spillway are different for existing structures (NRC, 1985). This is based on the philosophy that, *"It is accepted in most countries that safety standards for existing structures including dams need not necessarily be the same as for the proposed structure"* (Cantwell and Murley, 1987). This is because the construction of a dam poses a new risk (Murley, 1992).

It was felt that the publishing of the guidelines would face some opposition from owners of existing dams which were shown to have undersized spillways (Cantwell and Murley, 1987). The guidelines did not however, directly cause the large costs of remedial work but rather highlighted the need for a greater level of safety.

2.5.2.2. The Effect of the ANCOLD Guidelines

The formation of the ANCOLD guidelines had an important effect on the analysis of the safety of dams throughout Australia. This was because uniform procedures were adopted which assisted in the prioritising of remedial works.

Parsons (1987) analysed the effect of the ANCOLD guidelines in South Australia. The twenty large dams owned and operated by the Engineering and Water Supply Department were examined in light of the recommendations. For each dam the evaluated design flood, the imminent failure flood, the incremental flood hazard category and the recommended design flood were calculated. Three dams were considered to have a high priority for further evaluation; Hope Valley, Tod River and Mt Bold.

Cantwell and Laird (1985) discussed the effect of the guidelines for the NSW Dam Safety Committee. Thirty four high hazard dams with significant catchment areas were considered. A total of 10 flood reviews had been reported to the NSW Dam Safety Committee and this had resulted in modifications being made to 6 dams. It was noted that the guidelines had only a moderate effect in NSW because many of the operators and owners of dams had already undertaken reviews as a result of the increases in the estimates of the PMP.

Murley (1985) suggested that there exists a misconception that the guidelines require that all dams must be designed to pass the PMF. The choice of the PMF as the design flood is conservative and therefore should only be adopted where there is high hazard.

2.5.3. Imminent Failure Flood

The imminent failure flood (IFF) is defined by ANCOLD (1986) as,

> *"the flood event which, when routed through the reservoir, with the existing spillway, just threatens the failure of the dam."*

The IFF is used as the basis for determining the adequacy of existing spillways. The ratio of IFF to PMF is used to prioritise remedial works.

For embankment dams the IFF is taken as the flood which just causes the embankment to be topped. For concrete dams the depth of overtopping which causes failure can be determined from a structural analysis.

Once the IFF is determined, the IFF hydrograph is determined by scaling the ordinates of the PMF hydrograph. Murley (1992) suggested that the scaling of the PMF is not rigorous but it is a practical means of making an initial determination of the spillway capacity.

The procedure of scaling the PMF hydrograph is based on several assumptions which do not have any physical basis. If a linear routing model is used the effect of scaling is understood, however if a nonlinear routing model is used the scaling will have a uncertain effect (Green, 1991).

The use of the PMF to determine the IFF hydrograph assumes that the same storm mechanism that causes the IFF also causes the PMF. It is assumed that the same storm duration and pattern apply but that there is a reduction in the PMP depth.

The proportioning of the PMF ordinates assumes that (Green, 1991):

1. the IFF will be the same duration as the PMF;
2. the critical duration of the IFF is the same as that of the PMF; and
3. the IFF hydrograph will have the same time to peak as the PMF, but flatter rising and falling limbs.

The outflow frequency curve is the envelope of outflows resulting from different storm durations and probabilities. Once the IFF has been determined, it is necessary to make a choice as to the combination of storm duration and AEP which will produce the required outflow (Milner and Alvarez, 1989). Green (1991) suggested that the IFF hydrograph should be the most likely hydrograph with the determined IFF peak flow.

A more consistent method of determining the IFF is to scale the rainfall depth until an outflow of the required peak is obtained. Temporal and areal patterns can then be used which are appropriate for a storm of that AEP (Green, 1991).

2.5.4. Society's Perception of Risk

Most disasters affect a particular sample of the population. Examples of such disasters are cyclones and earthquakes. People appear to accept these risks from natural causes. Other disasters relate to a specific identifiable population. An obvious example is people living downstream of a major dam. People are less willing to accept risks of this nature (Cantwell and Murley, 1987). People are generally more willing to accepts risks of their own choosing rather than have additional risks thrust upon them by authorities.

The level of risk accepted by the community is largely dependent upon their awareness of the risk. Many people are unaware of the risk of dam failures.

The level of acceptable risk is also determined by the standard of living of the particular society. In developed countries where other risks such as disease are reduced, a much higher level of safety is expected; *"societies with developed economies expect a fail-safe design for facilities which could endanger identifiable lives"* (Cantwell and Murley, 1987).

In most developed countries the risks of dam failure is shared across society. This is done in the form of national disaster relief programs and other forms of insurance.

In order for a design flood magnitude to be determined, a decision must be made as to the risk which is willing to be accepted by the community. This is a difficult task.

> *"There is no accepted figure, or methodology, for assessing socially acceptable risk for a dam failure, although there are many references to risk for other causes."* (Murley, 1992)

Doherty (1988) examined the risks that the community accepts in other areas such as leisure activities, transport and unavoidable unnatural fatalities (such as poisoning and lightning). It was concluded that an annual probability of failure of a dam of 1 in 10^5, where a fatality would occur, and an annual probability of 1 in 10^6 where widespread fatalities would occur was consistent with the community's acceptance of risk.

Murley (1992) suggested that the public accepts a risk of between 1 in 10^6 and 1 in 10^8 in cases where multiple loss of life is at risk.

A recent pilot study undertaken by the CSIRO and funded by ANCOLD analysed the community's perception of the risk of dam failure (Bishop and Syme, 1992). This involved a survey of a limited population. It was concluded that dam safety is not a noteworthy issue in the community. People who were fearful of dam failure were generally more fearful of other technological hazards. Most of those surveyed did not feel that dam failure was a short term risk. A larger proportion considered that dam failure was a significant long term risk (Syme et al., 1992). The majority of people surveyed did not consider that it was necessary to consult the community on matters concerning dam safety (Bishop et al., 1992).

Although the probabilities considered in dam safety appear to be very low, it is important that the probability of failure of the structure over the design life is considered. For an AEP of 1 in 10^4 and a design life of 100 years this equates to a chance of 1 percent (Murley, 1985).

Lave et al. (1990) argued that the present safety criteria for dams is too conservative when compared to other areas of risk.

> *"The criteria require much more to be spent to prevent a premature death from overtopping a dam than is spent to prevent a death in virtually any other social decision."* (Lave et al., 1990)

The failure of dams result in the loss of 11 lives per year on average in the United States (Lave et al., 1990). Although any premature death is regrettable this is substantially less than the number of deaths caused by other incidents such as road fatalities. It appears that people are risk averse in situations in which large loss of life is possible.

There is the perception that dam designers are adopting extremely conservative positions compared with other designers.

> *Devoting resources to ensure a dam survives extremely rare floods amounts to 'worshipping' the dam rather than seeing it as a utilitarian structure.* (Lave et al., 1990)

For the case of high hazard dams, a high level of safety can be justified by the horrific consequences of dam failure. The perception of undue conservatism could possibly be altered if the consequences of dam failures were further researched and the results published (Taylor and McDonald, 1988).

If the consequences of the failure of a dam are too large for people to tolerate, it may mean that the only conclusion possible is not to construct the dam. An example of such a dam is the Auburn Dam in the United States (Lave et al., 1990). The proposed dam was not constructed because the consequences of failure were too large for the community to accept, regardless of how small the probability of failure.

The area of risk analysis is a complex and developing area. A thorough summary of risk analysis is given in Wellington and McDonald (1991). Wellington (1987a) described the risk assessment procedures in the United States and Europe. Other references on risk analysis include Dawdy and Lettenmaier (1987) and Wellington (1989 and 1987b).

2.5.5. Economic Risk Analysis

In the past, spillway capacities have been designed for the PMF. The increased estimates of the PMF, based on the new PMP estimates, have resulted in the practice of designing for the PMF being questioned (Chow et al., 1988). There is a growing consensus that a more reasonable approach is to adopt a probabilistic approach. This is based on an analysis of the costs associated with the risks of failure and the costs of remedial works (Wright, 1988).

The development of water resource projects is usually based on the expected benefits exceeding costs. The goal for a dam is to maximise social net benefit (Lave et al., 1990). In the past the cost of the failure of dams has been neglected because it was felt that well built dams would not fail. This has not proven to be the case and therefore the failure of a dam is a real cost and should be incorporated in any economic analysis (Baecher et al., 1980).

Economic risk analysis necessitates that a value be placed upon human life. Although this appears to be ethically unpleasant, it is important to note that a certain value is put on human life by many common decisions. A decision such as purchasing a smoke detector or taking a dangerous job indicates how much a person is willing to pay to reduce the chance of dying (Lave et al., 1990).

The failure of a dam can have important costs which occur beyond those which occur at the instant of failure. These include the loss of economic activity, decreased production in other areas and the loss of future benefits of the dam (Baecher et al., 1980).

The ANCOLD guidelines allow economic risk analysis for dams of intermediate or low incremental flood hazard categories. For dams of high incremental flood hazard category the design flood should be the PMF. In the September 1992 ICOLD conference in Granda Spain there was no reference to economic risk analysis being used for high hazard dams (Murley, 1992).

2.5.5.1. Economic Risk Analysis for Kangaroo Creek Dam

The economic risk analysis undertaken for Kangaroo Creek Dam is one of the few such studies undertaken for high risk dams. The study was completed in the early 1980s, before the development of the ANCOLD guidelines.

Kangaroo Creek Dam is situated on the Torrens River upstream of Adelaide. Failure of the structure would result in severe damage to the suburbs of Adelaide and therefore the ANCOLD hazard classification is high. The outflow PMF for the dam was calculated to be approximately 5,000 m^3/sec (Water Resources Branch, 1981). Because of the difficulty and expense of providing a spillway to safely pass the PMF, an economic analysis was undertaken.

The most difficult component of the study was the determination of the breach mode (Laing, 1982). It was estimated that the failure of the dam would result in a peak flow of 11,300 m^3/sec.

The damage resulting from the PMF was calculated using the results from floodplain mapping. It was estimated that 100 lives would be lost as a result of the PMF and each of these were valued at $250,000. The repairs to the dam were estimated to cost $20 million and the design life of the dam was assumed to be 300 years. The greatest cost was associated with property damage, and the total cost of the dam failure was estimated to be $6,000 million (Laing, 1982).

Different spillway capacities were analysed to determine the most economical. The final design inflow was 1,500 m^3/sec which has an estimated AEP of 1 in 50,000 (Good, 1985). This design flood resulted in the greatest net benefit ($8 million). The total cost of modifications was $1.7 million.

The decision to adopt a design flood significantly less than the PMF based on economic analysis was ultimately a decision which was made by the Government on the strength of the information presented by the E&WS. It was not considered prudent to divert funds from other areas to upgrade the dam to pass the PMF (Good, 1985).

2.5.6. Flood Warning

Another method of increasing the safety of a dam is to instigate flood warning systems. An effective and credible warning system can prevent loss of life due to the failure of a dam. A warning time of between 1 and 2 hours can decrease the number of deaths to almost zero (Lave et al., 1990).

Although the use of flood warning systems to reduce the loss of life is a cheaper alternative to increasing the spillway capacity, it is not as appealing to the communities downstream which still risk property damage. From the analysis of three dam upgrade projects in New South Wales, there was strong community support for full spillway upgrade to the PMF standard rather than the installation of flood warning schemes (Murley, 1992). Flood warning schemes were also noted as being unpopular by Bishop and Syme (1992).

Unfortunately a comprehensive flood warning system would be difficult to operate for the two major dams posing a risk on the suburbs of Adelaide; Mt Bold Dam and Kangaroo Creek Dam. This is because the critical duration PMP for both of these catchments is less than 6 hours (Daniell and Hill, 1993c; and Water Resources Branch, 1981).

2.6. Occurrence of Extreme Storms

2.6.1. Introduction

Estimates of the PMP and PMF usually far exceed all observed extreme events. For this reason the estimates have suffered a lack of credibility. The occurrence of extreme rainfalls and floods are important in reaffirming the existing design approach. The records of extreme events serve as a reminder of, *"the vagaries of nature and a justification for a high degree of conservatism in the design flood selection process"* (Brown, 1988).

It is unlikely that rainfall records at any one location will contain an extreme storm which approaches the PMP (Brown, 1988). This is because the records are too short and the network of raingauges is too sparse. In general the failure to record such extreme rainfalls does not mean that these estimates are unrealistic but rather that the current network and period of record are insufficient.

It is also important to note that in many cases the rainfalls are caused by mechanisms which were not thought to apply in the given area (Brown, 1988).

2.6.2. Extreme Events Recorded in the USA

It is interesting to consider the records of floods and storms in the USA because there exists a far more comprehensive database. The greater population has resulted in a greater density of raingauges and their longer period of settlement has resulted in generally a longer period of record.

Riedel and Schreiner (1980) examined recorded storms in the USA and compared these with generalised estimates of the PMP. It was found that there were 243 storms for which the recorded depth was greater than or equal to 50 percent of the estimated PMP.

In a later study (Bullard, 1988), the magnitudes of recorded rainfall induced floods were compared with the PMF estimates across the USA. The results from 61 floods are shown in Table 2.2. The distribution of floods in Table 2.2 appears unusual and there is no indication in the study as to why this might be.

It was concluded that the methods used by the United States Bureau of Reclamation to determine the PMF were technically reliable and consistent with observed floods.

Table 2.2 Extreme Floods Measured in the USA (source: Bullard, 1988)

Percent of Estimated PMF Peak	Number of Recorded Events
91-100	2
81-90	7
71-80	4
61-70	9
51-60	8
41-50	13
31-40	8
< 30	10

2.6.3. Extreme Events Recorded in other Countries

There are many examples of extreme events occurring overseas. Brown (1988) listed 17 notable storms and 9 major floods which have occurred overseas.

Many of the events listed occurred in tropical regions and therefore the figures quoted are not entirely applicable to south-eastern Australia, and in particular to South Australia.

One notable exception in Oman in 1981 is referred to by Brown (1988). A flood of 1,150 m^3/sec occurred in a catchment of 370 km^2. This flood was produced by a rainfall of approximately 140 mm in 24 hours. This occurred in a month in which the mean rainfall was less than half a millimetre. In a period of record of 60 years the previous maximum fall was about 9 mm.

2.6.4. Extreme Events Recorded in Australia

Most of the large point rainfalls recorded in Australia have been in the tropics. The Crohamhurst storm on 2 February 1893 in Queensland produced 908 mm in 24 hours (Brunt, 1958). The Australian record 24 hour rainfall of 1,140 mm occurred at Bellenden Ker in north Queensland in 8 January 1979 (Shepherd and Colquhoun, 1985). There are however some notable exceptions of extreme point rainfalls occurring outside of the tropical regions (Bulletin 51).

Most of the records of high short duration rainfalls in Australia are near major cities because of the generally greater density of raingauges. This is particularly important because large portions of Australia are sparsely populated, and therefore it is probable that many extreme short duration storms occur without being recorded.

2.6.4.1. Failure of Briseis Dam in Tasmania

At about 4.30 am on 4 April 1929 Briseis Dam located in north eastern Tasmania failed due to overtopping. The failure resulted in 14 deaths and considerable property damage. This is the only example of a large dam failing in Australia.

The dam was constructed between 1924 and 1928 and the design flood was based upon the judgement of an experienced engineer. The flood of April 1929 far exceeded all observed floods in the region. Livingston (1993) estimated that the flood had an annual exceedance probability of 1 in 10^4.

The failure serves as an important warning of the possibly disastrous effects of the occurrence of extreme floods.

2.6.4.2. Extreme Short Duration Storms Outside of the Tropical Region

Pierrehumbert and Kennedy (1982) described the rationale for using adjusted United States thunderstorm data for the prediction of short duration PMP in Australia. Five notable point rainfalls are listed which occurred outside of the tropical and sub-tropical regions. A summary of short duration storms recorded in inland and southern Australia is also included in Bulletin 51.

With the exception of the Buckleboo Storm, all of these storms occurred over relatively densely populated areas and this reinforces the belief that most of the severe short duration storms which have occurred in Australia have not been recorded.

Spark (1992) examined the database of storms which have occurred over the Sydney Metropolitan Area. Storms of up to 6 hours in duration were examined and no recorded storms approached the PMP. The ratio of the PMP to recorded rainfalls was in the range from 2.5 to 3.1.

2.6.4.3. Dapto Storm of 18 February 1984

On 18 February 1984 a severe storm occurred over Dapto near Wollongong in NSW. The resulting floods caused considerable damage but no lives were lost.

The maximum recorded 24 hours rainfall was 796 mm at Wongawilly and approximately 75 percent of the rainfall occurred in the 8 hours to 1200 EDST 18 February. The rainfall recorded at Wongawilly is shown in Table 2.3.

Table 2.3 Rainfall Recorded at Wongawilly 18 February 1984 (source: Bulletin 51)

Duration (min.)	Rainfall (mm)
10	30
60	136
120	220
180	294
360	515
720	717

The rainfall set new Australian point rainfall records for durations between 8 and 17 hours (Shepherd and Colquhoun, 1985). The storm far exceeded the 5 severe short duration, non tropical storms referred to in Pierrehumbert and Kennedy (1982).

The storm resulted in an increase in the short duration PMP estimates for rough terrain. The greatest difference was 160 mm for an area of 25 km². It also led to the questioning of the procedure of distinguishing between rough and smooth terrain for the purpose of short duration PMP estimation (Shepherd and Colquhoun, 1985).

No thunderstorms were associated with this event. This is unusual as it is assumed that extreme short duration rainfall is produced by stationary thunderstorms.

2.6.4.4. Comparison of Recorded Point Rainfalls in Australia

Appendix II of Bulletin 51 summarises notable point rainfalls recorded in Australia on a state by state basis. These point rainfalls for Queensland, New South Wales, Victoria and South Australia are plotted in Figure 2.1. Where more than one point rainfall was recorded for a duration, the greatest rainfall was chosen. Figure 2.1 therefore is an approximate envelope of recorded rainfalls in Australia.

The recorded rainfall depths are similar for the shorter durations. As expected the rainfall in Queensland and New South Wales generally plot above those recorded in Victoria and South Australia for longer durations. This is because storms of a tropical nature are less common in the southern states of Australia.

It is important to note two point rainfalls recorded in South Australia for durations between 100 and 200 minutes. Both of these rainfall records occurred in the Barossa storm of 2 March 1983 which was influenced by tropical moisture. It is therefore clear that although tropical storms are more common in the northern regions of Australia, storms of tropical origins can cause extreme rainfalls in the southern states such as South Australia.

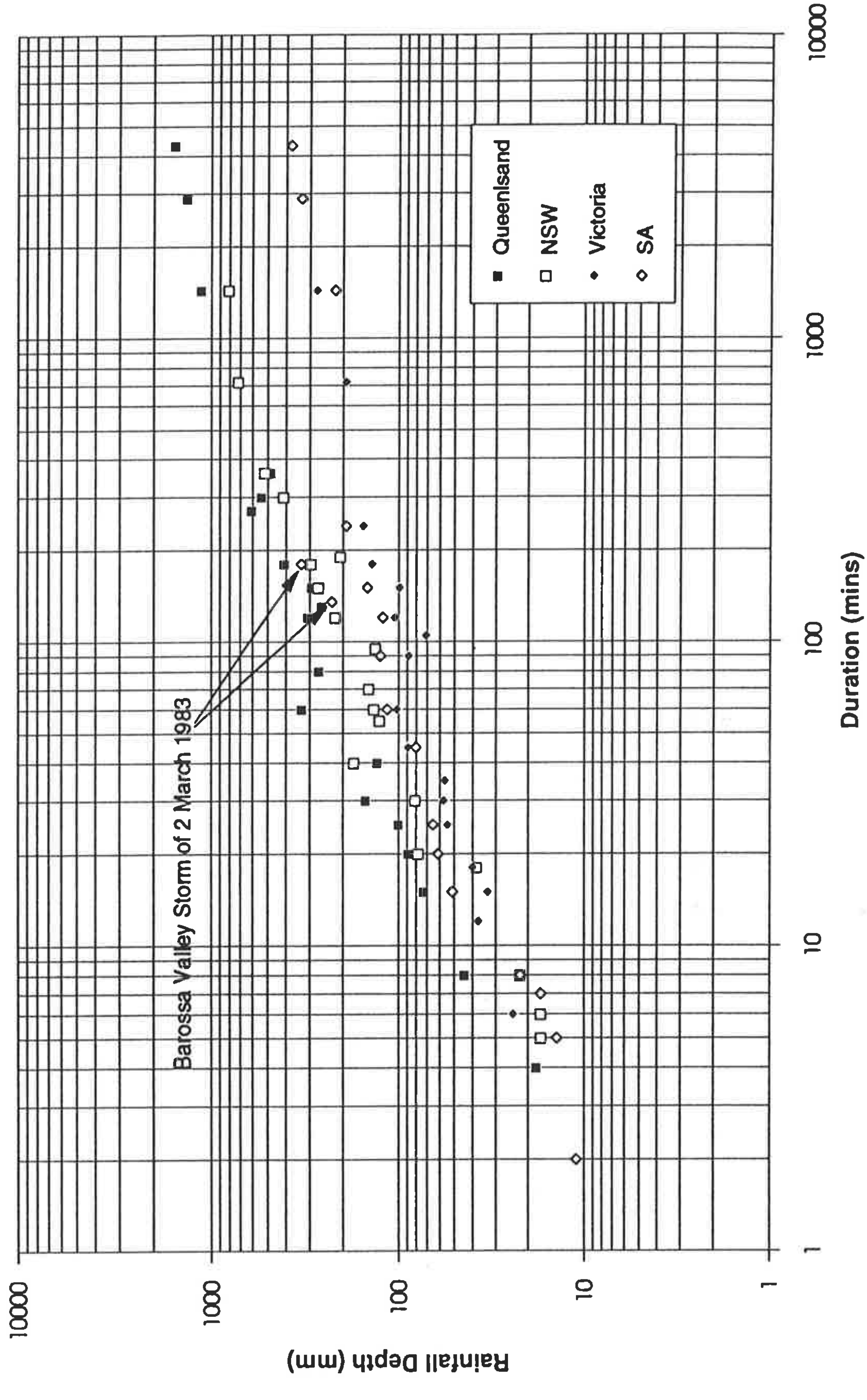


Figure 2.1 Comparison of Notable Point Rainfalls Recorded in Australia (source: Bulletin 51)

Chapter 3

Hydrometeorology of the Onkaparinga Catchment

3.1. Catchment Description

3.1.1. Introduction

The Onkaparinga River's source is near Mount Torrens in the Mt Lofty Ranges, 30 km east of Adelaide. It flows in a south westerly direction and enters the Gulf of Saint Vincent at Port Noarlunga.

There are two storages on the Onkaparinga River. The largest of these is the Mt Bold Reservoir which is used to supply water to the Adelaide Metropolitan Area. Water is not directly extracted from the reservoir but is released downstream to the second storage, Clarendon Weir, where it is diverted to the Happy Valley Reservoir.

The total area of the Onkaparinga Catchment is 557 square kilometres. The catchment area to Mt Bold Reservoir is 388 square kilometres and to Old Noarlunga 522 square kilometres. The Onkaparinga Catchment to Mt Bold is shown in Figure 3.1.

The main tributaries of the Onkaparinga River to Mt Bold Reservoir are Inverbrackie, Lobethal, Lenswood, Cox, Aldgate and Echunga creeks. Downstream of the reservoir the two main tributaries are Scott Creek and Baker Gully.

The major towns in the catchment to Mt Bold Reservoir are Lobethal, Balhannah, Hahndorf, Stirling, Aldgate and Bridgewater. Downstream of the reservoir, the

Onkaparinga River passes through Clarendon and through the outer suburbs of Adelaide of Old Noarlunga and Port Noarlunga.

3.1.2. Land Use

The Onkaparinga Catchment is used for many different activities. Part of the catchment is urbanised, while large portions are used for farming.

The different land uses employed in the Onkaparinga Catchment to Mt Bold Reservoir are shown in Appendix A. Some general classifications regarding land use are:

- The most common land use type in the catchment is grazing, both dairying and non-dairying.
- The dominant land use in the northern catchment is rotational potatoes and dairy pasture, while the north-western section of the catchment is characterised by perennial horticulture.
- Along the western side of the catchment there exist large areas of native vegetation, particularly surrounding Mt Bold Reservoir.
- The urbanised region generally lies in the Aldgate Creek Catchment which is located in the western portion of the catchment; stretching from Crafers to Bridgewater.
- The land use in the Baker Gully Catchment consists of vineyards and grazing.
- The Onkaparinga Gorge is used for recreational purposes.

3.1.3. Farm Dams

The number of farm dams in the Mt Lofty Ranges has dramatically increased in the last decade. Although these dams are generally small in volume, collectively they affect the hydrology of the catchments.

The number of farm dams in the Mt Bold Catchment has been estimated from GIS data as approximately 2,070. The total volume of these farm dams is approximately 8,800 ML which represents 20 percent of the storage capacity of Mt Bold Reservoir (Cresswell, 1993, pers. comm.).

The number of farm dams can affect the observed hydrology of the catchment. Large initial losses may be attributed to the presence of farm dams. A significant volume of rainfall may be required to fill farm dams before they spill and the surface runoff reaches the major streams which are being modelled.

Because farm dam spillways are generally designed for flows less than the 1 in 100 AEP event, the failure of these farm dams may have an effect on flows greater than the 1 in 100 AEP event, including the PMF.

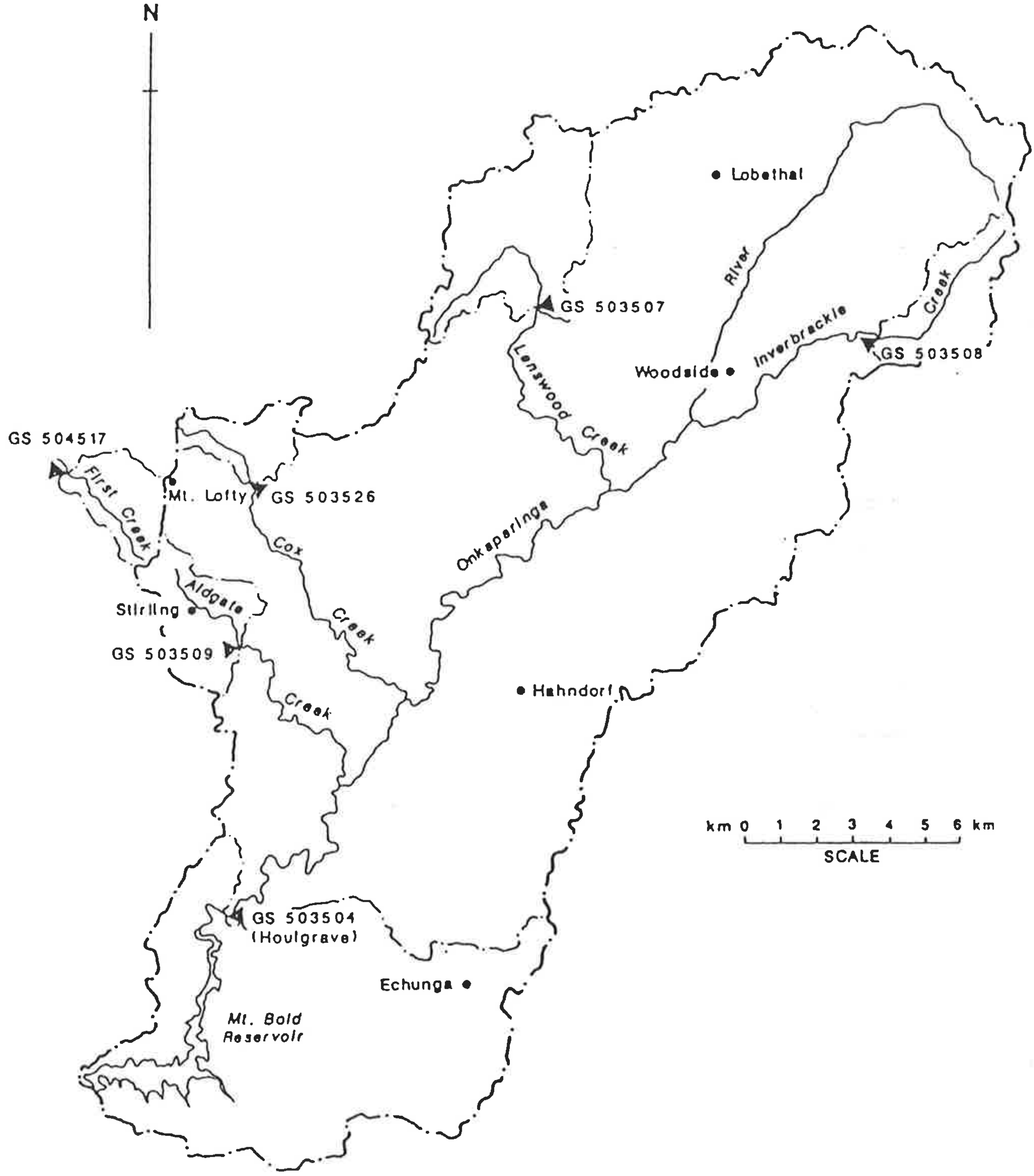


Figure 3.1 Onkaparinga Catchment to Mt Bold Reservoir (Source: Maguire et al., 1986)

3.2. Catchment Instrumentation

3.2.1. Storage of Data

Use was made of the data archiving system HYDSYS to store and manipulate all time series data. This system is used by both the Engineering and Water Supply Department (E&WS) and the Bureau of Meteorology (BoM).

The system allows the storage and retrieval of time series data. It also performs calculations such as baseflow extraction and flood frequency analysis. A complete description of the system was given in HYDSYS Pty Ltd (1992).

3.2.2. Pluviometers for the Onkaparinga Catchment

In order to calibrate runoff routing models and to study the behaviour of the catchment, it is necessary to record the variation of rainfall with time. The E&WS and the BoM operate a network of pluviometers in the Onkaparinga and surrounding catchments.

3.2.2.1. Engineering and Water Supply Department Pluviometers

The locations of the E&WS pluviometers are shown in Appendix B. The pluviometers and their respective periods of record are shown in Table 3.1.

Although Table 3.1 indicates the official commencement date of the pluviometers, in many cases only a portion of the recorded data has been processed. There are also periods of instrument failure.

Initially the pluviometer data were recorded on charts. This required that the charts be digitised in order to obtain the record of rainfall. Large portions of the pluviometer records have not been digitised, and it seems likely that unless specific funding is made available, these records will remain unprocessed.

Requests were made to the Scientific Services Branch of the E&WS for a number of pluviometer records to be digitised. Continuous records were not requested, but rather only those days which were required to calibrate the runoff model. A good response was obtained from the E&WS in the processing of the required data.

Unfortunately the early records at Echunga Creek and at Ackland Hill were recorded on punch tape. At present the E&WS do not possess the necessary equipment to retrieve data from a punch tape record. This record has remained unprocessed.

Table 3.1 E&WS Pluviometers for the Onkaparinga Catchment

Number	Station Name	Commenced	Ceased
AW426638	Mt Barker Effluent	18/03/86	-
AW503502	Scott Creek	08/03/91	-
AW503504	Houlgraves	10/12/91	-
AW503508	Inverbrackie Ck	13/09/84	-
AW503521	Gallasch Ck	30/06/77	04/11/82
AW503524	Vince Ck	08/06/82	01/04/87
AW503525	Sutton Ck	23/07/82	-
AW503529	Burnt Out Ck	12/01/78	17/11/88
AW503530	Kerber Ck	30/07/87	08/11/89
AW503531	Juers Ck	10/08/87	08/11/89
AW503532	Happy Valley Reservoir	29/09/88	-
AW503533	Echunga Ck	25/01/84	-
AW503534	Mt Bold (Island)	04/10/88	-
AW504550	Ackland Hill	07/02/85	-
AW504552	First Ck Mt Lofty	12/09/84	-
AW504558	Angas Ck Station	16/10/80	-
AW504559	Sixth Ck Cherryville	22/09/81	-

Note: "-" in the final column indicates that the pluviometer is still operational.

All of the E&WS pluviometers are now equipped with data loggers and in the future all recorded data will become available in a shorter time frame.

Although the pluviometer at Gallasch Creek commenced on 30/6/77, the chart record has been lost for a period in late 1979. This is most unfortunate because during this time there were three intense rainfalls which produced significant flows in the Onkaparinga River.

3.2.2.2. Bureau of Meteorology Pluviometers

The BoM have operated a pluviometer at the Lenswood Research Centre since 1972 which is administered by the CSIRO. There was also a pluviometer at Stirling (023843) which commenced in 1964 and was moved in 1985 to Heathfield.

The facilities for digitising pluviometer records are located in Melbourne. This results in lengthy delays.

The BoM have recently installed a network of pluviometers throughout the Torrens and Onkaparinga Catchments. Many of these pluviometers have been installed as part of the ALERT scheme for flood forecasting. The newly installed pluviometers are equipped with data loggers.

The locations of the BoM pluviometers in the Onkaparinga Catchment and surrounding region are shown in Appendix B. The pluviometers and their commencement dates are indicated in Table 3.2.

Table 3.2 Bureau of Meteorology Pluviometers for the Onkaparinga Catchment

Number	Station Name	Commenced
023101	Killara Park	01/08/89
023108	Longwood	26/07/89
023846	Belair Alert	05/09/88
023861	McLaren Flat	28/11/90
023862	Lobethal	16/07/92
023865	Stringybark	11/06/91
023866	Verdun - Sutton	16/07/92
023867	Ashton	31/01/91
023XXX	Eagle on Hill Alert	26/02/92

The pluviometer at Eagle on the Hill was originally at Crafers, but it was subjected to vandalism and then was stolen. A pluviometer was reinstalled at Crafers in February 1992.

3.2.3. Daily Read Raingauges for the Onkaparinga Catchment

The distribution of rainfall over the Onkaparinga Catchment is measured by the network of pluviometers combined with daily read raingauges. The distribution of daily read raingauges is shown in Appendix B.

Many of the raingauges are located at Post Offices. This unfortunately in many cases results in the rainfall only being recorded on weekdays.

There also exist many private unofficial raingauges in the Onkaparinga Catchment. These can be used to supplement the official raingauges.

3.2.4. Streamflow Gauging Stations in the Onkaparinga Catchment

The E&WS have a number of streamflow gauging stations in the Onkaparinga Catchment. The location of the stations is shown in Appendix B. The gauging stations which have digitised data are shown in Table 3.3. The dates refer to the time from which digitised data is available.

Table 3.3 E&WS Gauging Stations in the Onkaparinga Catchment which have Digitised Data

Number	Stream	Station Name	Area (km^2)	Commenced	Ceased
AW503500	Onkaparinga R	Clarendon Weir	441	19/09/37	-
AW503501	Onkaparinga R	Mt Bold Res	384	28/07/86	-
AW503502	Scott Ck	Scotts Bottom	26.8	27/03/69	-
AW503503	Baker Gully	WNW Kangarilla	45.6	11/04/69	26/06/89
AW503504	Onkaparinga R	Houlgraves Weir	321	17/04/73	-
AW503505	Dashwood Gly	Snow Hill	3.1	06/11/72	19/01/83
AW503506	Echunga Ck	U/S Mt Bold Res	34.2	22/03/73	17/09/76
AW503507	Lenswood Ck	Lenswood	16.5	18/05/72	22/06/89
AW503508	Inverbrackie Ck	Craigbank	8.4	17/05/72	-
AW503509	Aldgate Ck	Aldgate Rly Stn	7.8	13/07/72	22/06/89
AW503521	Gallasch Ck	Verdun	0.15	30/06/77	04/11/82
AW503522	Onkaparinga R	Noarlunga	522	27/06/73	14/07/88
AW503524	Vince Ck	Piccadilly Valley	0.65	08/06/82	01/04/87
AW503525	Sutton Ck	Piccadilly Valley	0.43	23/07/82	04/07/88
AW503526	Cox Ck	Uraidla	4.3	23/06/76	27/06/89
AW503528	Onkaparinga R	D/S Mt Bold Res	385	04/08/77	07/02/89
AW503529	Burnt Out Ck	U/S Mt Bold Res	0.56	12/01/78	17/11/88
AW503530	Kerber Ck	near Woodside	0.8	30/07/87	08/11/89
AW503531	Juers Ck	near Charleston	1.2	10/08/87	08/11/89

Note: The station at Mt Bold records reservoir level

It is evident from Table 3.3 that most of the gauging stations in the Onkaparinga Catchment have been closed. The usefulness of streamflow records for flood analysis increases with the length of record. It is therefore vitally important that the recent trend to close gauging stations is reversed.

The catchment areas listed in Table 3.3, were obtained from the schedule of hydrometric stations of the Scientific Services Group Hydrometric Data Unit. These catchment areas were checked against the catchment area included in the HYDSYS Database, and many discrepancies were discovered. Calculation of catchment areas using ARC-INFO resulted in yet another set of catchment areas. It is important that these inconsistencies are rectified. In light of the recent digitising of the Onkaparinga sub-catchments, it is recommended that the values from ARC-INFO be adopted as the areas.

The gauging station located just downstream of Mt Bold Reservoir (AW503528) was installed to measure low flows. The station only accurately records flows of less than 5 m^3/sec. For flows of greater than approximately 5 m^3/sec the small weir becomes drowned. This means that it is unsuitable for use in flood studies.

The station at Mt Bold Reservoir (AW503501) records reservoir level. Continuous measurement of the reservoir level only commenced in July 1986. Before this time the reservoir level was only measured daily and a reading taken whenever an adjustment was made to the spillway gates or outlet valves. These records make it difficult to estimate outflows from the reservoir prior to this time.

Although digitised data at Clarendon Weir is only available from September 1937, flow records commenced in 1889. These early records of flow in the Onkaparinga River have been used in the flood frequency analysis undertaken in Section 7.3.

3.2.4.1. Houlgraves Weir (AW503504)

Houlgraves Weir has a continuous digitised record from April 1973. The catchment to the gauging station covers 83 percent of the catchment to Mt Bold Reservoir. Because the continuous recorder has been operational at Mt Bold Reservoir since July 1986, Houlgraves Weir provides the best estimate of inflow to Mt Bold Reservoir between April 1973 and July 1986

The rating of Houlgraves Weir is considered to be good, with 89 different gaugings performed up to the end of September 1992. A summary of the gaugings of over 50 m^3/sec is shown in Appendix B. Only one of the gaugings has been recorded on the rising limb of the hydrograph. This is important because the ratings for the rising and falling limbs of the hydrograph may be different.

3.2.4.2. Clarendon Weir (AW503500)

The Clarendon Weir was completed in 1896. It primarily serves to divert water from the Onkaparinga River to Happy Valley Reservoir for water supply. The weir has a spillway 61 metres long, a vertical upstream face and a stepped downstream face.

Stage records are available from 1889, but it is not until September 1937 that the record has been processed and stored in a digital form. From May 1978 the quality of record is considered to be good.

A theoretical rating has been calculated for the weir, but this has not been verified by a series of gaugings. Only one gauging has been performed. This was done in July 1981 and only measured 35.1 m^3/sec.

There are problems with undertaking gaugings at Clarendon Weir:

1. Gaugings cannot be taken near to the weir, for safety reasons, and therefore gaugings must be performed well upstream; and

2. The rating is affected by the diversion of water. Although this is not significant during flood events, this could significantly affect the rating during low flows.

3.2.4.3. Old Noarlunga (AW503522)

The gauging station at Old Noarlunga is important because it has the largest catchment of all the gauging stations with digitised data in the Onkaparinga Catchment. It is therefore the preferred station at which to calibrate hydrologic models. The gauging station at Old Noarlunga was closed however in 1988.

The control at the station is a log lying across the channel. The log is considered to be stable with little change in the cease to flow level having occurred. The log only acts as a control for low flows. For larger flows the log is submerged and the channel cross section determines the rating.

There were problems in determining a stable rating for low flows due to the unstable bed. In the latter years that the gauging station was operated, the channel downstream of the log became overgrown, particularly in the summer months. The vegetation downstream of the control obstructed flows to such an extent that the control section was drowned out at very low flows.

There have been a total of 58 gaugings performed at the site. Most of these were for low flows however, and only 8 gaugings occurred for flows which exceeded 1 m^3/sec. A summary of these 8 gaugings is included in Appendix B. The maximum gauging was at a flow of 72.5 m^3/sec.

The rating curve was extrapolated to a gauge height of 10.30 metres which represents a flow of 203 m^3/sec. The extrapolation of the rating represents a sizeable extrapolation of the rating, but it was the best available method of determining flows at the site.

A single rating curve for all periods was used in this study. This is because the differences in ratings appear to only apply for low flows (Good, 1993, pers. comm.).

3.2.4.4. Bureau of Meteorology Gauging Stations

As part of its flood warning scheme, the BoM have installed some manually read staff gauges without instrumentation on the Onkaparinga River.

There are two gauge locations for the Lower Onkaparinga River. The first of these is situated just downstream of the road bridge at Old Noarlunga. The second is located at the Commercial Road Bridge at Port Noarlunga. No regular readings are taken at these stations.

There are also three stations in the catchment to Mt Bold Reservoir. Staff gauges are located at the bridge at Balhannah, at Hacks Bridge at Mylor and at Verdun. Data was not obtained for these stations.

3.3. Previous Reports

There have been a number of reports and papers which have dealt with hydrological modelling of catchments in the Mt Lofty Ranges, and in particular the Onkaparinga Catchment and Mt Bold Reservoir.

The hydrology of the Onkaparinga Catchment was described in Kotwicki (1984). The report described routing of synthetic flood hydrographs through the Mt Bold Reservoir to obtain information on the operation of the spillway gates. A RORB runoff model was generated to Mt Bold Reservoir and calibrated using historical events. A flood frequency analysis was undertaken on the existing record at Houlgraves Weir. The PMF was estimated using an estimate of the PMP from the BoM.

BC Tonkin and Associates (1986) included the modelling of ten catchments in the Adelaide Hills using the unit hydrograph method. Two of the catchments (Lenswood and Aldgate Creeks) are located within the Onkaparinga Catchment to Mt Bold Reservoir. This study involved the selection of suitable events which had hydrograph and pluviometer data available, and the analysis of different loss models.

In 1985 the catchment hydrology of 17 of the dams operated and owned by the E&WS was analysed by BC Tonkin and Associates (1985). This did not include the Mt Bold Reservoir. The flood frequency curves for each reservoir were calculated and the inflow hydrographs determined for specified return periods. The design flood hydrographs were then routed through the storages. Since this report, the methods of estimating PMP for longer durations have been changed by the BoM and this may result in increased estimates of the PMF.

Parsons (1987) summarised the adequacy of existing spillways in South Australia. Mt Bold Reservoir was one of the three dams that were assigned a high priority for further evaluation.

The feasibility of a flood inundation study of the Onkaparinga River below Mt Bold Dam was described by Lange Dames and Campbell Australia Pty Ltd (1990). The aim of this study was to determine the likely nature and extent of flooding sufficient to define the boundaries of a more detailed study.

The flood hydrology component of the inundation study was discussed in Daniell and Hill (1993c).

3.4. History of Flooding in the Onkaparinga River

A study of the floods in the Onkaparinga River was undertaken. This involved examination of all of the relevant hydrological and meteorological data collected by the E&WS and the BoM.

A search was also made of newspaper articles. Daniell and Hill (1993c) contained a chronological listing of the relevant newspaper articles and a commentary of the major aspects of the articles.

A summary of floods in the Onkaparinga River prior to 1933 was made in E&WS (1933) and this is shown in Appendix C.

It has been regarded that floods only occur at Noarlunga when there is a high tide. Generally this is the case but on at least three occasions (19/8/1851, 5/10/1867 and 29/8/1971) flooding occurred in the absence of a high tide (Daniell and Hill, 1993c).

3.5. Extreme Storms for the Onkaparinga Catchment

It is important in studying the hydrology of a catchment that the season or months of highest rainfall are determined. This is because the time of year will have an effect on the catchment losses adopted.

The time of year is even more important when flows are to be determined downstream of a reservoir as the flows downstream are affected by the storage of the reservoir. This is especially relevant for Mt Bold Reservoir because for a large proportion of the year it is drawn down.

In the winter months there exist more examples of strong mechanisms to produce rainfall but because the atmosphere is at a lower temperature, there is less moisture vapour available. In summer the air is warmer hence more moisture is able to be stored in the atmosphere, but there are less examples of mechanisms that produce rainfall. PMF is therefore most likely to occur during the summer months; November through to March.

Although the most intense rainfalls are likely to occur during the summer months, there are examples of heavy rainfalls occurring outside of this period. The storms of April 1889 and August 1992 in the Onkaparinga Catchment are two examples of rainfalls which occurred outside of this period.

BC Tonkin and Associates (1985) listed the major point rainfalls recorded in South Australia. This list has been reproduced in Appendix C. From this list of point rainfalls, Figure 3.2 was generated which shows the months of the year in which the major point rainfalls have been recorded. The storm of August 1992 was included.

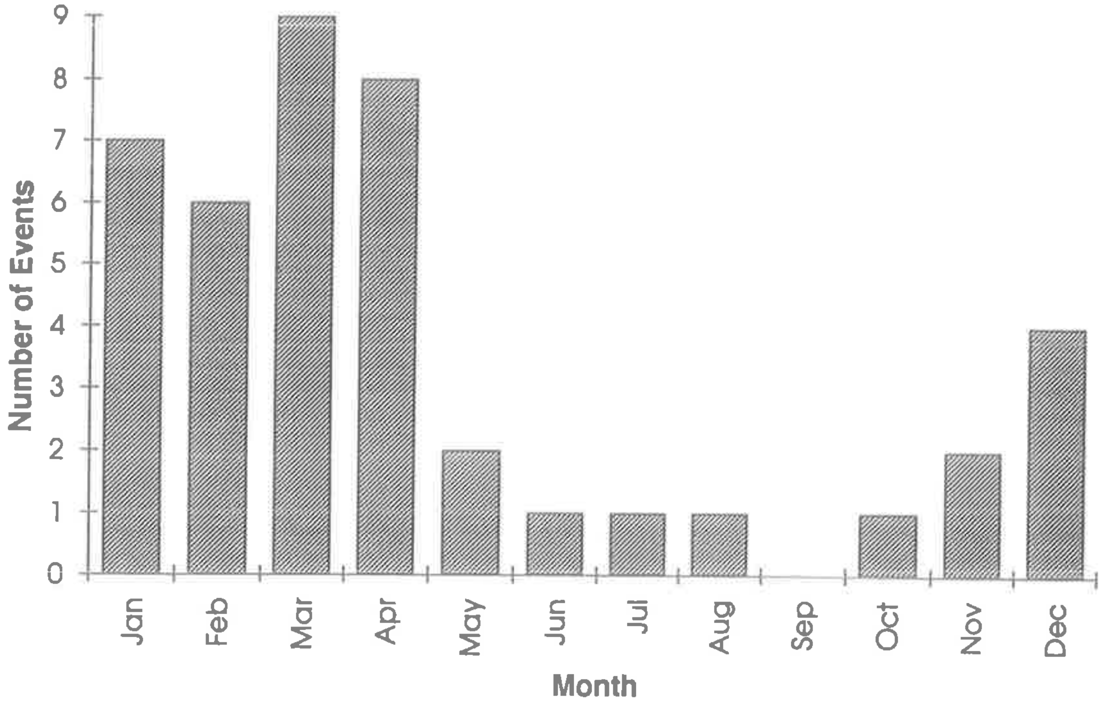


Figure 3.2 Distribution of Major Point Rainfalls for South Australia (Source: BoM)

Although severe rainfalls are more likely in the summer months, most of the high flows in the Onkaparinga River are recorded in the late winter and early spring. This is a result of the joint probability of intense rainfall and a wet catchment. Figure 3.3 shows all of the peak flows recorded at Houlgraves Weir between 17/4/1973 and 20/12/1992 that were greater than 75 m^3/sec.

Appendix C contains a summary of the peak flows at Houlgraves Weir, Clarendon Weir and Old Noarlunga.

3.6. Three Major Storm Events in the Vicinity of the Onkaparinga Catchment

Three examples of notable rainfall events in the vicinity of the Onkaparinga Catchment are described. The three events occurred in different seasons and were a result of different meteorological conditions.

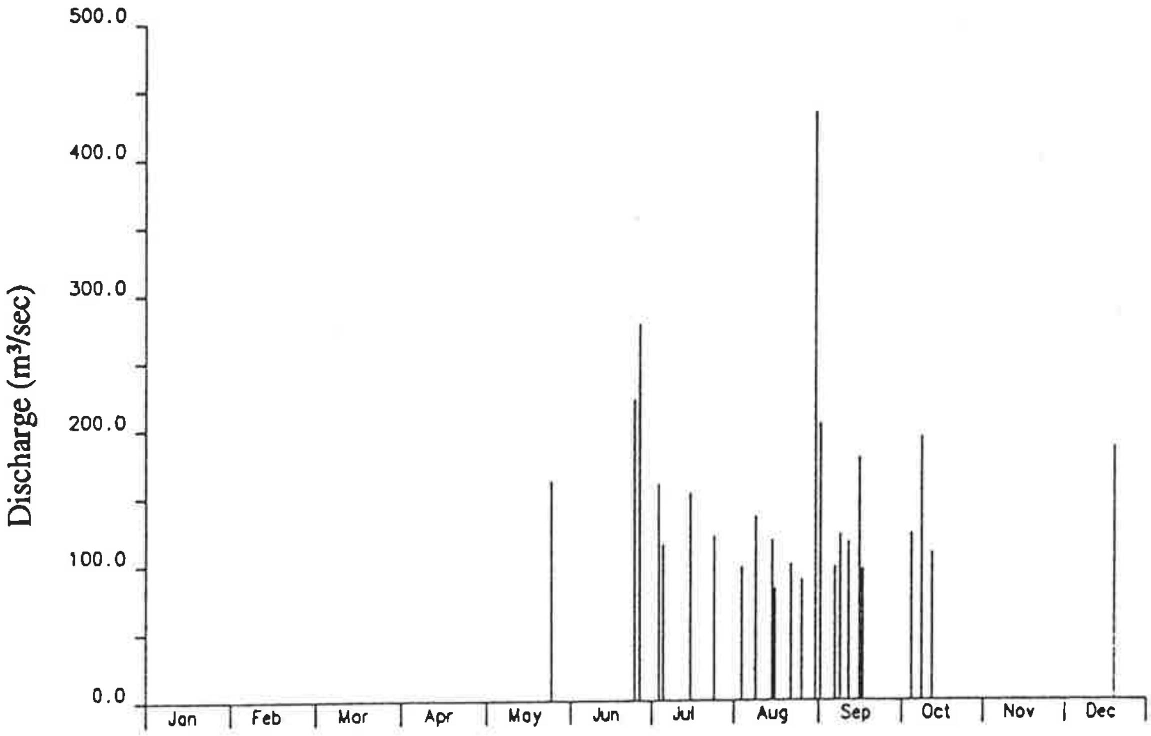


Figure 3.3 Peak Flows at Houlgraves Weir Greater than 75 m^3/sec

3.6.1. April 1889

During 1889, three severe storms were recorded in the Adelaide region. The first of these occurred in January while the other two occurred in April (2nd and 17th). The rainfall which fell on 17 April 1889 resulted in the largest recorded flood in the Onkaparinga River.

A survey party in the vicinity of Clarendon pegged the progress of the flood. It was estimated that the peak flow was approximately 680 m^3/sec (E&WS, 1933).

The rainfall was centred over Stirling West which recorded a 24 hour rainfall to 9am on April 17th of 208 mm (8.20 inches). The rainfall recorded at five different stations is shown in Table 3.4. The 24 hour rainfall to 9am 17 April 1889 recorded at Stirling West is one of the greatest daily rainfalls recorded in South Australia.

From Table 3.4, it is clear that the heavy rainfall to 9am on the 17th was preceded by pre-frontal rain. The rainfall which resulted in most of the runoff was produced by embedded convective cells. The storm was accompanied by very strong westerly winds which increased the uplift.

Table 3.4 Daily Rainfall Records (mm) - April 1889

	15 th	16 th	17 th	18 th
Lobethal	14	28	144	6
Woodside	20	26	70	3
Stirling West	11	28	208	7
Hahndorf	7	13	89	5
Clarendon	4	26	138	13

From eye-witness accounts, most of the rainfall fell in the 12 hours to the commencement of the 17th (Purton, 1993, pers. comm.). Unfortunately because of the sparse meteorological network in place at the time, little is known of the meteorological conditions which led to the heavy rainfall.

Because of the importance of this event, the storm was modelled using a RORB model of the Onkaparinga Catchment to Clarendon Weir. The results of the modelling were shown in Daniell and Hill (1993c). The calculated peak flow was very sensitive to the adopted temporal pattern.

3.6.2. August 1992

Heavy rainfall on 30 August 1992 produced widespread flooding in the Mt Lofty Ranges and surrounding districts. The water inundated many houses and the flooding resulted in both loss of life and property. There was extensive damage to infrastructure such as roads, dams, culverts, bridges and weirs.

The rainfall was centred over the western branch of the Onkaparinga River, with very large flows being recorded in both the Onkaparinga and Torrens Rivers. Very large flows were also experienced in the South Para System. The floods, in many cases, exceeded all previously recorded flows (Hill, 1992).

The peak flow recorded at Houlgraves Weir was 431 m^3/sec, which was the largest on record. The largest of the gaugings was at a flow of 330 m^3/sec on the falling limb of the hydrograph. The gauging was within 2% of the extrapolated rating curve (Leaney, 1992, pers. comm.).

3.6.2.1. Meteorological Situation

The rainfall of 30 August 1992 was produced by an intense low which formed south of Adelaide. This mechanism produced a continuous supply of warm, moist air which was fed into the Adelaide region. The Mt Lofty Ranges caused orographic lifting which resulted in large falls occurring just over the ranges.

Figure 3.4 shows the synoptic chart prior to the storm. A low front approached the Adelaide region from the south-west. This resulted in strong westerly winds.

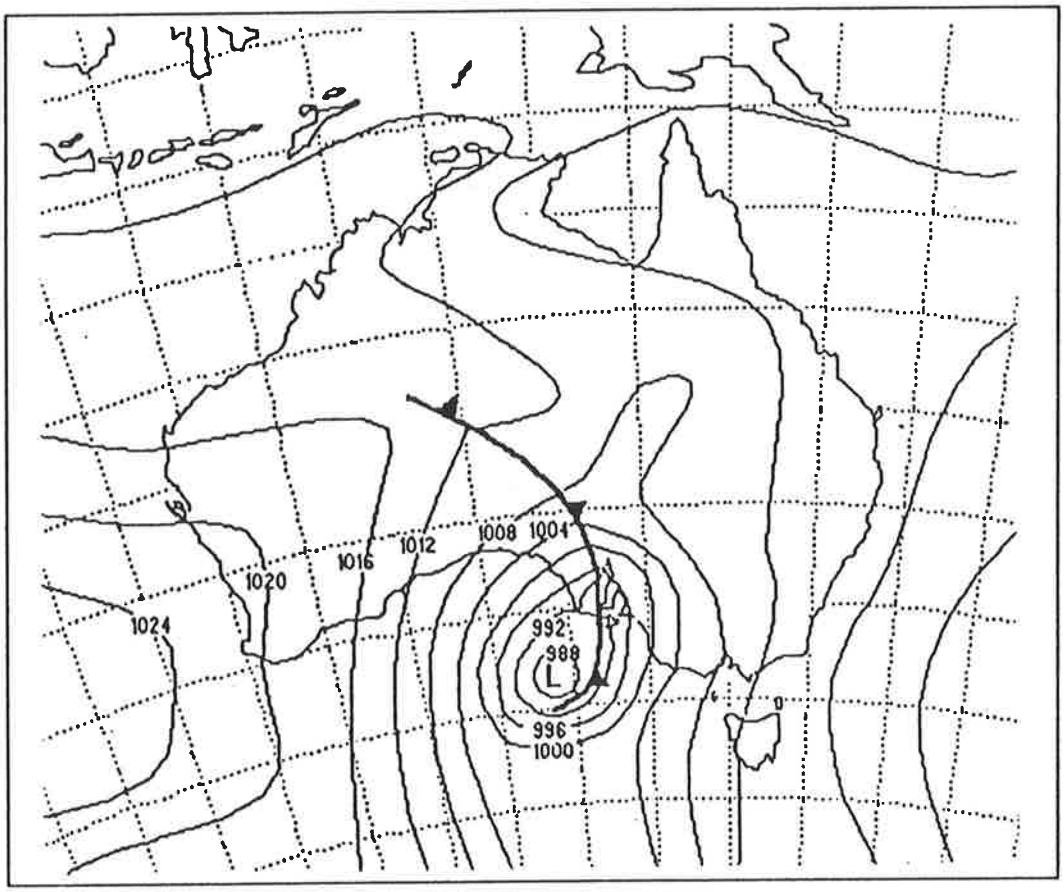


Figure 3.4 Synoptic Chart for 0900 CST 29 August 1992

3.6.2.2. Recorded Rainfall

The official BoM daily-read raingauges were supplemented with private records. This resulted in the distribution of rainfall over the catchment being accurately recorded. The heaviest 3 day total to 9am on 31 August 1992 was at Lenswood which recorded 174 mm. The distribution of rainfall over the Onkaparinga Catchment is shown in Figure 3.5.

The variation of rainfall with time over the Onkaparinga Catchment is shown in Appendix D. From midnight to 0200 hours on 30 August 1992 light to moderate rainfall fell over the catchment. In the two hours to 0400 hours a burst of rainfall was located over the centre of the catchment. In the 2 hours to 0600 hours, a heavy burst of 35 mm was recorded over Ashton.

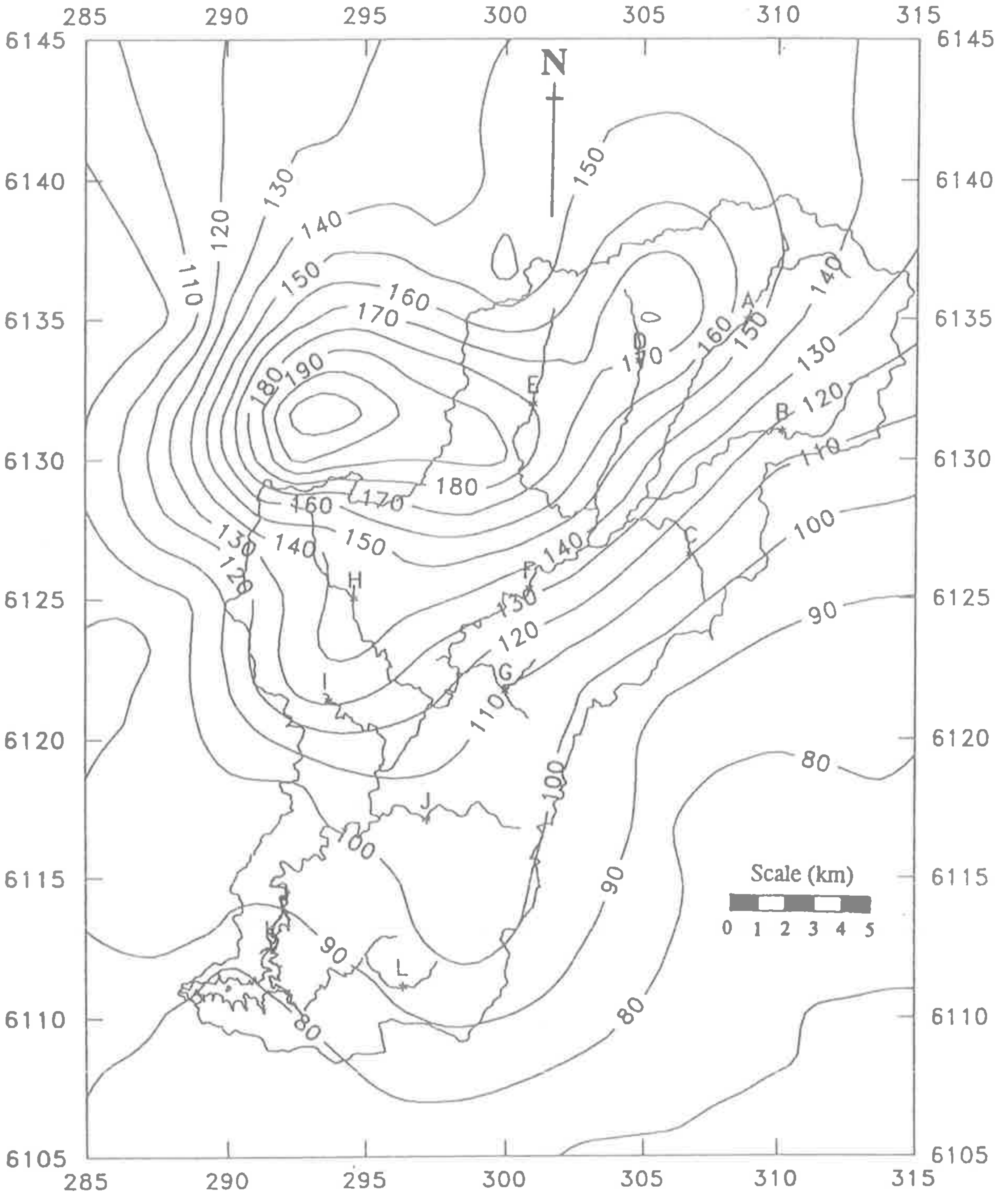


Figure 3.5 5 Day Rainfall (mm) to 0900 hours 2/9/1992

Figure 3.6 shows the recorded hyetograph at four BoM pluviometers.

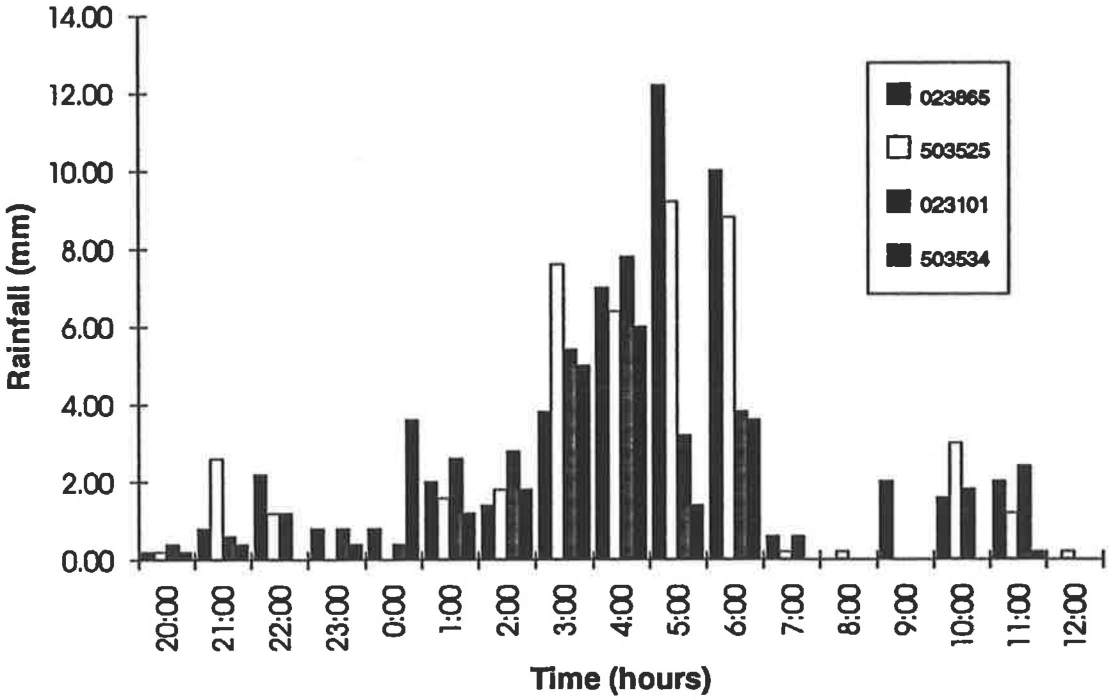


Figure 3.6 Comparison of Pluviograph Records for the Onkaparinga Catchment - 2000 hours 29/8/92 to 1200 hours 30/8/92

Figure 3.7a shows the display from the weather radar at 0220 hours on 30 August 1992. A broad area of mainly light to moderate rain fell over the Adelaide region.

Figure 3.7b shows the radar image at 0450 hours on 30 August 1992. The light rainfall band was replaced by a narrow band of moderate to intense rainfall orientated in a north westerly direction. By 0600 hours the rain band had passed east of the Mt Lofty Ranges and only scattered showers remained over the Mt Lofty Ranges (BoM, 1992).

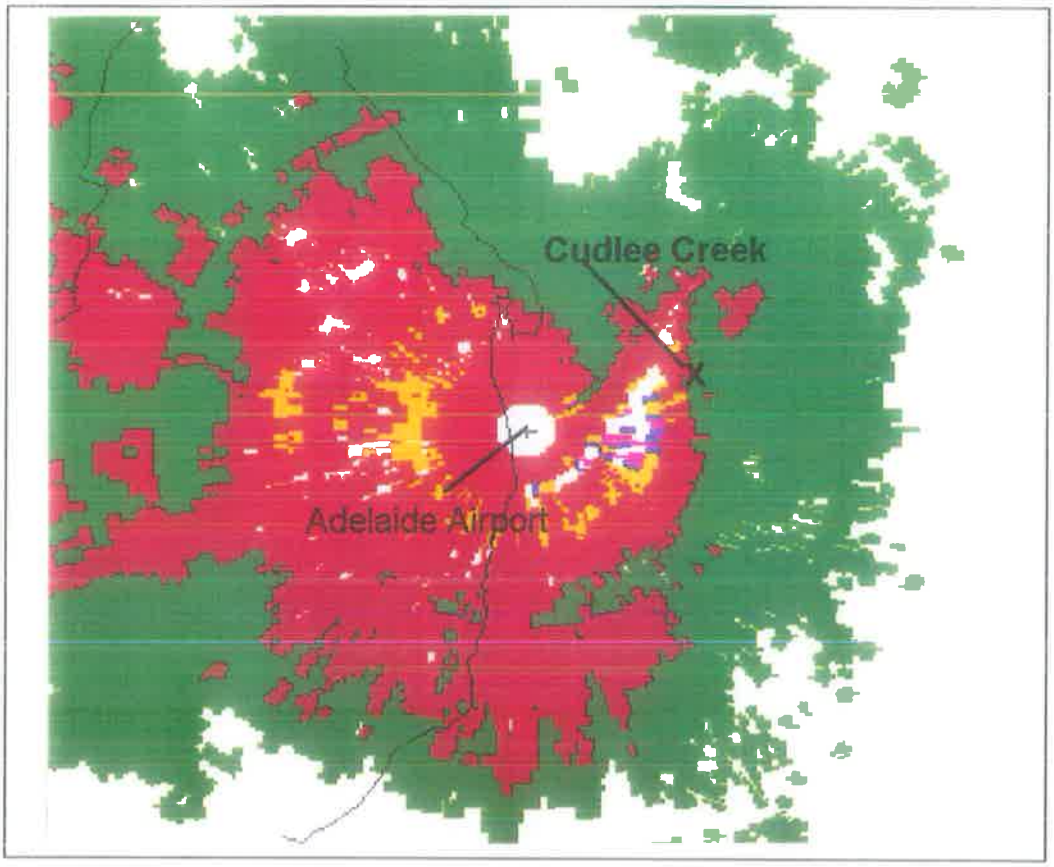


Figure 3.7a Weather Radar Image for 0220 hours 30 August 1992 (source: BoM)

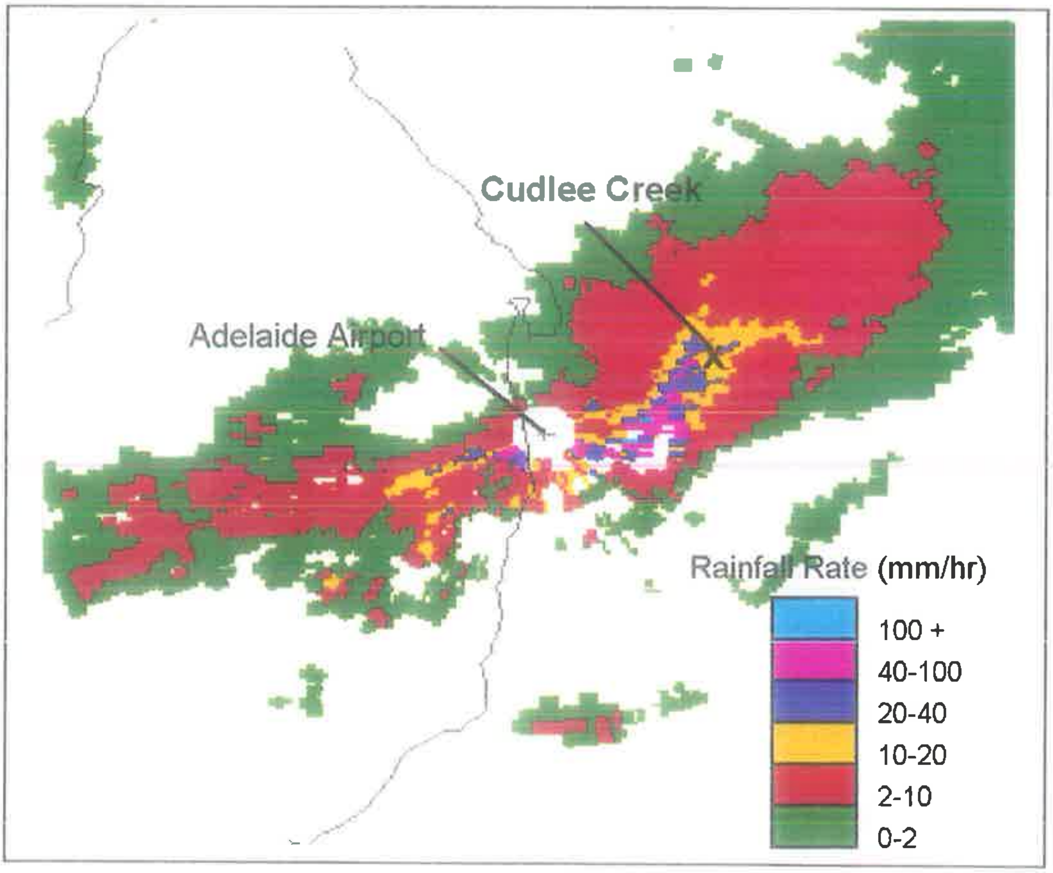


Figure 3.7b Weather Radar Image for 0450 hours 30 August 1992 (source: BoM)

3.6.3. December 1992

A severe thunderstorm on 18 December 1992 resulted in the loss of a life and damage to property in Callington and Kanmantoo to the east of the Onkaparinga Catchment.

The storm was a result of a different meteorological situation than that which produced the storms of August 1992. The storm of December 1992 was produced by a north easterly low level moist air stream. There was atmospheric instability and the topography acted as the trigger. There was also synoptic scale convergence producing greater uplift. Although large falls were recorded near Kanmantoo, very little rain was recorded in Adelaide. This is because the low level stream was descending from the Mt Lofty Ranges.

The pluviometer record at Kanmantoo for 17 and 18 December 1992 is shown in Appendix D. From these records, it is evident that during the 17 December 1992 steady rainfall fell at approximately 5 mm/hr. Two heavy bursts were recorded on 18 December 1992, at 6 am and 10 pm. The one hour rainfalls in the two bursts were 24.5 mm and 20 mm respectively. The second of these bursts caused a large amount of runoff.

Figure 3.8a and 3.8b show the radar image at 0620 hours and 2300 hours on 18 December 1992. Figure 3.8b also indicates heavy rainfall occurring over the Lower Onkaparinga River.

This event resulted in only moderate flows in the Onkaparinga River as the heavy rainfall was centred to the east and to the north of the Onkaparinga Catchment. It is important to note however that the meteorological conditions which produced this event were such that the rainfall could have occurred over any of the Mt Lofty Ranges Catchments; in particular the Onkaparinga Catchment (Watson, 1993, pers. comm.).

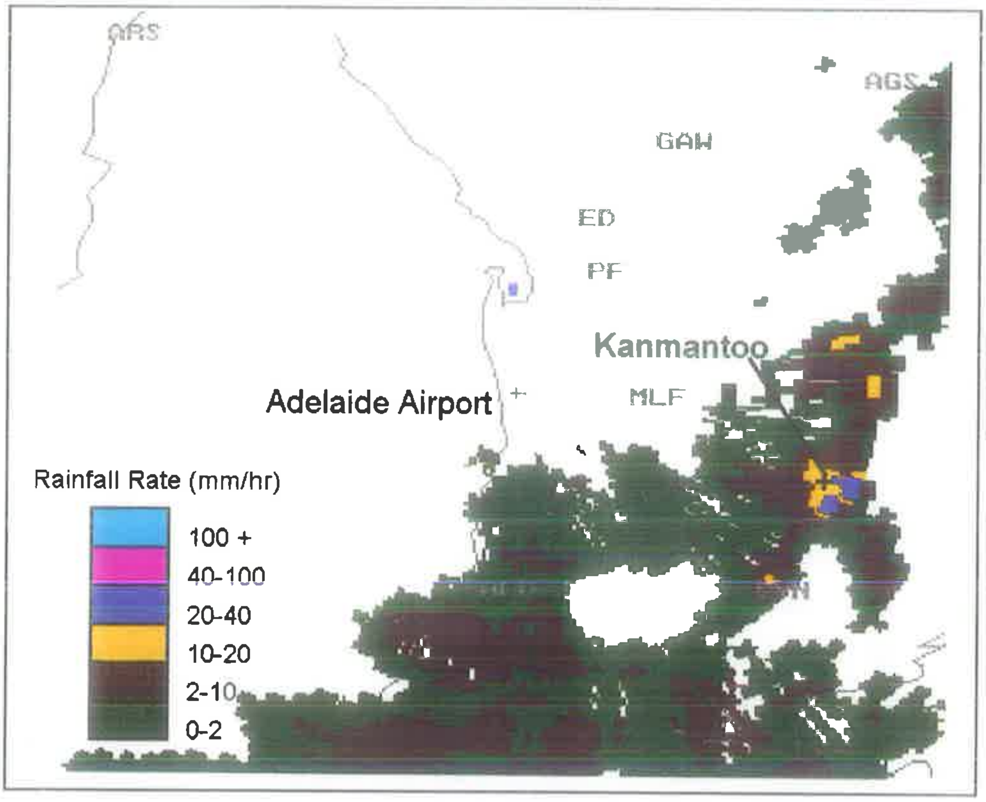


Figure 3.8a Weather Radar Image from Adelaide Airport for 0620 hours 18 December 1992 (source: BoM)

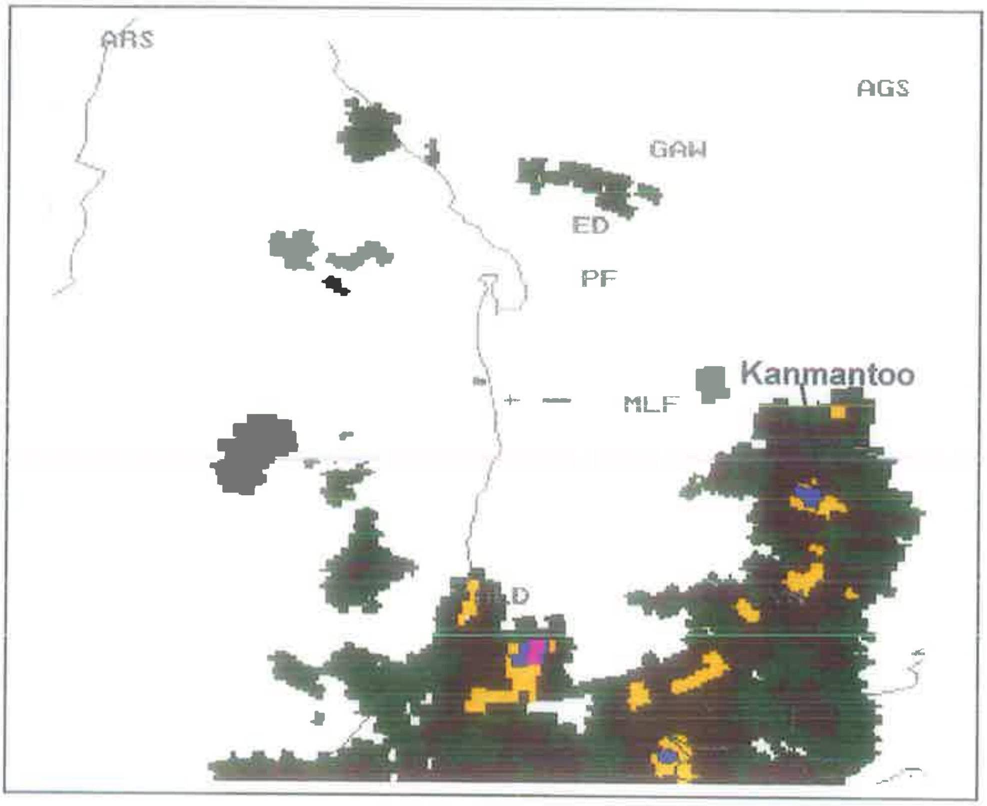


Figure 3.8b Weather Radar Image from Adelaide Airport for 2300 hours 18 December 1992 (source: BoM)

Chapter 4

Mt Bold Reservoir

4.1. Introduction

Mt Bold Dam is located on the Onkaparinga River 23 kilometres SSE of Adelaide. The dam was constructed between 1932 and 1938. Mt Bold Reservoir is not used to directly supply Adelaide with water, but water is released to Clarendon Weir where it is diverted to Happy Valley Reservoir and to the southern suburbs of Adelaide.

The spillway was raised and associated modifications made between 1961 and 1963. During these modifications, the reservoir crest was raised from EL 240.50 to EL 244.00 and 8 vertical lift gates installed. These gates are 2.9 metres high and 7.32 metres long (Design Services, 1982).

The raising of the reservoir crest increased the estimated storage capacity above the minimum draw off level to 47,300 ML. The storage capacity has since been revised to 45,900 ML.

4.2. Mt Bold Reservoir Gates

4.2.1. Gate Operation

The outflow from Mt Bold Reservoir is regulated by a gated spillway and by two outlet valves. The gates are operated with the intent to safely pass flood waters while maintaining the reservoir level at the full supply level of 41.40 metres (EL 246.90).

The gates can be operated from the control room or from the walkway above the gates. Depressing the appropriate switch opens or closes the gates by an increment of 100 mm. In the control room a digital display for each gate indicates the opening in centimetres. The maximum gate opening is determined by a limit switch.

Schurer (1986) stated that the operators of dams can help prevent the failure of dams by careful gate operation. It is noted however that most dam operators are under trained and possibly lack motivation to effectively carry out their safety roles.

Lewin (1986) also examined the control of spillway gates during floods. The correct operation of the spillway gates can greatly increase the safety of the structure. This paper recommended automated control of spillway gates with a number of redundancies included in the system.

4.2.1.1. Official E&WS Gate Operation Policy

A copy of the current official operating policy for the Mt Bold Reservoir spillway gates is shown in Appendix E.

The operating policy is designed to match the outflow with inflow to achieve the maximum operating level of a gauge height of 41.325 (EL 246.825). This is performed by monitoring the rate of rise of the reservoir.

The guidelines require that the gates remain in the lowered position until the reservoir level reaches a gauge height of 41.10 (EL 246.60), when two gates are opened 100 mm. If the reservoir level continues to rise, the gates are opened by various increments until the reservoir level stabilises or begins to fall. A similar procedure is used for the case of a falling reservoir level.

Kotwicki (1984) noted that the advantages of this method were that it was simple and precise, but that it was unsatisfactory for larger floods and that it also excluded pre-releasing.

4.2.1.2. Current Operation of Mt Bold Reservoir Gates

Several telemetered stations have been installed throughout the catchment. Rainfall is measured and reported at Echunga Creek, Inverbrackie Creek and Scott Creek. The stage height at Houlgraves Weir is also reported.

The stage height in the Onkaparinga River is measured at Balhannah by the BoM using manually read gauge boards (Section 3.2.4.4). The stage heights are then phoned through to the BoM which relays the flows to the operators via the phone. It is believed that the peak flow takes approximately 6 hours to reach the reservoir from Balhannah (Bailey, 1993, pers. comm.).

Pre-releasing is currently being used in an attempt to mitigate the downstream flooding. The pre-release is based upon the reported flow in the Onkaparinga River at Balhannah obtained from the BoM. No pre-release is made according to rainfall.

The decision on the magnitude of the pre-release is currently made by the operators, taking into consideration the recorded peak flow at Balhannah and the downstream conditions. It is important that the pre-release is made considering the weather forecasts, rain reports and knowledge of general catchment conditions (Ballard, 1984).

The policy of pre-releasing can successfully reduce flooding downstream (Munro et al., 1967). Figure 4.1 shows the reduction of the peak outflow from Mt Bold Reservoir that was achieved using pre-releasing for the event of 8 October 1992.

If the practice of pre-releasing is to continue, it is important that a pre-release policy is determined which is based upon hydrological data. The practice of pre-releasing would be better implemented if a telemetered station was installed higher in the catchment, possibly at Balhannah.

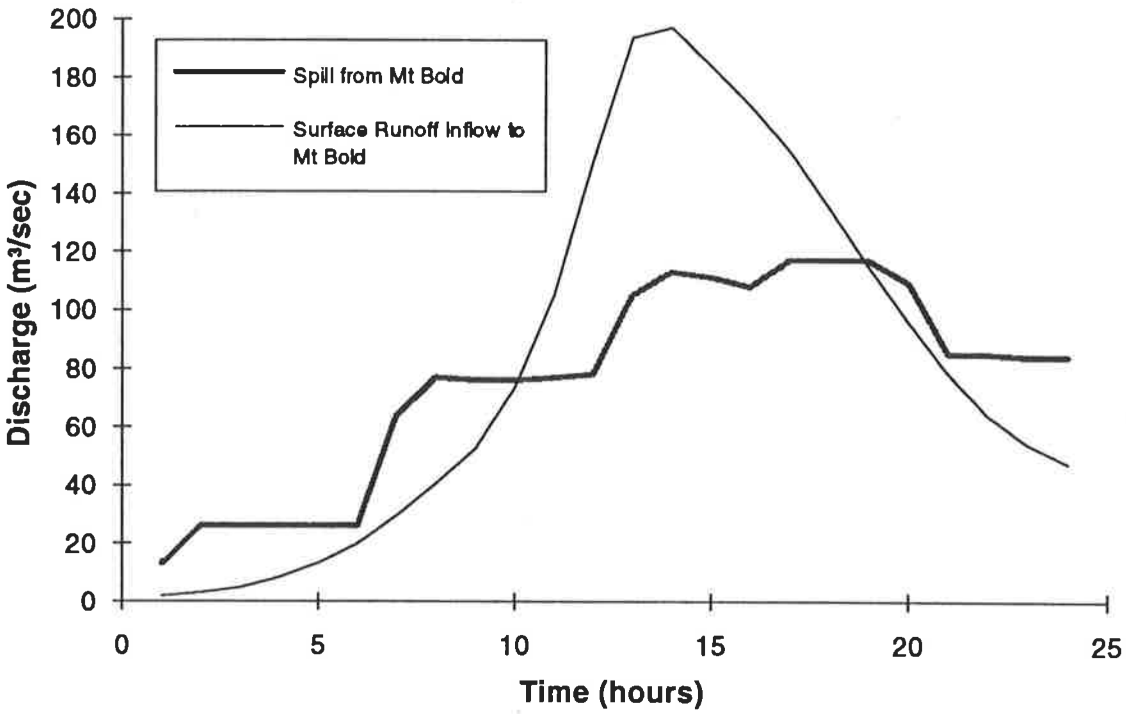


Figure 4.1 Effect of Pre-releasing from Mt Bold Reservoir for Peak Flow Recorded on 8/10/92

4.2.2. Maximum Gate Openings

In order to rout the PMF and other extreme floods through Mt Bold Reservoir the storage - discharge relationship was extended. For this to be calculated, the maximum gate settings were required.

4.2.2.1. Previous Studies

Simpson (1990) examined the gates and their operation. A field trip was undertaken by Simpson on 27 February 1990. Operating procedures were discussed and measurements were taken of gates 4 and 5.

The digitally displayed gate opening was checked by measuring the gate opening using a pole and tape. A measurement was taken at both ends of the gate.

A discrepancy was discovered between the maximum gate opening for gate 4 and that obtained from the operator's records. The observed maximum gate opening of 2.74 metres was significantly different to the recorded 3.50 metres. The other 7 gates were not opened to their maximum settings, but it was recommended that the discrepancy be investigated.

A problem was also identified with the digital display for gate 4. During the opening of the gate, the digital display did not indicate the correct opening between a gate opening of 1 and 2 metres. A '1' did not appear as the first digit. It was recommended that this problem be rectified as soon as possible.

4.2.2.2. Field Trip August 1992

A field trip was conducted by the author on 7 August 1992, with the intent of observing the gate operation procedures and to measure the maximum gate openings for each of the 8 gates.

The gates were operated from within the control room. Each gate was opened by increments of approximately 100 mm to its maximum opening. The same problem with the display for gate 4 that was identified by Simpson (1990) was observed.

The gates were opened to their maximum opening which was determined by the limit switch. Gate 3 opened significantly more than the others and was not opened to its maximum for fear of damaging the overhead walkway.

The digital display was then checked by measuring the gate openings using a tape. The gate opening could not be measured directly, but had to be inferred from the movement of the top of the gate relative to the overhead walkway.

In Table 4.1 the measured maximum gate openings are compared to those in the operator's records from a test conducted in November 1985. The digital display and the measured values corresponded within acceptable limits for all of the gates with the exception of gate 5. The error in the digital display was 0.33 m.

Table 4.1 Maximum Gate Openings for Mt Bold Reservoir Gates (August 1992)

	Maximum Gate Opening (m)		
Gate	Measured	Digital Reading	Operator's records
1	2.78	2.75	3.45
2	2.83	2.80	2.35
3	3.78	3.75	3.65
4	2.73	2.70	3.50
5	2.88	2.55	3.10
6	3.13	3.10	3.20
7	2.93	2.90	3.25
8	3.03	3.00	3.40

Note: Gate 3 was not opened to its maximum for fear of damaging the overhead walkway.

Figure 4.2 shows the reservoir gates open to their maximum settings on 7 August 1992.

Figure 4.2 Mt Bold Reservoir with Gates set to Maximum Openings (August 1992)

4.3. Calculation of Spillway Rating

4.3.1. Previous Studies

Several recent reports have estimated the discharge from the gated spillway of Mt Bold Reservoir.

4.3.1.1. Ebsary (1987)

Ebsary (1987) concluded from a water balance study that the actual outflow from the spillway was about 70 percent of the outflow based on the current discharge rating for the gates. This was confirmed by the reservoir operators who had experienced difficulty in stabilising the reservoir using the rating curve to set outflow to inflow.

The problem of gated spillway flow was considered as comprising of two elements; free overflow and flow through a rectangular orifice. For small gate openings, relative to the head, the flow was assumed to act as flow through a rectangular orifice. For large gate openings, the flow was assumed to behave as free overflow. The point at which the behaviour changed was considered difficult to determine analytically and hence the discharge for both methods was calculated and the lowest value chosen as the outflow.

The two expressions used were:

free overflow: $$Q = C.L_e.H^{1.5} \tag{4.1a}$$

flow through rectangular orifice: $$Q_g = u.Q\left[1-\left(1-\frac{G_0}{H}\right)^{1.5}\right] \tag{4.1b}$$

where: $C = 2.28 - 0.357\left(1.33 - \dfrac{H}{H_d}\right)^{1.47}$

H_d = design head for crest profile;

L_e = effective weir length for each gate allowing for end contractions;

$u = 1.14$ for $H/H_d < 0.5$;

$u = 1.14 - (H/H_d - 0.5)/5$ for $0.5 < H/H_d < 1.0$;

$u = 1.04$ for $H/H_d > 1.0$;

G_o = gate opening; and

H = depth of water above spillway crest.

The assumed design head was the full supply level of 2.896 metres. In most instances, only the daily 8.00 a.m. reservoir levels were available. The outflows were therefore calculated using a reservoir level which was calculated by interpolating the daily read values. This was not considered to produce significant errors because the fluctuations in head were small when compared to the design head, and because the reservoir level was generally at full supply level during times of spill.

The calculated values of outflow were then multiplied by 0.7. This was done because a water balance study indicated that the theoretical calculations overestimated the outflow by approximately 30 percent.

4.3.1.2. Simpson (1990)

A preliminary study was undertaken by Simpson in 1990 to investigate the reliability of the Mt Bold spillway rating curve.

The original design of the Mt Bold Spillway in 1958 was based on the USBR Boulder Canyon Final Project Report. This was however only known to give an approximate result as the profile of the upstream face of Mt Bold was not covered in this report.

The 1958 calculations assumed a design head H_d of between 3.29 and 3.31 metres. A more comprehensive analysis however, showed that the design head was greater than this value. The report also concluded that it was not possible to determine the exact design head for the spillway.

The report concluded that the perceived overestimation of the discharge was due to:

1. the assumption of a design head which was too low, and
2. the US Army Corps of Engineers had changed their method of calculating partial gate opening discharges.

4.3.1.3. Simpson (1991)

The objective of this phase of the study was to determine if the rating curve for partial gate openings could be corrected analytically or whether a physical hydraulic model study of the spillway would be necessary.

The design head for Mt Bold spillway was estimated to be 3.80 metres. This was based on a visual comparison of the existing parabolic profile and the US Corps of Engineers Chart 312. The average decrease in discharge compared with the existing rating curve was between 9 and 13 percent.

An investigation of various model study reports concluded that one or more model studies of the Mt Bold spillway should be carried out. The recommended model study never eventuated.

In addition to the model study, it was also recommended that the actual discharges for small gate openings be measured. With some modifications the weir downstream could be used for this purpose.

Simpson et al. (1991) summarised the investigations undertaken to identify the perceived errors in the existing rating for the Mt Bold spillway. The paper included a description of the gates and their operation.

4.3.2. Ungated Spillway Flow

The calculation of flow over a gated spillway, first requires that the flow over an ungated spillway be calculated. Two different expressions for the flow over an ungated spillway were considered.

The first method considered was that recommended by United States Bureau of Reclamation (1977) and is shown in Equation 4.2.

$$Q = 0.522\ C\,L\,(H_e)^{1.5} \tag{4.2}$$

where: Q is the discharge (m³/sec);

C is a variable coefficient of discharge;

L is the effective length of the crest (m); and

H_e is the total head on the crest (m), including the velocity of approach head.

The discharge coefficient is dependent upon (USBR, 1977):

- the depth of approach;
- the relation of the actual crest shape to the ideal nappe shape;
- the upstream face slope;
- downstream apron interference; and
- downstream submergence.

At the design head, C is designated C_o, and is determined from the ratio of p, the height of the upstream face and H_d, the design head. For calculations based on heads other than the design head, the coefficient of discharge is determined as a ratio of C_o, the coefficient of discharge at the design head.

The velocity of approach head, h_d, was calculated for the design head to be 0.00611 metres. This is only a very small percentage of the design head and it was therefore not included in subsequent calculations (Simpson, 1991).

A second method of calculating the discharge over an ungated spillway proposed by Hager and Bremen (1988) was considered and is shown in Equations 4.3 and 4.3a.

$$Q = C_d b\left(2gH^3\right)^{0.5} \tag{4.3}$$

where: b is the effective spillway width (m);
g is the gravitational constant;
H is the head (m); and
C_d is the coefficient of discharge.

Hager and Bremen (1988) recommended a new formula for evaluating C_d. This formula was confirmed by Hager (1991). The expression for C_d is shown in Equation 4.3a.

$$C_d = \frac{2}{3\sqrt{3}}\left(1+\frac{4X}{9+5X}\right) \tag{4.3a}$$

where: X is the dimensionless head ($=H/H_d$)

The two different methods were used to calculate the ungated discharge from Mt Bold Dam for 8 gates fully open and the reservoir level at the design head (3.80 m) and at the full supply level (2.896 m). The results are shown in Table 4.2.

Table 4.2 Comparison of Ungated Discharges

	Head	Q (eqn 4.2)	Q (eqn 4.3, 4.3a)	Difference
H_d	3.80 m	916 m³/sec	921 m³/sec	0.5%
FSL	2.896 m	594 m³/sec	594 m³/sec	0.0%

The two methods give comparable results and therefore the method suggested by Hager (1991) was adopted as this did not require any values to be read from tables or graphs and hence could easily be incorporated into a spreadsheet.

4.3.3. Gated Spillway Flow

The discharge for a gated spillway was given by Hager and Bremen (1988) as Equation 4.4.

$$\frac{Q_g}{Q_D} = \left[X_o^{3/2} - (X_o - Z_l)^{3/2}\right]\left(\frac{1}{6}+Z_l\right)^{1/9} \tag{4.4}$$

where: Q_g is the discharge over a gated spillway;
Q_D is the discharge at the design head;
X_o is the dimensionless head($=H_o/H_d$);
Z_l is the dimensionless gate opening ($=z_l/H_d$);
H_o is the head on the gated spillway;
H_d is the design head; and
z_l is the gate opening.

Equation 4.4 for gated spillway discharge is valid until the gate is no longer immersed in fluid. For greater gate openings, ungated spillway flow prevails and Equation 4.3 applies. Equation 4.4 is only valid for $X < 2.5$ and Equation 4.3a is only valid for $X < 2$.

4.3.3.1. Horizontal Contraction

If the piers and abutments are shaped to cause a side contraction of the flow, the effective length of the spillway will be less than the total physical length. The effective length of the spillway is calculated using USBR (1977):

$$L_{eff} = L - 2(NK_p + K_a)H_e \tag{4.5}$$

where: L_{eff} = effective length of the crest;
L = total length of the crest;
N = number of piers;
K_p = pier contraction coefficient;
K_a = abutment contraction coefficient; and
H_e = total head on crest.

For the Mt Bold spillway, K_p has been estimated to be 0.02 and K_a has been estimated to be 0.1 (Simpson, 1991).

The gate operation procedures are such that the gates are opened in pairs, starting from the middle two gates (Appendix E).

The effective width for 2 ,4 or 6 gates open is therefore given by the expression:

$$L_{eff} = 7.315N_G - 2(0.02N_G)H_e \tag{4.5a}$$

where: N_G = number of gates; and
H_e = total head on crest.

The effective width for 8 gates open is:

$$L_{eff} = 58.520 - 0.480H_e \tag{4.5b}$$

4.3.3.2. Calculated Rating

The rating curve was calculated using the equations suggested by Hager and Bremen (1988). The gated flow discharge was calculated using Equation 4.4, and the ungated discharge was calculated using Equation 4.3.

The calculated rating is shown in Figure 4.3. Gated flow prevails until the gates opening is 0.77 of the head. At this point ungated spillway conditions occur.

At the point at which gated spillway flow is replaced by ungated spillway flow, there is a discontinuity which is shown in Figure 4.3. This discontinuity is a result of the increased coefficient of discharge. This phenomenon is discussed in more detail in Chapter 11, where its implications on the overtopping of the dam crest are outlined.

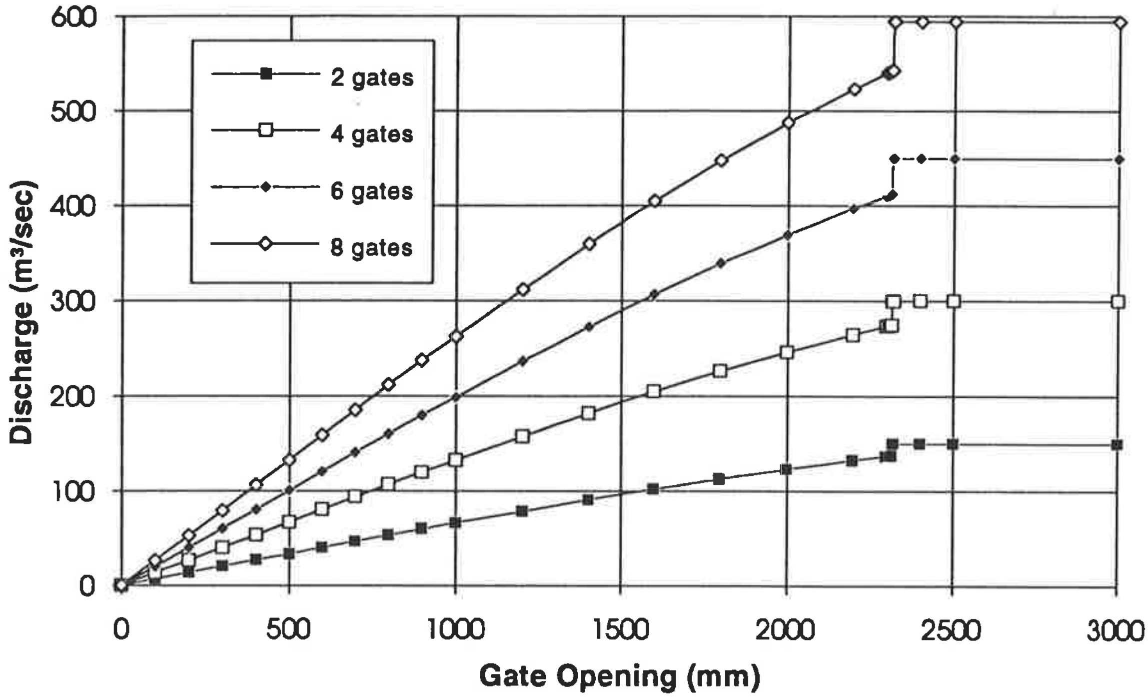


Figure 4.3 Rating Curve for Mt Bold Spillway for Full Supply Level

4.4. Recommendations for the Operation of the Spillway Gates

From the discussions in the previous sections, the following recommendations are made:

1. The new rating calculated in Section 4.3 should be adopted as the rating for the spillway and supplied to the operators, until such time as a more comprehensive hydraulic study can be undertaken. Note should be made that the rating changes with head, and therefore a single rating for all heads is not appropriate.
2. There needs to more care taken in the recording of the operation of the gates. This will assist any further hydrological or hydraulic studies involving the dam spillway.
3. The current operation of the gates which involves pre-releasing to mitigate downstream floods, should be addressed. If undertaken correctly, pre-releasing can mitigate the downstream flooding. If pre-releasing is to continue, it is important that the procedures are documented and are embraced as official policy.

4. The limit switches on the gates should be reset to allow all of the gates to open to a maximum opening of at least 3.8 metres. At present the maximum gate openings vary, but are less than this amount. The increase in the maximum gate opening will increase the maximum capacity of the spillway and hence will reduce the chance of overtopping of the dam.

5. The observed discrepancy between the measured gate openings and those from the digital display should be checked and the problem rectified.

The E&WS is planning to upgrade the gate operation at Mt Bold Reservoir which will ensure more reliable gate operation and this will address the above recommendations (Parsons, 1993, pers. comm.).

4.5. Mt Bold Outlet Valves

The flow in the Onkaparinga River below Mt Bold Reservoir is maintained via the two outlet valves.

Since the completion of the dam in 1938 there have been 5 different outlet valves in operation. Table 4.3 shows the outlet valves and their abbreviations (Ebsary, 1987).

Table 4.3 Mt Bold Reservoir Outlet Valves

OLD1:	Northern Needle Valve	(1938 - March 1981)
OLD2:	Southern Needle Valve	(1938 - August 1984)
TEMP:	Temporary Valve	(January 1982 - March 1984)
NEW1:	Northern Jet Valve	(November 1984 - present)
NEW2:	Southern Jet Valve	(December 1984 - present)

Ebsary (1987) determined the following relationships for the outlet valves:

OLD1: $$Q_{est} = 0.208N^{1.008}H^{0.31} \tag{4.6a}$$

OLD2: $$Q_{est} = 0.327N^{0.928}H^{0.333} \tag{4.6b}$$

TEMP: $$Q_{est} = 16.92H^{0.614} \tag{4.6c}$$

NEW1: $$Q_{est} = 0.96p^{1.056}H^{0.512} \tag{4.6d}$$

NEW2: $$Q_{est} = 1.552p^{0.923}H^{0.52} \tag{4.6e}$$

Where: Q_{est} is the estimated valve discharge (ML/day);
N is the valve opening (number of turns);
p is the valve opening (percentage open); and
H is the reservoir level (metres).

4.6. Total Outflow from Mt Bold Reservoir

Gauging station AW503528 is located under the original timber access bridge (now abandoned), just downstream of Mt Bold Reservoir. The station only measures low flows (less than about 5 m^3/sec) and it was therefore not suitable for determining flood outflows from the dam.

A spreadsheet was constructed which calculated the total instantaneous outflow from Mt Bold Reservoir. This is shown in Appendix E.

The spreadsheet could be used to calculate the discharge for any physically valid gate opening and head. The head is constrained by the limit of Equation 4.3a which calculates the discharge coefficient. This equation is only valid for $X < 2$. This implies that the analysis is only valid for heads which are less than 7.60 metres (Section 10.2).

4.7. Storage Elevation Relationship

The raising of the reservoir crest between 1961 and 1963 increased the storage capacity above the minimum draw off level to an estimated 47,300 ML. This storage was based upon the contour survey conducted before the construction of the dam.

In 1977 a survey of the storage was undertaken, and a 1:5000 Bathymetric map constructed. The storage capacity was revised from 47,300 ML to 45,900 ML. The new cross sections were compared to the original, and little siltation was discovered (Will, 1993, pers. comm.).

In order to rout the PMF through the reservoir it was necessary to extrapolate the storage elevation relationship. This was done by fitting Equation 4.6 to the storage elevation relationship.

$$S = 1.531(GH)^{2.769} \qquad (4.6)$$

where S is the storage in ML; and

GH is the gauge height in metres (>30m).

The extrapolated storage elevation relationship is shown in Figure 4.4.

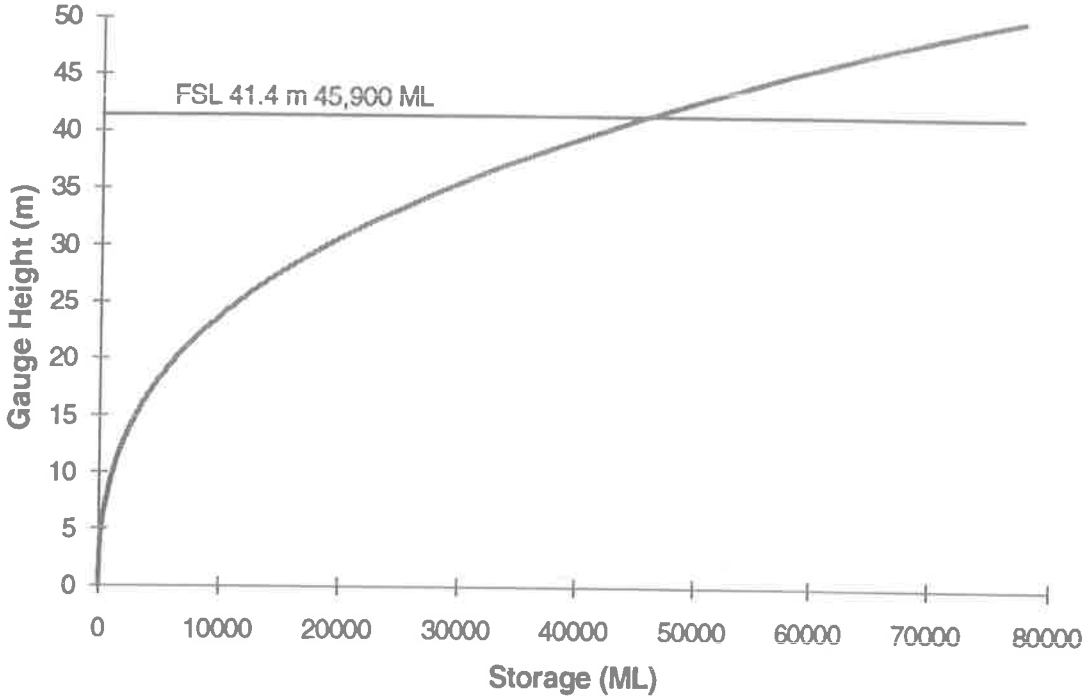


Figure 4.4 Storage Elevation Curve for Mt Bold Reservoir

Chapter 5

Design Rainfalls

5.1. Rainfall in the Onkaparinga Catchment

The Onkaparinga Catchment and the Adelaide region are influenced by two distinctly different rainfall mechanisms:

1. In the summer, heavy rainfalls are generally a result of tropical weather being advected from north-western Australia. This gives rise to generally intense short duration storms.

2. In winter, the heavy rainfalls are usually a result of rainfall produced by low frontal systems coming across the Great Australian Bight. These storms are generally less intense and of longer duration.

The rainfall pattern is winter dominant. The first significant falls generally arrive in April or May. June, July and August are usually the wettest months. From November to March rainfall is slight (Burrows, 1989).

5.2. Design Intensity Frequency Duration Rainfall Data

IEAust (1987) includes a method of obtaining intensity-frequency-duration (IFD) curves for use in design for all regions of Australia. These procedures produce, *"accurate, temporally and spatially consistent, IFD design rainfalls"* (IEAust, 1987).

The use of single station rainfall records for design rainfall purposes was considered to be unreliable and not temporally or spatially consistent. A generalised method was therefore developed.

The generalised procedure makes use of maps of isopleths of rainfall intensity, as this smooths sampling errors and best accounts for poor quality data. The method is based on a log Normal distribution.

The method in IEAust (1987) allows IFD curves to be constructed for durations from 6 minutes to 72 hours. The data used to develop these procedures included both daily read raingauges and pluviometers. The pluviometer records were scarce in comparison to the daily read raingauges. For example, at the time of the development of IEAust (1987) in NSW, there were 989 daily read raingauges which had periods of record which exceeded 30 years. There were however only 45 pluviometers in NSW that had a period of record greater than 13 years (Canterford et al., 1985b).

The generalised method in IEAust (1987) was established using 600 pluviometers with more than 6 years of data supplemented by 7,500 daily read rainfall gauges with more than 30 years of data (IEAust, 1987).

In order to supplement the short duration records, statistical methods were used to relate the 1 and 12 hour duration records to daily rainfall records and other meteorological and physical parameters.

5.2.1. Generalised IFD values Using IEAust (1987)

The procedures included in IEAust (1987) were used to obtain estimates of the design IFDs. A spreadsheet was set up to calculate the IFD values for any location.

Appendix F shows the values calculated for IFDs for the complete range of durations and average recurrence intervals (ARIs).

5.2.2. Comparison of Design IFDs for the Onkaparinga Catchment

Table 5.1 shows the comparison of IFD values obtained for Balhannah, Stirling PO, Old Noarlunga and Inverbrackie Creek with ARIs from 2 to 100 years for storm durations of 3, 6 and 24 hours. Balhannah is centrally located in the Onkaparinga Catchment and was considered representative of the catchment. Stirling PO, Old Noarlunga and Inverbrackie Creek are on the extremities of the catchment and should therefore give the greatest variation from the values at Balhannah.

It can be seen from Table 5.1 that the maximum variation is in the 24 hour duration values for the 100 year ARI and this is 12.8 percent for the Stirling PO. Generally there is no significant differences between the IFDs for Old Noarlunga and Balhannah. The IFDs for Stirling PO are all higher. This is expected because it is in this region where the highest rainfalls in the catchment occurs. The design temporal patterns and intensities for the Onkaparinga Catchment are shown in Appendix F.

Table 5.1 Comparison of IFD Rainfalls for the Onkaparinga Catchment with those calculated at Balhannah

3 hour duration

ARI (yrs)	Balhannah (mm/hr)	Stirling (mm/hr)	difference (%)	Old Noar. (mm/hr)	difference (%)	Inverbrackie Ck (mm/hr)	difference (%)
2	9.20	9.78	6.3	9.00	-2.2	9.2	0.0
5	11.63	12.27	5.5	11.54	-0.8	11.63	0.0
10	13.25	13.93	5.1	13.29	0.3	13.25	0.0
20	15.51	16.26	4.8	15.71	1.3	15.51	0.0
50	18.73	19.57	4.5	19.21	2.6	18.73	0.0
100	21.37	22.29	4.3	22.14	3.6	21.37	0.0

6 hour duration

ARI (yrs)	Balhannah (mm/hr)	Stirling (mm/hr)	difference (%)	Old Noar. (mm/hr)	difference (%)	Inverbrackie Ck (mm/hr)	difference (%)
2	6.13	6.53	6.5	5.90	-3.8	6.13	0.0
5	7.59	8.09	6.6	7.47	-1.6	7.59	0.0
10	8.55	9.12	6.7	8.53	-0.2	8.55	0.0
20	9.91	10.58	6.7	10.02	1.1	9.91	0.0
50	11.82	12.63	6.9	12.16	2.9	11.82	0.0
100	13.38	14.31	7.0	13.94	4.2	13.38	0.0

24 hour duration

ARI (yrs)	Balhannah (mm/hr)	Stirling (mm/hr)	difference (%)	Old Noar. (mm/hr)	difference (%)	Inverbrackie Ck (mm/hr)	difference (%)
2	2.59	2.82	8.9	2.40	-7.3	2.55	-1.5
5	3.15	3.46	9.8	3.01	-4.4	3.1	-1.6
10	3.50	3.88	10.9	3.41	-2.6	3.46	-1.1
20	4.02	4.48	11.4	3.99	-0.8	3.97	-1.2
50	4.74	5.32	12.2	4.80	1.3	4.69	-1.1
100	5.32	6.00	12.8	5.47	2.8	5.27	-0.9

5.2.3. Site Specific IFDs

Many of the pluviometers in the Onkaparinga catchment and in the surrounding areas now have 7 or 8 years more record than at the time of the development of IEAust (1987). Analysis of the pluviometer records in the catchment was therefore undertaken to compare the IFDs with those generated from the generalised procedures of IEAust (1987).

This was done using the HYDSYS database. Several different bucket sizes were examined for the analysis, with a 2 minute bucket being chosen. The lengths of the pluviometer records varied from 7 to 15 years. The stations used for review are shown below in Table 5.2.

Table 5.2 Length of Pluviometer Records Analysed

Location	Length of Record (years)
Stirling PO.	15
Inverbrackie Creek at Craigbank	8
Sutton Creek at Piccadilly Valley	9
Sixth Creek at Cherryville	7

The Stirling record may have been used in the original generalised analysis by the BoM as this record runs from 1965 to 1980. Further analysis should now be undertaken with the extra stations for this area. The method of extending the daily rainfall records to the shorter durations could also now be done with a much larger data base.

In Appendix F the IFDs obtained from IEAust (1987) are compared to the IFDs calculated from the pluviometer records.

5.2.4. Spatial Pattern

The design rainfalls procedures in IEAust (1987) assume uniform rainfall over the entire catchment. Many of the events used for the calibration of the model had a typical spatial pattern, with the isohyets centred over the higher portion of the catchment close to Mt Lofty and Mt Bonython.

The problem of spatial patterns for particular storms was not addressed. It is felt that the inclusion of rainfall spatial patterns in general flood analysis should be further researched.

5.3. Areal Reduction Factors

The design rainfall IFDs are only applicable to a single point. The design rainfall IFDs are therefore reduced using areal reduction factors (ARFs).

Because of the scarcity of Australian data, little work has been done on the profile of storms within Australia. The ARFs recommended in IEAust (1987) are based on work done in the United States.

The derived ARFs are strongly dependent upon the rainfall type. The ARFs used in IEAust (1987) were based upon frontal rain, rain from decaying storms of tropical origins and local thunderstorms. The ARFs apply for ARIs from 2 to 100 years and for catchment areas up to 1,000 km².

Inadequacies have been noted in the ARFs included in IEAust (1987). Meynink and Brady (1993) derived ARFs for large Australian catchments and for equatorial regions where storms tend to be of limited areal extent. Porter and Ladson (1993) derived ARFs for use in Northern Victoria.

5.4. Design Temporal Patterns

IEAust (1987) includes methods for determining design temporal patterns to be used in conjunction with the design IFD curves. The intention of the temporal patterns is to convert a design rainfall of an ARI Y years to a design flood of the same ARI.

Patterns are presented for 20 durations from 10 minutes to 72 hours. The patterns were developed from 83 pluviometers with an average period of record of 30 years (Rowbottom, et al., 1986a). The average variability method (Pilgrim et al., 1969) was used to smooth the recorded patterns.

The temporal patterns apply to intense bursts of rainfall within longer duration storms and not to complete storms. For this reason the patterns should not be applied to historical storms. The patterns should not be considered to be typical patterns because for most locations great variability occurs in the observed temporal patterns.

A feature of the patterns is that they have multiple peaks. This is because a shorter time step was used than in the 1977 addition of Australian Rainfall and Runoff. Another reason for the multiple peaks in the temporal pattern is that all of the sample patterns were used, whereas in the past the 'odd' patterns were rejected (Rowbottom et al., 1986a).

The procedures outlined in IEAust (1987) were used to derive temporal patterns for various durations and ARIs.

5.5. Concluding Remarks

The guidelines in IEAust (1987) were used to derive design IFDs and temporal patterns. The design rainfalls were entered into the RORB model developed in Chapter 6. The resulting flows were compared to the flood frequency curve. The design rainfalls were then modified so that the flows matched the flood frequency curve.

Chapter 6

Modelling of Streamflow

6.1. Streamflow Hydrographs

6.1.1. Physical Processes of Streamflow

Three different physical processes have been identified as contributing to streamflow. These are IEAust (1987):

1. surface runoff, often referred to as overland flow;
2. interflow, sometimes referred to as subsurface flow; and
3. baseflow which is the component of the hydrograph resulting from groundwater flow.

In the analysis of the components of a hydrograph, differentiation between flow components is not made according to physical sources of runoff however, but rather on the basis of travel times.

The classification of component flows according to travel times is a very useful analytical tool with the procedure having an empirical basis (Klaassen and Pilgrim, 1975). For this analysis, only the streamflow components of surface runoff and baseflow were considered.

Surface runoff occurs when the rainfall intensity exceeds the infiltration capacity of the soil. This is referred to as Horton-type runoff (IEAust, 1987). The infiltration rate is highly dependent on the soil and vegetation type.

A second runoff producing mechanism, that of saturated source areas, is now recognised (IEAust, 1987). Rainfall intensities less than the infiltration rate may produce surface runoff if the rainfall falls on a saturated section of the catchment: a source area. Most of the runoff therefore comes from the valley floor which is saturated by subsurface flow, and from soils which have low storage capacities. A large proportion of the surface runoff may be produced by a small section of the catchment.

6.1.2. Modelling of Baseflow

The characteristics of the baseflow will depend on the size of the catchment and other catchment characteristics. Nathan and McMahon (1989) identified some general characteristics of baseflow hydrographs.

1. The baseflow recession continues after the rise of the total hydrograph and peaks after the total hydrograph peak. This is a reflection of the fact that the groundwater travel time is greater than the travel time of the surface runoff.
2. The baseflow recession most likely follows an exponential decay function.
3. The baseflow hydrograph will rejoin the total hydrograph at the point at which surface runoff ceases.

Although the above characteristics of baseflow hydrographs are generally accepted, the exact baseflow hydrograph is difficult to define.

Runoff-routing models rout the rainfall excess through a conceptualised model of the catchment to produce an outflow hydrograph. The model therefore only simulates the physical processes of surface runoff and interflow.

The separated flow components are referred to as baseflow and surface runoff, and these components are assumed to correspond approximately to the physical processes. Some of the interflow component however is included with the baseflow and removed from the total hydrograph when calibrating the model.

The process of the separation of hydrographs is quite subjective. There is still some uncertainty regarding the inclusion of interflow in the modelling process. IEAust (1987) recommends that only the true surface runoff should be modelled and that the separated baseflow should include both interflow and groundwater flow.

Two different graphical methods of baseflow separation were tested by Bates and Davies (1988) and the sensitivity of the model to the choice of method was investigated. It was discovered that the sensitivity of the model to the chosen baseflow separation model was dependent on the model linearity. Linear models were found to be less susceptible to baseflow separation errors than non-linear models. The effect on the modelling of the PMF and other extreme floods was small.

The non-linear runoff-routing model RORB was chosen for this study, and it is described in Section 6.2. Baseflow is allowed for in the model by two different methods (Laurenson and Mein, 1990):

1. Baseflow Separation: The baseflow hydrograph can be calculated by an appropriate technique, and then the baseflow hydrograph is subtracted from the total hydrograph, which includes all flow components. Only the surface runoff hydrograph is used to calibrate the model.

2. Modelling of Baseflow: RORB allows the baseflow to be included in the routing process. The baseflow hydrograph calculated at the gauging station is then entered as a distributed channel inflow over all reaches of the model. The model is calibrated using the entire recorded hydrograph.

Baseflow separation was undertaken as the modelling of the baseflow was not considered to be suitable for this analysis (Dyer et al., 1992b).

6.1.2.1. End Point of Surface Runoff

Most methods used to remove baseflow from the total streamflow hydrograph rely on the estimation of the end point of surface runoff. This is the point at which the surface runoff ceases and the baseflow component contributes all of the streamflow. This point can be determined using a number of different methods:

1. empirical formulae;
2. recession curves;
3. a master recession curve; and
4. estimation 'by eye'.

A combination of methods 2 and 4 was used to determine the end point at which surface runoff ceases.

6.1.3. Baseflow Separation Methods

There are many different methods which can be used to separate baseflow from a total hydrograph. Commonly used techniques that were considered included a graphical approach and a recursive digital filter.

6.1.3.1. Graphical method

IEAust (1987) recommends a simple graphical separation of baseflow. A straight line connects the estimated start and end points of the surface runoff hydrograph. It is argued that because the baseflow usually only represents a small proportion of the total flow, this approximation does not result in a significant error.

This method can be refined by drawing tangents at the start and end points of the surface runoff. The tangent from the start of surface runoff is continued until it reaches a point below the peak of the total hydrograph and then a smooth curve is used to connect this to the tangent from the end of surface runoff point.

6.1.3.2. Recursive Digital Filter

Digital filters are used in signal analysis and processing. They can be used to separate an unwanted signal from a set of data. The digital filter has two main characteristics. The first of these is the gain, which is the factor by which the original signal is multiplied when passing through the filter. The second characteristic of the digital filter is that it can produce a shift in phase.

Lyne (1979) examined the use of digital filters to model streamflow responses. The streamflow hydrograph was separated into quick and slow response components using a recursive digital filter. A recursive digital filter is one which operates by weighting both the previous input and the output data. It acts as a transfer function. Although the separation was not made according to any differences in physical processes, it was assumed that the quickflow would represent the major portion of the surface runoff.

A filter of the form of Equation 6.1 was chosen.

$$f_k = a.f_{k-1} + \frac{(1+a)}{2}(y_k - y_{k-1}) \tag{6.1}$$

where: f_k is the filtered quick response at the kth sampling instant;

y_k is the total streamflow; and

a is the filter parameter.

Two restrictions were placed on the digital filter: the separated slow flow was never negative or greater than the original streamflow.

After the forward pass filtering, a reverse pass filter was applied to the slow flow starting from the end of the data. This was done to nullify any phase distortions. It is also suggested that the data may be passed through the filter again if the separated slow flow appeared to be responding too quickly. The choice of filter parameter (a) was not found to be a critical factor.

Lyne and Hollick (1979) suggested that a good separation of components could be achieved by using a filter parameter between 0.75 and 0.9. It was also recommended that a reverse pass filter be applied to nullify any phase distortions.

Application of this technique has been discussed by O'Loughlin et al. (1982) and Nathan and McMahon (1989 and 1990). The recommended value of the filter parameter was 0.925.

6.1.3.3. Treatment of Baseflow in other Studies

Several hydrological studies which deal with the Onkaparinga Catchment and other nearby catchments were studied in an attempt to determine the most appropriate method of estimating the baseflow.

Schalk (1986) analysed 7 different catchments, 5 of which were in the Mt Lofty Ranges. The baseflow was separated using two different methods. The first of these was an average recession curve. Where sufficient recessions had been recorded, a master recession curve was constructed. For catchments which did not have sufficient data to construct master recession curves, a depletion ratio was used. This is an empirical relationship which can be used to differentiate between baseflow and surface runoff. The baseflow was then included in the routing procedure as described in Section 6.1.2.3 as a uniformly distributed inflow. As already stated, Dyer et al. (1992b) strongly advised against this method as it incorrectly delays the baseflow.

BC Tonkin and Associates (1986) prepared unit hydrographs for ten Adelaide Hills' catchments. Master recession curves were constructed for each catchment. A smooth curve was drawn between the commencement and cessation of surface runoff. The baseflow was then subtracted from the total hydrograph.

6.1.4. Adopted Method of Baseflow Separation

The recursive digital filter was chosen as the method for the separation of baseflow. The baseflow was separated using the program HYBASE which is included in the data archiving system HYDSYS. This uses a recursive digital filter of the same form as Equation 6.1. The method is considered to be objective and repeatable.

There are three parameters which must be chosen. Each of the parameters were varied to observe the sensitivity of the calculated baseflow. The parameters are:

1. The time interval in minutes. This is the increment at which the calculations are performed. Increasing the time interval resulted in an attenuation of the calculated baseflow peak. The default value in HYBASE is 60 minutes.

2. The filter factor. The filter factor is designated as the filter parameter (a) in Equation 6.1. Increasing the filter factor resulted in an attenuation of the peak, but no significant shift in phase. The default value is 0.925.

3. The number of passes. The number of passes defines the number of successive forward and then backward passes that the filter makes over the data. Increasing the number of passes resulted in an attenuation of the calculated baseflow and also resulted in a slightly increased lag. The default number of passes is three.

The user also defines the start and end times of the calculations. The choice of the start time was not found to be significant, and therefore it was set equal to the start time in the RORB data file.

The end time represents the end of the surface runoff. This point was first estimated by plotting a semi-log plot of the hydrograph recession curve. The baseflow was then separated using HYBASE. The separated baseflow was compared to the total hydrograph and the end point adjusted until the calculated baseflow hydrograph and the total hydrograph converge tangentially.

The default values of the HYBASE parameters were used: a filter factor of 0.925, 3 passes and a 60 minute time interval. This combination resulted from consideration of Nathan and McMahon (1989), O'Loughlin et al. (1982), Lyne and Hollick (1979) and Lyne (1979).

A typical hydrograph separation is shown in Figure 6.1.

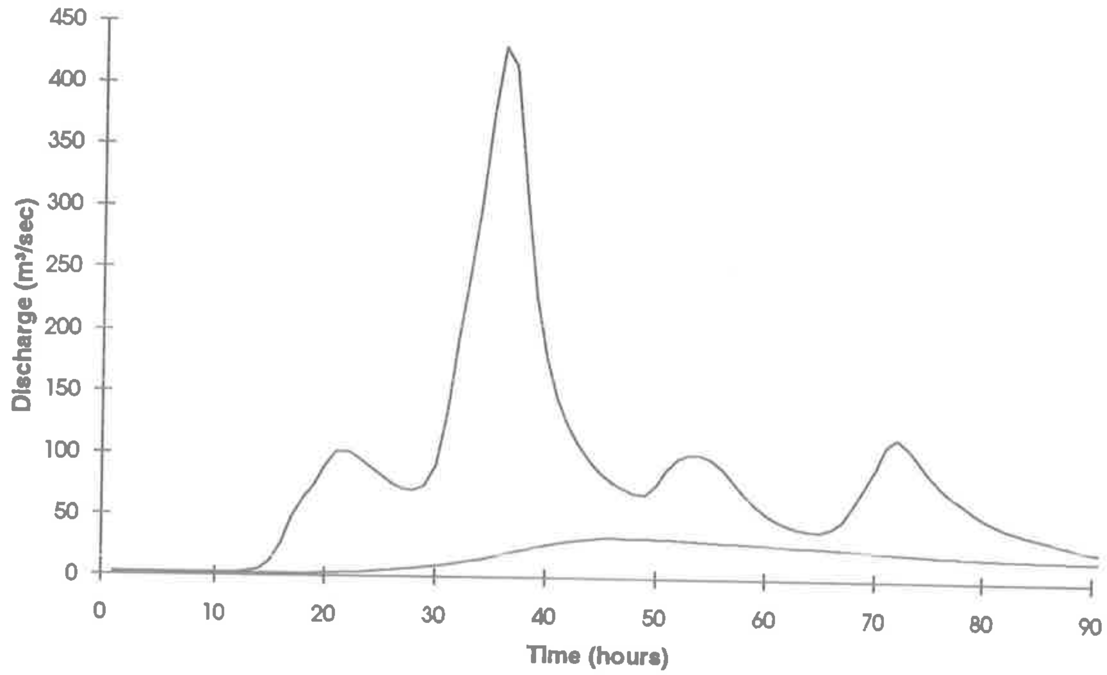


Figure 6.1 Example of a Typical Hydrograph Separation (30 August 1992)

The peak values of the baseflow are shown in Table 6.1 for the different flood events examined for calibration of the runoff routing model. The average peak baseflow is 15 m^3/sec, which represents approximately 10 percent of the total streamflow peak.

Table 6.1 Peak Baseflows Separated using HYBASE

Date	Peak Flow (m^3/sec)	Peak Baseflow (m^3/sec)	Proportion of Peak Flow (%)
26/06/81	243	24	10
31/07/81	154	23	15
24/07/81	118	11	9
03/08/81	95	9	9
08/08/81	132	13	10
14/08/81	114	14	12
25/08/83	86	7	8
08/09/83	119	10	8
21/08/84	97	13	13
24/06/87	206	17	8
15/07/87	148	16	11
24/05/88	156	5	3
15/08/90	80	10	13
30/08/92	432	34	8
16/09/92	96	20	21
08/10/92	193	13	7

6.2. RORB Runoff Routing Program

A unit hydrograph or a non-linear runoff routing model is used to convert the design rainfalls to design hydrographs at the particular point of interest. The advantage of non-linear runoff routing programs is that the non-linear response of the catchment can be modelled.

The two most commonly used non-linear runoff routing packages in Australia for rural catchments are RORB and RAFTS. The RORB model was adopted in this study because:

1. Its widespread use in South Australia allows the chosen model parameters and modelling procedures to be compared and verified by other similar studies.
2. RORB allows different model non-linearities to be used, whereas RAFTS has a fixed exponent of 0.715.
3. The RORB model can be calibrated using historical data. The RAFTS model is not calibrated directly, but rather model parameters are obtained from physical properties of the catchment.

6.2.1. Description of Model

The RORB model is a non-linear runoff routing model and is suitable for rural catchments. A brief description of the model is included below, while a more complete description of the model is included in Laurenson and Mein (1990).

The catchment is divided into sub-areas of approximately equal area, based on drainage divides. Laurenson and Mein (1990) recommended that between 10 and 20 sub-areas are used. Weeks (1980) showed that the calculated flood peak was delayed as the number of sub-areas was increased.

Nodes are placed at the centroids of each sub-area, confluences of all major streams and at any other points of interest on the river network.

The rainfall specific to each sub-area is assumed to fall at the centroid of the sub-area. Rainfall losses are then subtracted from the hyetograph to produce the rainfall excess hyetograph, and consequent surface runoff. The storage effect of each river reach is represented by a model storage. The runoff is routed through the series of model storages and the hydrographs added at stream confluences, producing an outflow hydrograph.

The reaches are assumed to have a storage-discharge relationship of the form of Equation 6.2.

$$S = 3600\ k\ Q^m \tag{6.2}$$

where:

S is the storage (m³);
Q is the discharge (m³/sec);
m is a dimensionless exponent (a measure of the catchment's non-linearity); and
k is a dimensionless empirical coefficient.

The empirical component k is expressed as the product of k_c and k_r:

- k_c is a measure of the storage of the catchment and applies to the entire catchment and stream network; and
- k_r is called the relative delay time and is calculated by the program for each individual reach storage.

If the model is to be used to predict flows which greatly exceed those used in calibration, the choice of m is very important. The choice of m is less important if the floods used to calibrate the model are of the same magnitude as the design floods. For this reason a fixed value of m is not recommended (Weeks, 1980).

There are 4 model parameters which must be determined by the user: the initial loss (IL), the continuing loss (CL), the model linearity m and the coefficient k_c.

6.2.2. Calibration of the Model

Because large variations are observed in model parameters, it is important that the model is calibrated using historical events in cases where sufficient data exists.

For ungauged catchments the only method of determining model parameters is a generalised method. This can involve transferring data from neighbouring catchments or using regional relationships (Kemp, 1993; Dyer et al., 1992a).

No optimisation method currently exists in the program to determine the choice of parameters, although Dyer et al. (1993) has made some advances in automating the fitting procedure. This method is still in the research stage and requires large computational effort.

Calibration of the model involves varying the four model parameters until a good fit is obtained between the calculated and observed hydrographs.

The criterion used to determine the quality of fit should depend upon the purpose to which the results are to be put. Depending on the situation, the hydrograph peak, volume, lag and overall shape may be important. In most cases the hydrograph peak is considered to be the most important characteristic of the hydrograph. McMahon and Muller (1983) recommended that the model is calibrated upon the consideration of peak flow.

The model is usually calibrated using the largest recorded floods on record. The recording of large floods often involves the extrapolation of rating curves. For this reason the over reliance on the exact matching of peak flows may not prove to be the best method of model calibration. It is recommended that the peak flows are matched, but with due consideration given to other characteristics of the hydrograph.

The principal model parameter for fitting is k_c. Increasing k_c decreases the hydrograph peak and increases the lag. Decreasing k_c has the opposite effect. Different k_c values are obtained for different events on the same catchment. This is due to (Laurenson and Mein, 1990):

1. errors in the rainfall and streamflow data;
2. baseflow separation errors;
3. rainfall variability; and
4. the model not fully representing the hydrological processes.

Varying the initial loss is also a means of obtaining a fit. The initial loss affects the start of the hydrograph rise and may also affect the hydrograph shape and peak.

If the value of m is decreased, and the consequent change is made to k_c, the start of the rise and the tail of the recession are slightly delayed whereas the peak is slightly advanced.

The calibration of individual events does not usually yield a single set of model parameters but rather each event which is analysed results in a different set of parameters which pertain only to that individual event.

Different parameters can also be calculated from the same event. The two parameters, m and k_c, have compensatory effects and therefore a fit can be obtained between the recorded and the calculated hydrograph for a number of different values of m and corresponding values of k_c.

In order to produce design flows, a set of model parameters need to be selected.

6.2.3. Parameter Interaction Curve

A commonly used method to resolve the decision of the optimal choice of the parameters k_c and m makes use of a parameter interaction curve. This is also referred to as a parameter indifference curve. The method is discussed by Weeks (1980), Stewart (1983), McMahon and Muller (1983), McMahon and Muller (1986), IEAust (1987) and Laurenson and Mein (1990).

The model is calibrated using each event for a range of m. The relationship between k_c and m is then plotted for each event. Each line plotted on the parameter interaction curve represents the unique relationship between m and k_c for a given event. The model, in terms of peak flow, is indifferent to the choice of model parameters along the curve. The model is not however indifferent in terms of calculated hydrograph shape. The quality of fit in terms of shape may vary significantly for different model non-linearities.

It is assumed that an intersection point on the parameter interaction curve represents the optimum choice of model parameters.

McMahon and Muller (1983) showed that for an idealised catchment with error free data, the lines on the parameter interaction curve should intersect at a single point. For real catchments the lines very rarely intersect and this is attributed to the errors in the data. A subjective choice is then made as to the point of intersection on the parameter interaction curve (Stewart, 1983).

The discovery of an intersection point does not guarantee that the non-linearity of the catchment has been identified (McMahon and Muller, 1983; Daniell, 1987). Sensitivity analysis is recommended to determine the optimum model parameters.

It is important that the data used in the calibration of the model is checked. The use of erroneous data can produce non-representative parameter interaction curves (McMahon and Muller, 1983).

6.2.4. Sensitivity Fitting

Sensitivity fitting is a simple extension of the parameter interaction curve method of determining the optimal choice of model parameters and is described by Hill et al. (1993). The method tests the sensitivity of the model to a variation in k_c over a range of m. A range of m should exist for which a change in k_c results in a minimal change in the calculated flood peak.

McMahon and Muller (1983) referred to the possibility of such a method being developed:

> *It is conceivable that, with indifference curves for a number of calibration events, some sort of statistical approach should be taken to choosing a single (k, m) value of a closed set of (k, m) values for use in the flood prediction stage of the analysis.*

For a number of different values of m, the average value of k_c is calculated from the parameter interaction curve. The RORB models are run for the events used to calibrate the model using values of m and the corresponding average values of k_c with the average values of the initial and continuing losses. The error in the calculated peak flow is then determined for each event as a percentage. The absolute average error for each choice of m is then calculated.

The percentage error in the calculated peak flow is then plotted against the parameter m. A minimum in the average error curve represents an optimal choice of m and k_c. If such a minimum exists, the values of m and k_c produce the smallest average error in calculated peak flow.

The use of sensitivity fitting resolves the problem of the choice of model parameters. It also examines the uncertainty of the flood estimates from the model.

6.3. RORB Model of the Onkaparinga Catchment

6.3.1. Catchment to Mt Bold Reservoir

A RORB model was developed for the catchment to Mt Bold Reservoir following the procedures described in Section 6.2 and Laurenson and Mein (1990). The adopted RORB model is shown in Figure 6.2.

The catchment was divided into 12 sub-areas based upon drainage divides. The sub-areas were based on those used in Kotwicki (1984). The area of each sub-area was determined using a planimeter. The areas of the sub-areas are shown in Table 6.2.

Table 6.2 Areas of RORB Sub-Areas

RORB Sub-Area	Area (km²)
A	51.5
B	27.6
C	19.4
D	30.6
E	29.1
F	46.5
G	22.0
H	34.8
I	23.0
J	37.7
K	22.9
L	39.8

The location of the centroid of each sub-area was found by determining the centre of gravity of each sub-area shape using a plumb line. Nodes were placed at the point at which the Murray Bridge Onkaparinga Pipeline discharges into the Onkaparinga River and also at Houlgraves Weir.

The reach lengths were difficult to measure and were therefore determined by digitising the major rivers in the catchment. This was done using ARC-INFO.

A RORB data file was generated using the procedures in Laurenson and Mein (1990).

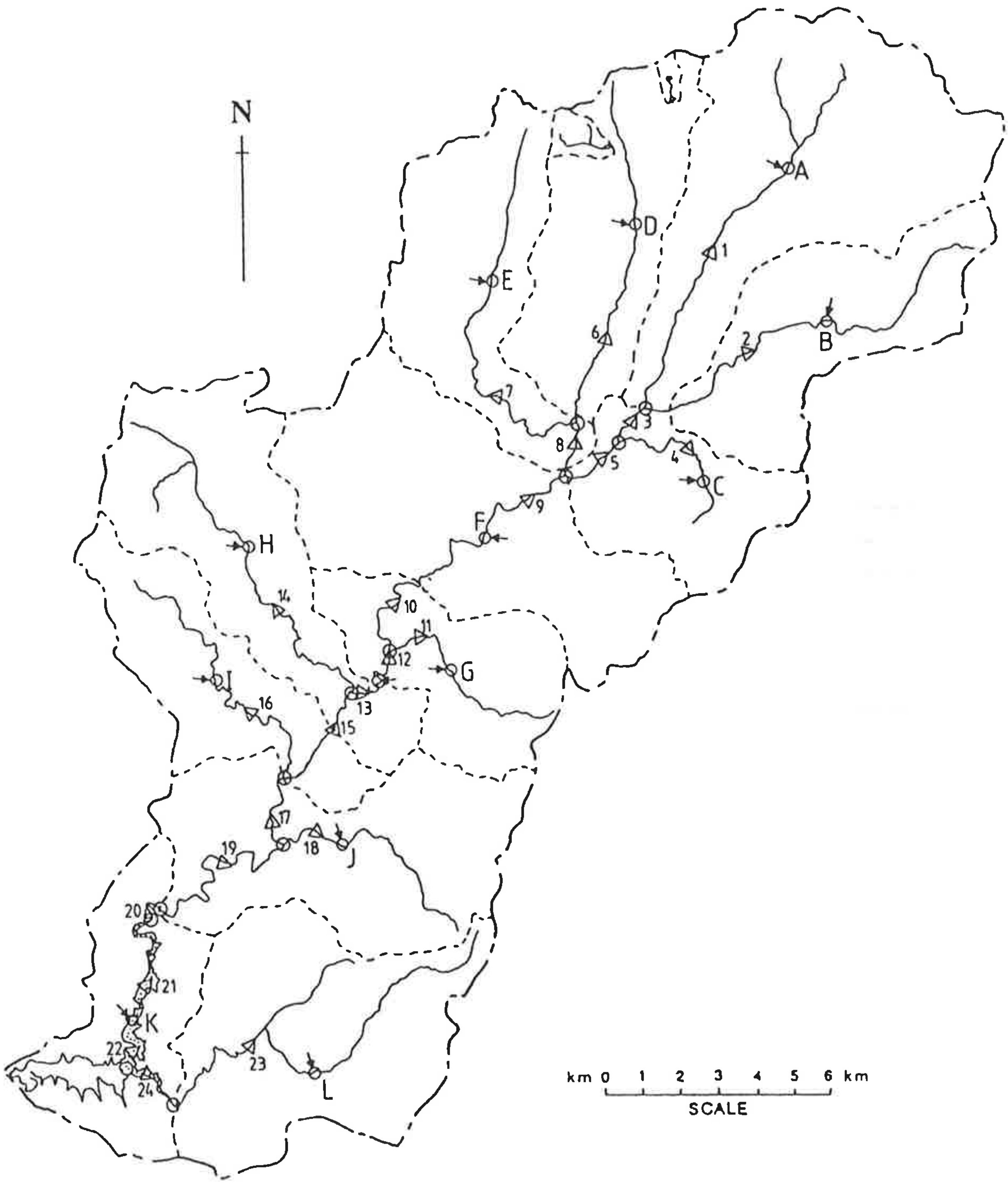


Figure 6.2 RORB Model for the Onkaparinga River to Mt Bold Reservoir

6.3.2. Mt Bold Reservoir

Storage reservoirs, lakes and retarding basins are treated by RORB as special storages and are modelled differently from the normal channel reach storages. In order to rout hydrographs through a special storage, the model requires the relationship between discharge and the storage volume. This relationship is usually inferred from the relationships between storage and elevation, and the relationship between elevation and discharge.

6.3.2.1. Rating Curve for Mt Bold Reservoir

For the case of a reservoir with a gated spillway, the rating depends on gate operation and therefore the operating policy.

As described in Section 4.2.1, the current operation of the gates differs from that of the official policy. The official operating policy was adopted for routing the design floods through the reservoir. This was because pre-releasing has not been embraced as official policy. At present the practice of pre-releasing is very subjective and therefore proves almost impossible to model in a systematic manner.

The rating of a reservoir can be expressed in RORB in several different ways. Because the rating of Mt Bold Reservoir is quite complex, it was decided to input the rating as a table of storage and discharge values.

This involved assigning gate openings for different reservoir levels. It was assumed that the gates were operated initially according to the official E&WS policy. As the level of the reservoir rises it was assumed that the gates are further opened until all gates are opened to their maximum setting at a gauge height of 41.9 metres; representing a reservoir level 500 mm above full supply level. The assumed gate operations are shown in Appendix G, and the consequent rating is shown in Figure 6.3. It was assumed that the maximum gate openings were reset to 3.8 metres as outlined in Section 4.2.2.

An example of an inflow and the calculated outflow hydrograph using the adopted rating curve, and the model parameters described in Section 6.4, is shown in Figure 6.4.

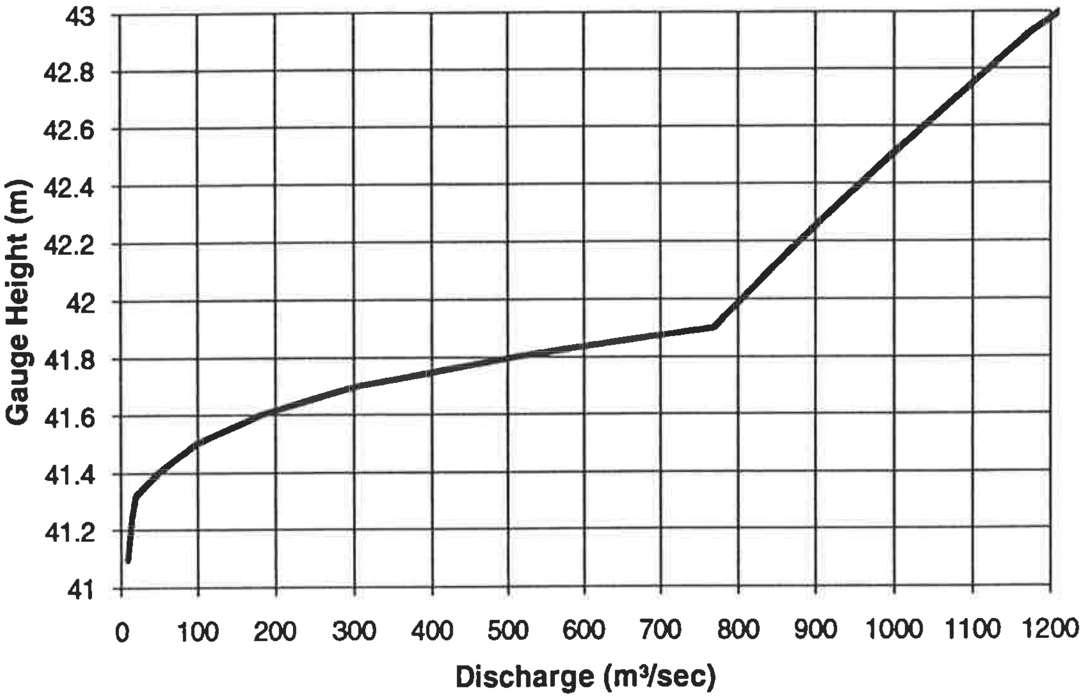


Figure 6.3 Rating Curve for Mt Bold Reservoir (Official Operating Policy of the E&WS)

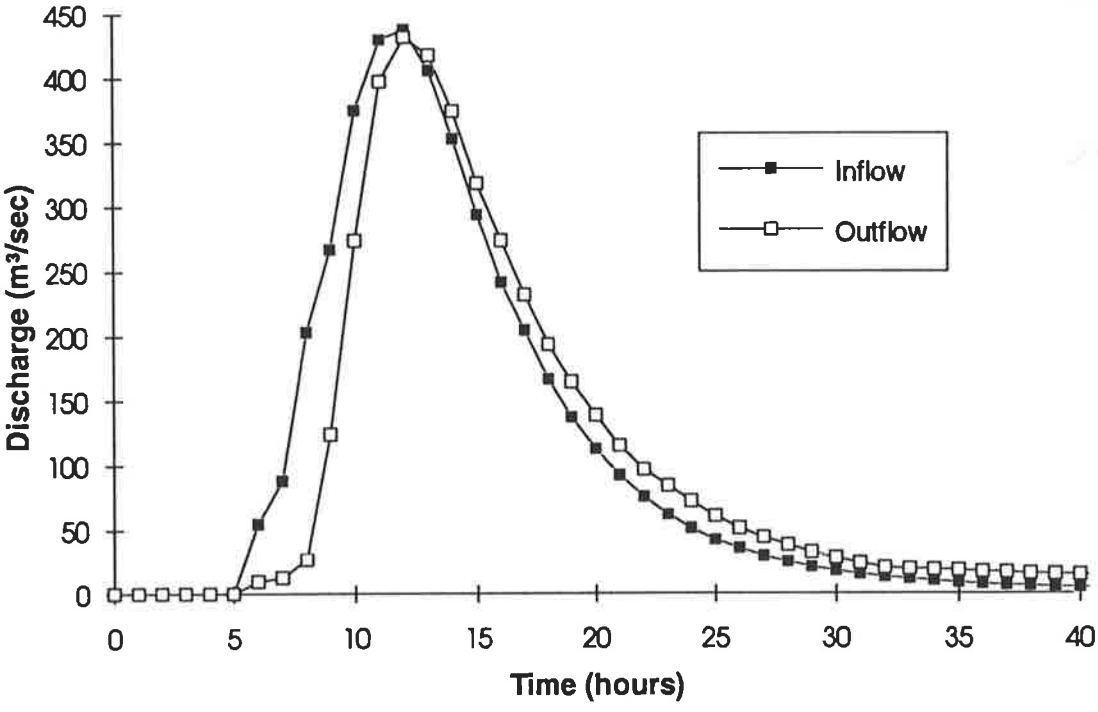


Figure 6.4 Example of Inflow and Outflow Hydrographs Calculated using the Official Operating Policy of the E&WS

6.3.2.2. Storage Elevation Relationship

The relationship between storage and elevation can be entered into the model in the form of Equation 6.3.

$$S = a(H - H_0)^b \tag{6.3}$$

where S is the storage volume (m^3);
H is the water surface elevation (m);
H_o is the elevation corresponding to zero storage (m); and
a and b are constants.

The constants a and b can be evaluated by plotting storage against (H-H_o) on log-log paper. A line of best fit is then drawn and a and b determined as the intercept and slope of the line respectively (Laurenson and Mein, 1990).

A linear regression was performed on the logs of the storage and water surface elevation. An expression of the form of Equation 6.3 could not be found to represent the measured storage elevation relationship with sufficient accuracy. The relationship was therefore entered into the model as a table of elevations and corresponding storages.

6.3.2.3. Initial Drawdown

A special storage can be assumed to be drawn down below its cease-to-flow point at the commencement of the event. The drawdown is the volume of water required to bring the water level up to the cease-to-flow point. When the hydrograph is routed through the reservoir by the model, this volume is subtracted off the front of the hydrograph.

It was initially assumed that the reservoir was at full supply level prior to the design inflow. Chapter 8 includes a thorough analysis of different reservoir storage distributions.

6.3.3. Onkaparinga Catchment Below Mt Bold Reservoir

A RORB model was developed for the Onkaparinga River below Mt Bold Reservoir to Old Noarlunga. The model is shown in Figure 6.5.

Catchment boundaries had already been digitised by the Water Resources Section of the E&WS using 1:10,000 maps. Digitising of the streams had been commenced and was completed for this study using 1:50,000 maps. The information was loaded into ARC-INFO where the necessary catchment information was calculated and then down loaded into an 'x, y' format.

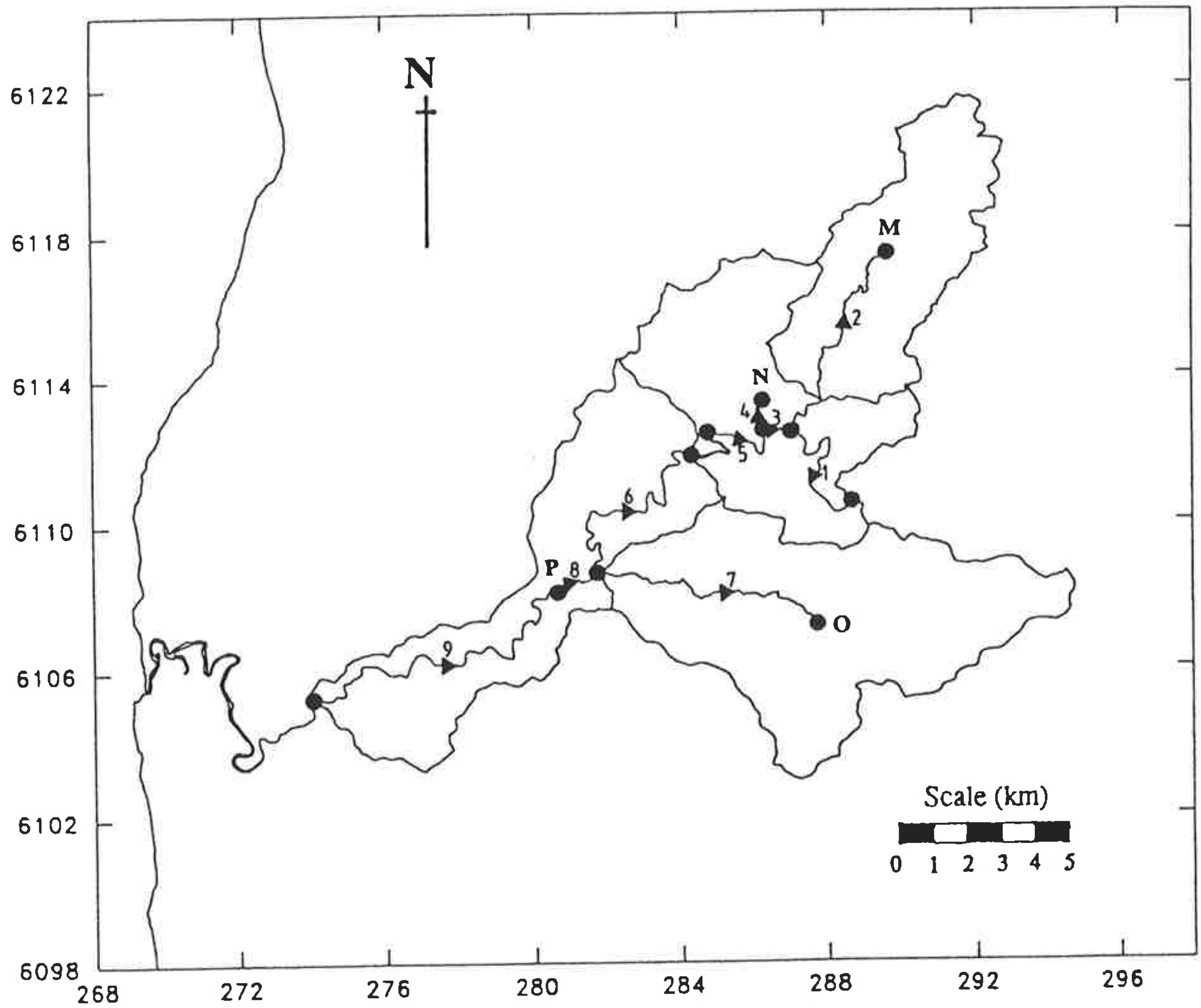


Figure 6.5 RORB Model for the Onkaparinga River from Mt Bold Reservoir to Old Noarlunga

The areas of the sub-areas were calculated using ARC-INFO. The calculated areas were checked using a planimeter. The descriptions and calculated areas of the sub-areas are shown in Table 6.3.

Table 6.3 RORB Sub-areas for the Lower Onkaparinga River

Sub-Area	Description	Area (km²)
M	Scott Creek Catchment	26.5
N	Onkaparinga Clarendon and d/s Mt Bold	31.2
O	Baker Gully and Dashwood Gully	48.0
P	Onkaparinga Noarlunga	36.0

The model has 9 model storages and the special storage of Clarendon Weir. The reach lengths were calculated using ARC-INFO.

6.3.3.1. Clarendon Weir

A spillway rating for the weir was obtained from the E&WS. A mathematical equation was sought which represented the rating. The weir equation for an ungated spillway is expressed in Equation 6.4.

$$Q_s = K_w L_s (H - H_s)^{3/2} \qquad (6.4)$$

where: Q_s is the spillway discharge (m³/sec);
K_w is the weir coefficient for the spillway;
L_s is the effective length of the spillway (m);
H is the water surface elevation (m); and
H_s is the spillway crest elevation (m).

The effective length of the spillway was not determined, but the total length of 61 metres was used. The weir coefficient was determined by varying it until Equation 6.4 matched the known rating. The calculated value of weir coefficient was 1.54. Table 6.4 shows typical values of weir coefficients for different weir geometries (Laurenson and Mein, 1990). The calculated value lies between a broad and sharp-crested weir.

Table 6.4 Typical Weir Coefficients

Weir Type	Kw
Ogee	2.15
Broad-crested with sloping approach	2.0
Broad-crested with vertical upstream face	1.45
Sharp-crested with vertical upstream face	1.74

The rating for Clarendon Weir was expressed as Equation 5.5. This expression was checked over a range of water surface elevations from 10 to 12.5 metres. The maximum difference between the reported and calculated rating was less than 1 percent.

$$Q_s = 93.94\ (H - 10)^{3/2} \qquad (6.5)$$

Information for the storage elevation relationship was only available up to the spillway crest (10 metres). A relationship for the storage elevation relationship was sought of the form of Equation 6.3.

A value of 6,800 was calculated for a and a value of 1.856 for b. The elevation corresponding to zero storage is 2 metres. The calculated storage-elevation relationship for the Clarendon Weir is shown in Equation 6.6. The expression is only accurate for elevations greater than 6 metres.

$$S = 6{,}800\ (H - 2)^{1.856} \qquad (6.6)$$

A copy of the RORB data file is included in Appendix G.

6.4. Calibration of the Mt Bold RORB Model

6.4.1. Selection of Events

The model was calibrated using the sum of the recorded flow of the Onkaparinga River at Houlgraves Weir (AW503504) and Echunga Creek (AW503506). The gauging station at Houlgraves Weir commenced operation on 17 April 1973, while the gauging station at Echunga Creek commenced on 22 March 1973.

Events which have a small peak flow are difficult to model using runoff routing models. This is because the estimation of the rainfall and losses dominates the results and the routing effect of the model is obscured. Only events that had a peak instantaneous flow of greater than 75 m^3/sec at Houlgraves Weir were therefore considered.

The events chosen to calibrate the model must be independent. An analysis of the recession curves suggested that peak flows separated by more than 3 to 4 days would in most instances be independent. For successive events which had peaks which were only separated by a few days the hydrographs were examined in order to confirm independence. If the events were not considered to be independent, only the event which had the greatest peak flow was modelled.

Another constraint upon the selection of events was the requirement for adequate rainfall records. The spatial variation of the rainfall was recorded accurately by the many daily read raingauges in the catchment and surrounding region. The variation of the rainfall with time was less easy to ascertain, as the first E&WS pluviometers in the catchment were only installed in 1977 and it was not until the middle of the 1980s that this network was increased substantially (Section 3.2). The BoM did have a pluviometer at Stirling before 1977 but this alone is insufficient for calibration of the model.

Further investigations discovered that the record for the Gallasch Creek Pluviometer (AW503521) has disappeared and this meant that 4 events prior to 1980 were excluded from consideration because of insufficient pluviometer record. The gauging station on Echunga Creek failed during September 1991 and therefore no records were available for the event of 15/9/91 and this event was therefore also excluded.

The event of 19/12/92, although having a peak flow of 186 m^3/sec, could not be considered in the study because the required pluviometer and streamflow information was not processed in time.

The 16 remaining events used in the calibration of the RORB model are shown in Table 6.5.

Table 6.5 Events used for the Calibration of the RORB Model

Time - Date	Peak Flow at Houlgraves Weir (m^3/sec)
18.00 26/06/81	243
21.00 03/07/81	154
11.00 24/07/81	118
21.00 03/08/81	95
14.00 08/08/81	132
11.00 14/08/81	114
04.00 25/08/83	86
08.00 08/09/83	119
09.00 21/08/84	97
07.00 24/06/87	206
17.00 15/07/87	148
05.00 24/05/88	156
06.00 15/08/90	80
12.00 30/08/92	432
05.00 16/09/92	96
21.00 08/10/92	193

6.4.2. Construction of Isohyets

In order to model historical storms using RORB, the spatial distribution of rainfall was required. Isohyets (lines of uniform rainfall) were constructed using the records from pluviometer and daily read raingauges. Because of the large numbers of isohyets which needed to be drawn, the contouring package Surfer was used to generate the isohyets. A description of Surfer is included in Appendix G.

The rainfall on each sub-area was estimated by eye using the isohyets.

6.4.3. Event Calibration

Model parameters were determined for each historical event following the procedures described in Section 6.2.2.

The event of 21/8/84 had a hydrograph with many small peaks over a period of many days, resulting from relatively small bursts of rainfall. The model could not accurately replicate the recorded hydrograph with the input rainfall information. This event was excluded from further analysis.

An analysis of the calculated hydrographs indicated that the peak flow was calculated several hours before the recorded hydrographs. A translation of 4 hours was inserted in the data files, which resulted in the calculated hydrograph peak being delayed 4 hours to match the observed hydrograph.

6.4.4. Parameter Interaction Curve

The two loss parameters varied for different events and initially a simple average was taken of both the initial and continuing losses (first burst losses of 21.2 mm IL and 1.4 mm/hr CL and second burst losses of 8.5 mm IL and 0.9 mm/hr CL).

A parameter interaction curve was constructed using the 15 available events, and this is shown in Figure 6.6. No single area of intersection was discovered. The general tendency of the curves to converge with increasing *m* is a property of the governing equations and does not indicate a optimal choice of k_c and *m*.

In an attempt to determine the optimal choice of k_c and *m*, sensitivity fitting was attempted as described in Section 6.2.3.2 and by Hill et al. (1993). This involves the consideration of the possible errors associated with a choice of *m* and k_c in order to determine the choice of *m* and k_c which produces the lowest average error in the calculated peak flow.

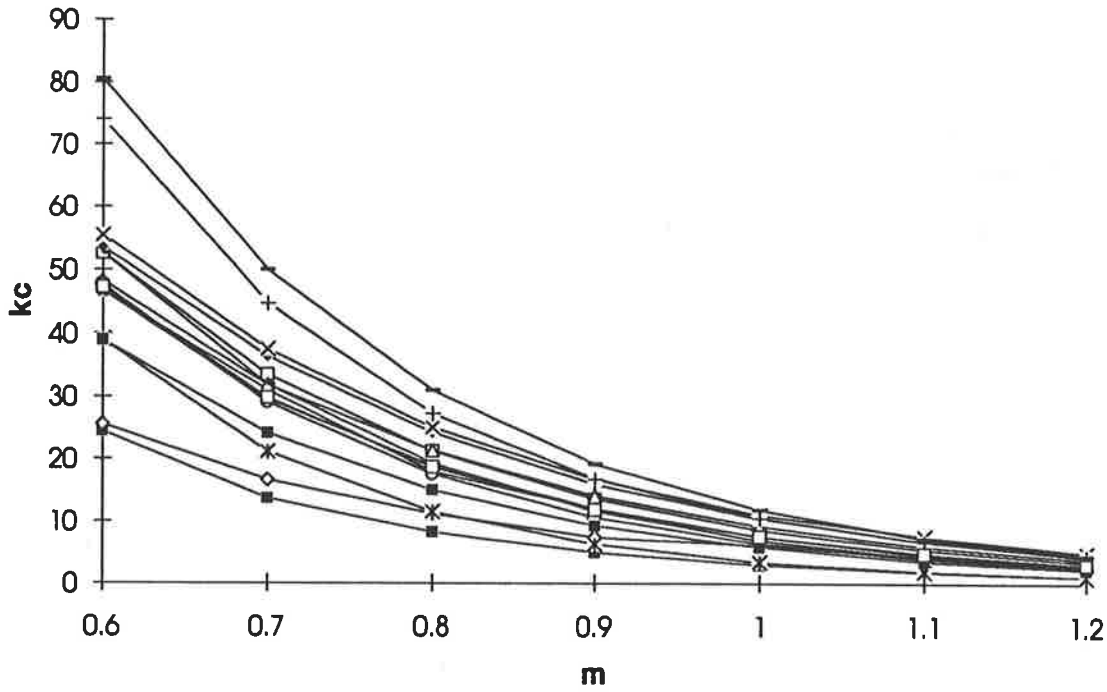


Figure 6.6 Parameter Interaction Curve for 15 Events

For different values of m between 0.6 and 1.2, the average value of k_c was calculated. The models were then re-run using these values with the average values of the loss parameters.

The event of 3/7/81 was calibrated using a very small continuing loss for the second burst and hence the adoption of the average continuing loss for the second burst resulted in the second peak being lower than the first. This event was excluded from further analysis.

An analysis of the quality of the fit used in the calibration of events indicated that two events produced poor fits between the calculated and observed hydrographs. The two events (26/6/81 and 14/8/81) were not included in any further analysis. The poor fit between the calculated and observed hydrographs was most likely a result of the poor record of the temporal pattern of the rainfall.

The sensitivity plot is shown in Figure 6.7. This shows that the average model parameters do not accurately model some of the events. The choice of parameters can result in an error of up to 170 percent in the calculated peak flow.

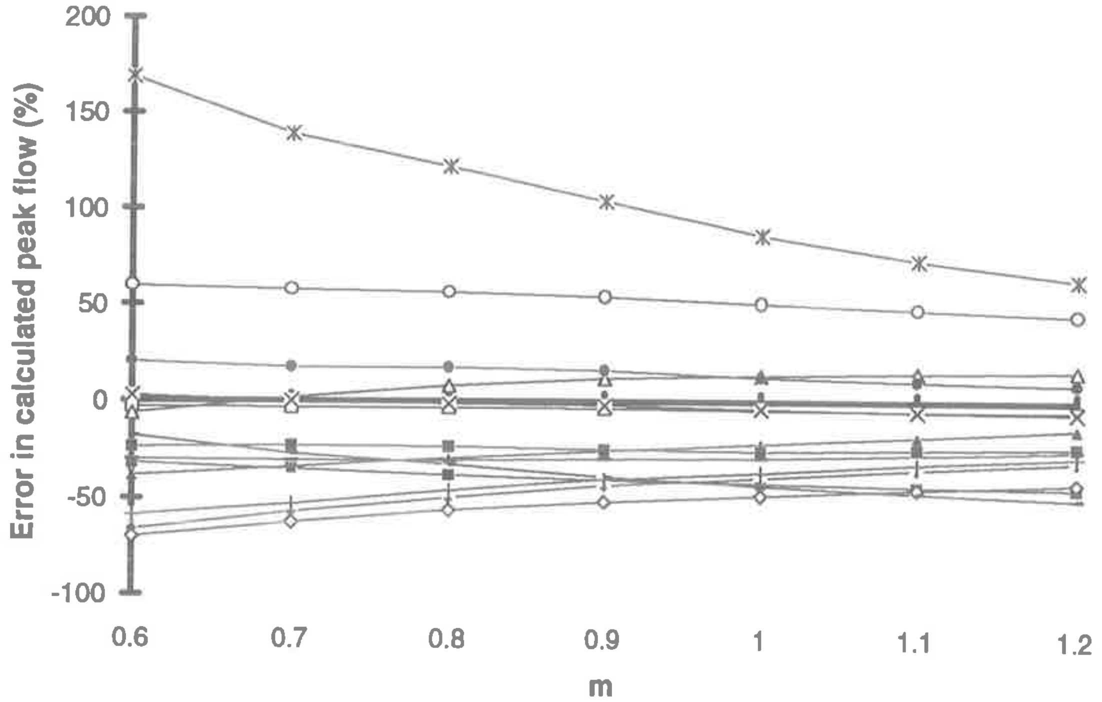


Figure 6.7 Sensitivity of the Model to the Choice of Model Parameters for 14 events

6.4.5. Division of Events According to Losses

For events which had been calibrated using a high initial and continuing loss, the adoption of the average model parameters resulted in the calculated peak flows being too high. The calibrated events were divided into two groups according to the losses.

The total loss was determined for each event as the sum of the initial loss and the product of the continuing loss and the duration of the rainfall burst. The total loss was then divided by the storm duration which produced the total loss per hour. The total loss per hour was not meant to represent a physical loss rate but rather provide a useful method of classification of events.

The modelled loss rates are shown in Table 6.6.

Table 6.6 Modelled Losses

	First Burst			Second Burst			
Time - Date	IL (mm)	CL (mm/hr)	Duration (hrs)	IL (mm)	CL (mm/hr)	Duration (hrs)	Total Loss / hr (mm/hr)
11.00 24/07/81	18	1.32	48	12	0.27	48	1.1
21.00 03/08/81	18	0.36	72	-	-	-	0.6
14.00 08/08/81	21	0.32	72	-	-	-	0.6
04.00 25/08/83	25	0.97	24	-	-	-	2.0
08.00 08/09/83	15	1.96	120	-	-	-	2.1
07.00 24/06/87	20	2.94	48	5	1.61	48	2.5
17.00 15/07/87	20	1.52	168	-	-	-	1.6
05.00 24/05/88	46	4.55	48	-	-	-	5.5
06.00 15/08/90	15	0.61	72	-	-	-	0.8
12.00 30/08/92	22	1.27	120	-	-	-	1.5
05.00 16/09/92	15	0.81	96	-	-	-	1.0
21.00 08/10/92	20	0.91	120	-	-	-	1.1

Two events (24/6/87 and 24/5/88) had significantly higher loss rates and were therefore considered separately. The flows recorded at Houlgraves Weir during 1987 and 1988 are shown in Appendix G. It is clear from these figures that the events of 24/6/87 and 24/5/88 were the first significant events of the winter, and this explains the greater losses.

Another important reason why these events had considerably higher losses than the other events is the effect of farm dams. As is described in Section 3.1.3, there are a significant number of farm dams in the Onkaparinga Catchment and these have an effect on the runoff in the catchment. Early in the winter most of the farm dams will be drawn down and this will result in much larger losses being associated with early winter events.

The design rainfalls were assumed to occur later in the winter, and therefore have lower losses. The decision to exclude those events with high losses may have biased the results towards winter events. The frequency of the flows derived from the design rainfalls were compared to those obtained from flood frequency analysis in Section 7.3.

In an attempt to reconcile the differences in the quality of fit between the calculated and recorded hydrographs, the hydrographs were analysed and a number assigned to represent the quality of fit. This fit parameter varied from 1 for an extremely poor fit and 10 for an excellent fit. Although the assignment of the fit parameter was both arbitrary and subjective, it provided a useful method of comparing the differences in the quality of fit for different events. This indicated that the average fit was best for an m of 0.7 to 0.8 and the fit deteriorated as m increased past 1.0.

For the 10 events with good fits and low loss rates, the models were re-run with average model parameters. For a given value of m, the average k_c was calculated as a weighted average dependent on the fit parameter. This was done in an attempt to place greater importance on those events which had the best observed fits.

The errors in the calculated peak flows were determined for each event and are shown in Figure 6.8. Each line in Figure 6.8 represents a different event. The average absolute error was calculated. This was weighted according to the magnitude of the recorded peak flows and was a minimum for an m of 0.7.

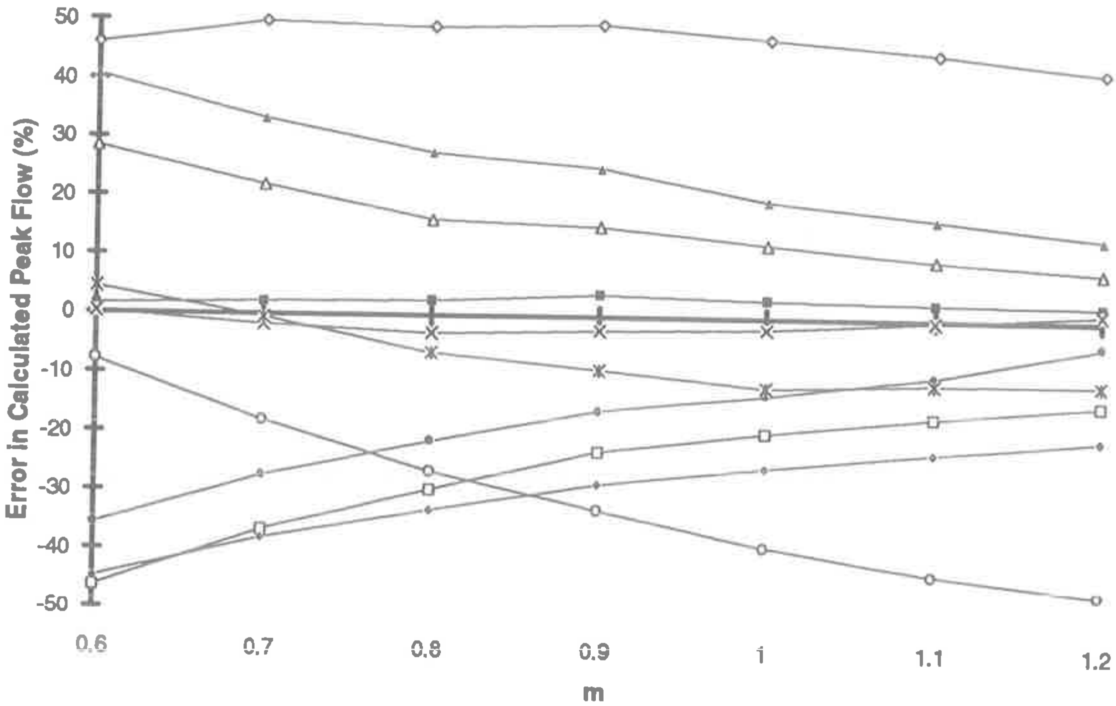


Figure 6.8 Sensitivity of the Model to the Choice of Model Parameters for Low Loss Events

6.4.6. Consideration of Largest Events

It was felt that the average parameters were adversely affected by the inclusion of the smaller events. Because the model was only used to calculate flows greater than the 5 year ARI event, the calibration was repeated with only the 3 largest events with low loss rates. The 3 events, are shown in Table 6.7. The ARI was assigned using the flood frequency curve calculated in Chapter 7.

Table 6.7 Largest Events with Low Loss Rates

Date	Peak Flow (m^3/sec)	Approximate ARI (years)
15/7/87	148	3.5
30/8/92	412	100
8/10/92	197	6

For the three largest events, the best fit and the smallest average absolute error was for a lower value of *m*. The average parameters for an *m* of 0.7 are shown in Table 6.8.

Table 6.8 Model Parameters Based on the 3 Largest Events with Low Loss Rates

Initial Loss	21 mm
Continuing Loss	1.2 mm/hr
m	0.7
k_c	27.4

The average model parameters shown in Table 6.8 were used to model the 3 largest events. The results are shown in Appendix G. The calculated hydrographs closely matched the recorded hydrographs. The peak was underestimated for 30/8/92 but overestimated for 8/10/92.

6.4.7. The Effect of the Number of Pluviometers

From the calibration of the RORB model for the Onkaparinga River, the importance of the number of pluviometers on the accuracy of the calculated hydrograph became apparent. The best fits were generally obtained for the more recent events for which there were more pluviometers operating in the catchment. It was more difficult to obtain satisfactory fits for events that occurred in the early 1980s. This was because for many of the events there was only 3 pluviometers operating in the vicinity of the Onkaparinga Catchment.

Because of the sparse network of pluviometers in South Australia, it is important to note the dramatic effect that the number of pluviometers had on the accuracy of the calculated hydrograph.

6.5. Final Choice of RORB Model Parameters

6.5.1. Model Parameters for the Catchment to Mt Bold Reservoir

Based on the preceding discussion, it was resolved to use the model parameters shown in Table 6.9. The need to determine different model parameters for calculating the PMF is discussed in more detail in Section 10.3. The choice of losses was based upon Table 6.6 and Table 6.8.

Table 6.9 RORB Parameters for the Catchment to Mt Bold Reservoir

	Design Flows	PMF
Initial Loss	20 mm	0 mm
Continuing Loss	1 mm/hr	0 mm/hr
m	0.7	0.8
k_c	27.4	16.3

6.5.2. Model Parameters for the Catchment to Old Noarlunga

The model parameters obtained for the catchment to Mt Bold Reservoir do not apply to the larger catchment. Although the RORB model could not be calibrated at Old Noarlunga, the inclusion of the reaches downstream of Mt Bold Reservoir affects the choice of model parameters. This becomes apparent when the storage equation is re-examined.

The storage equation used by the RORB model is discussed in Section 6.2. The constant k is the product of k_c and k_r. The coefficient k_c is determined by the user whereas the relative delay time k_r is calculated by the program using Equation 6.7.

$$k_{ri} = \frac{L_i}{d_{av}} \qquad (6.7)$$

where: k_{ri} is the relative delay time of storage i;

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

d_{av} is the average flow distance in the channel network (km).

The average flow distance for the RORB model to Mt Bold Reservoir is 27.3 km, whereas the average flow distance for the RORB model to Old Noarlunga is 49.8 km.

This means that the relative delay time for a given reach will be different in the two models.

The same storage - discharge relationship was assumed for the catchment upstream and downstream of Mt Bold Reservoir. It was therefore necessary to adjust the value of k_c for the RORB model to Mt Bold Reservoir by the ratio of the average flow distances (1.8).

This was verified using the three largest events which are shown in Table 6.7. Using the RORB model to Old Noarlunga, the model was calibrated at Houlgraves Weir. The values of k_c obtained were as expected, 1.8 times greater than the values using the model to Mt Bold Reservoir.

The model parameters adopted for the RORB model to Old Noarlunga are shown in Table 6.10.

Table 6.10 RORB Parameters for the RORB Model to Old Noarlunga

	Design Flows	PMF
Initial Loss	20 mm	0 mm
Continuing Loss	1 mm/hr	0 mm/hr
m	0.7	0.8
k_c	50	30

Chapter 7

Mt Bold Inflow Flood Frequency

7.1. Flood Frequency Analysis

A flood frequency analysis assigns a probability distribution to a series of recorded peak flows. It is assumed that the recorded series is stationary with respect to time. Long term climate fluctuations are not usually considered (Craig et al., 1993).

The estimation of large floods often requires the extrapolation of rating curves which can lead to an error in the recorded peak flows.

7.1.1. Length of Record

A 'moderate' length of record (10 to 15 years) is required for flood frequency analysis (IEAust 1987). In the absence of a sufficiently long record, flood events may be obtained from hydrologically similar catchments. The reliability of the results increases as the length of record is increased.

Great care must be shown in extrapolating the results of the flood frequency analysis. IEAust (1987) recommends that the maximum flood that should be directly obtained from a flood frequency analysis with a long period of record is the 100 year ARI flood, although in some circumstances it may be necessary to estimate the 500 year ARI flood.

Victorov (1971) analysed the effect of the period of record on the predicted flow probability for the log Pearson III distribution. It was concluded that the use of a short period of record meant that it was unlikely to obtain correct estimates of the flow probabilities.

Nathan and Weinmann (1992) recommended that for short periods of record, only two parameters should be calculated from the data. If a three parameter distribution is used, such as the log Pearson III distribution, the third parameter should be determined from average regional values.

7.1.2. Annual and Partial Series

In an annual series the highest instantaneous flows in each year of record are selected. The number of events is therefore equal to the number of years of record. The main advantage of this method is that because only one flow from each year is selected, it is quite likely that the flows are independent.

In a partial flood series all of the independent flows above a certain predetermined value are selected (Jayasuriya and Mein, 1985; Ashkanasy and Tickle, 1986). The year in which the flows occur is not considered. Because the base value is arbitrary, the number of events does not have to equal the number of years of record. The engineer and the community are generally concerned in all floods that have independent effects and therefore are basically interested in the partial series (Laurenson, 1987).

The probability of floods is expressed by two different terms (IEAust, 1987; Laurenson, 1987):

1. The annual exceedance probability, or AEP, is the probability of a certain flow being exceeded within a period of a year. The term AEP is only applicable to an annual series.
2. The average recurrence interval; or ARI, is the average value of the period between exceedances of a given event magnitude. The term ARI is only strictly applicable to a partial series.

IEAust (1987) recommends the use of an annual series for an ARI of greater than 10 years and the partial series for an ARI less than this. In most cases, the results obtained by both annual and partial series analysis for ARIs of greater than 10 years are very similar.

7.1.3. Exceedance Probability

The use of the AEP may lead to a misleading sense of security by the public (Laurenson, 1987). This is because the AEP appears to be very low. It is important that the exceedance probability of the design life is also determined. The exceedance probability can be determined using Equation 7.1.

$$J = 1 - (1 - P)^L \tag{7.1}$$

where: J is the exceedance probability;

L is the design life (years); and

P is the annual exceedance probability.

7.1.4. Plotting Position

In order to plot the data on a frequency diagram, the plotting position must be calculated. A general expression for calculating the plotting position in terms of the annual exceedance probability as a fraction is shown in Equation 7.2.

$$PP_m = \frac{m - \alpha}{N + 1 - 2\alpha} \tag{7.2}$$

where: m is the rank of the flood;

N is the number of years of record; and

α is a constant.

A value of α of 0.4 is recommended for the log Pearson III distribution because it results in an unbiased estimate of the population standard deviation, or the slope of the frequency curve (Srikanthan and McMahon, 1981; Laurenson, 1987; IEAust, 1987). Equation 7.2 therefore becomes:

$$PP_m = \frac{m - 0.4}{N + 0.2} \tag{7.2a}$$

7.1.5. Confidence Limits

Confidence intervals give the range within which the actual population is expected to lie for a given probability. The confidence limits on a frequency plot enclose the confidence interval.

IEAust (1987) recommends Equation 7.3 for constructing confidence limits about the discharge Q_y. For a selected AEP:

$$\log(CL(Q_Y)) = \log Q_Y \pm \frac{F\delta S}{\sqrt{N}} \tag{7.3}$$

where: Q_y is the peak flow (m^3/sec);

F is the frequency factor for the Normal Distribution;

δ is a parameter for determining the standard error of a Pearson III Distribution, and the values are tabulated in IEAust (1987);

S is the standard deviation of logarithms of flows (base 10); and

N is the number of years of record.

The confidence limits of interest are usually the 5% and 95% confidence limits. For these F is equal to 1.645 and Equation 7.3 becomes:

$$\log\left(CL_{5,95}\left(Q_Y\right)\right) = \log Q_Y \pm 1.645\frac{\delta S}{\sqrt{N}} \qquad (7.3a)$$

Confidence limits can be calculated for a log Normal distribution using Equation 7.3a for the case of a skew of zero.

7.1.6. Theoretical Distributions

The choice of an appropriate theoretical distribution is often difficult because of the large numbers of distributions and the fact that different fit criteria may indicate different distributions as being the most appropriate (Kopittke et al., 1976).

> *"There is no well proven theoretical connection between any analytical form of a distribution and the underlying mechanisms governing flood flows."* (Nathan and Weinmann, 1992)

Many different studies have tested the suitability of various distributions to Australian and overseas data. There still remains uncertainty and debate over the form of the probability distribution of flood magnitudes (Kirby and Moss, 1987). The chosen distribution should consistently provide a good fit to the data.

Kopittke et al. (1976) analysed annual series recorded in Queensland using 10 different distributions. The method of moments was used to fit the ten different distributions and the quality of fit was determined using four different goodness of fit tests. It was concluded that the Weibull distribution provided the best fit in sixty percent of the cases. The log Pearson III distribution and the Boughton Empirical distribution produced the best overall results.

McMahon and Srikanthan (1981) analysed the peak annual series of 172 Australian streams using moment ratio diagrams. It was concluded that the log Pearson III distribution was the most appropriate distribution.

The log Pearson III distribution is recommended by IEAust (1987) for an annual series. This recommendation is based on studies of both Australian and US data. IEAust (1987) also notes that the log Pearson III distribution has an upper limit when the skews of the logs of the peak flows are negative. This is the case with the inflow record at Mt Bold Reservoir.

The log Pearson III distribution is also recommended because the use of a standard distribution leads to consistency in design practices (IEAust, 1987). Recent studies have however, analysed the extended periods of record and various other distributions have been recommended.

Nathan and Weinmann (1991 & 1992) used the regional flood frequency method of regional *L*-moments combined with the Wakeby or Generalised Extreme Value (GEV) distributions fitted by probability weighted moments (Greenwood et al., 1979). This regional flood frequency method was considered to robust, efficient and superior to other methods.

A flood frequency analysis undertaken for Warragamba Dam was described in Craig et al. (1993). A number of conventional distributions were fitted to the annual and partial series. It was concluded that the log Pearson III distribution significantly over-estimated low frequency events. The most appropriate distribution was a mixture of two log Normal distributions. The mixed distribution model was recommended for annual flood series which exhibit reverse curvature when plotted on log Normal probability paper.

Vogel et al. (1993) analysed the suitability of different distributions for Australian flood peaks using data from 61 sites. *L*-moment diagrams and Beards' non-parametric test were used to determine the most suitable distribution. The Generalised Pareto and log Pearson III distributions were found to be the most suitable distributions.

A distinction was then made between winter and summer rainfall dominated regions. The southern portion of South Australia (including the Onkaparinga Catchment) was within the region designated as winter rainfall dominated. The Generalised Extreme Value and Wakeby distributions were recommended for the winter rainfall dominant regions. The Generalised Pareto and Wakeby distributions were recommended for the remainder of the continent. The Wakeby distribution is described in Houghton (1978) and Landwehr et al. (1979 a,b). The log Pearson III and log Normal distributions performed credibly across both of the regions.

Because of the uncertainty as to the most suitable distribution, a number of different distributions were tested for the Onkaparinga Catchment.

7.1.6.1. WSO87

The statistical package WSO87 (Kopittke et al., 1976; Kopittke and Tickle, 1976) was used to fit theoretical distributions to the data. Although not all of the distributions mentioned above were fitted, it was felt that the distributions tested were representative and provided a useful comparison.

WSO87 fits the following distributions:

- Pearson III distribution;
- log Pearson III distribution;
- Normal distribution;
- log Normal distribution;

- Gumbel distribution;
- log Gumbel distribution;
- Power Transformed Normal distribution; and
- Fisher-Tippett III distribution.

Four different goodness of fit tests were then used to determine how well the chosen mathematical distribution fits the data. For each test the smallest results represented the best fit. The four tests were the:

- Difference Test
- Modified Difference Test
- Chi-Squared Test
- Modified Chi-Squared Test

Kopittke et al. (1976) noted that the goodness of fit tests are a necessary but not a sufficient criterion for selection of the best distribution. This is because distributions which have a large number of parameters are more flexible and although they adhere to the sample data better, erratic events in the data might mislead the user in estimating the frequency curve.

7.1.6.2. Multiple Distributions

A flood frequency analysis assumes that the recorded series is representative of a single distribution. The Onkaparinga Catchment however is subject to two distinct rainfall mechanisms; rainfalls result from tropical moisture or low frontal systems passing through the Great Australian Bight (Section 5.1). It may therefore be possible to separate the events according to their meteorological origin and fit separate theoretical distributions.

Ashkanasy and Weeks (1975) considered the problem of two distinct classes of events within a catchment. The events were separated and separate log Normal distributions were fitted to each data set. It is noted that a misleading situation can occur when only a few (or even no) events of the alternative population have been recorded.

Because of the limited data in the Onkaparinga Catchment, the events were not separated according to their causative rainfall mechanism. It is suggested that this is an area which should be further researched.

7.2. Mt Bold Reservoir

Initially a flood frequency analysis was considered using the outflows from Mt Bold Reservoir. This was prevented because:

- the reservoir level has only been measured continuously since July 1986;
- some uncertainty still remains concerning the accuracy of the spillway rating curve;
- there exists considerable doubt as to the accuracy of the recorded gate openings, especially for small openings; and
- there is reason to believe that not all gate openings have been accurately recorded.

Because it was not possible to undertake a flood frequency analysis using the outflows, a flood frequency analysis was undertaken using inflows to Mt Bold Reservoir. This was done by modifying the record at Houlgraves Weir (AW503504) which is situated just upstream of the reservoir. The stage record has been continuous at Houlgraves Weir since 17 April 1973. During the flood of 30 August 1992 the station was rated up to 330 m^3/sec, which is more than 75 percent of the maximum flow recorded at the site (Section 3.6.2).

The first referenced flood frequency analysis at Houlgraves Weir used a partial series from the available 11 years of record from 1973 to approximately 1984 (Kotwicki, 1984). A log Pearson III distribution was then fitted to this data. The relatively short period of record meant that the analysis might not accurately represent the true distribution of floods.

Although the flood frequency analysis undertaken in Kotwicki (1984) had only used a relatively short data set, the results have been used widely.

7.2.1. Flood Frequency Analysis Undertaken Prior to 30 August 1992

With the longer data set available, a flood frequency analysis was conducted in early August 1992 for Houlgraves Weir. Both a partial and annual series were extracted from the available data set from the 17 April 1973 to the 30 April 1992. This represents 19 years of continuous reliable record. The peak events were extracted and a log Pearson III distribution was fitted to the data using HYDSYS.

During the 19 years of record, the maximum peak flow was 243 m^3/sec recorded on 26 June 1981. On the 30 August 1992 a peak flow at Houlgraves Weir of 431 m^3/sec was recorded. This flood event is described in Section 3.6.2.

The peak flow of 431 m^3/sec was significantly larger than any other peak flow in the record. Although the length of record exceeded the minimum of 10 to 15 years that is recommended by IEAust (1987), this one flow had a very large effect on the flood frequency curve. Analyses done which include or exclude the 431 m^3/sec flow produce significantly different results. This is shown in Figure 7.1 for an annual series and a log Pearson III distribution.

The frequency analysis conducted prior to August 1992 resulted in the flow of 431 m³/sec being assigned an AEP of approximately 1 in 500, whereas the analysis which included the event of August 1992 assigned an AEP of approximately 1 in 50. This is indicative of the subjectivity and uncertainty of conducting a flood frequency analysis on a relatively short length of record (Section 7.1.1).

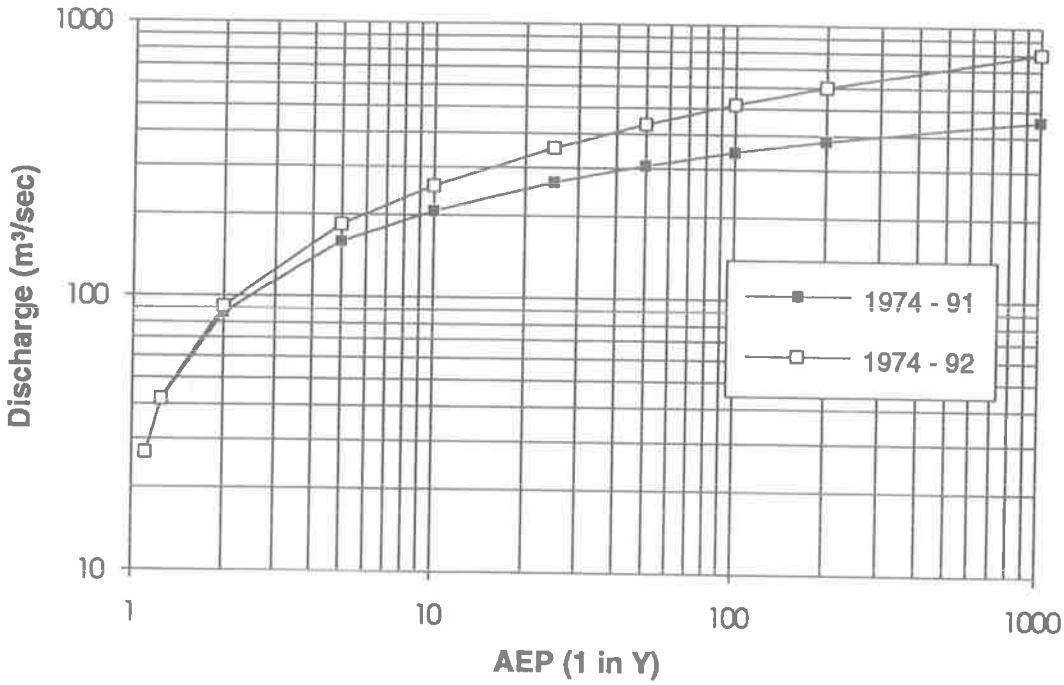


Figure 7.1 Flood Frequency Curves for Houlgraves Weir Excluding and Including August 1992

7.3. Extended Data Set of Inflows to Mt Bold Reservoir

To avoid the problems in using a short data set for flood frequency analysis, considerable effort was made to extend the data available at Houlgraves Weir. The record at Clarendon Weir (situated downstream of both Houlgraves Weir and the Mt Bold Reservoir) was examined to see if it could be used.

The stage record at Clarendon Weir commenced in 1889, but it is not continuous with the stage being recorded approximately every 5 hours. Sid Stephens from the Water Resources Section of the E&WS calculated the maximum instantaneous flows in each month at Clarendon Weir from 1888 to 1976. These calculations are supported by the summary of peak flows at Clarendon Weir tabulated in E&WS (1933).

The construction of the Mt Bold Reservoir between 1932 and 1938 meant that the flows measured after about 1936 were affected by the reservoir. The flows before 1936 however represent natural flow in the Onkaparinga River and the record was useful in extending the data set of inflows to Mt Bold Reservoir.

The annual peak flows recorded in the Onkaparinga River are shown in Figure 7.2. Although the stage readings began in 1888, there is a gap in the records from 1893 to 1896 and hence the useful period of record is from 1897 to the end of 1935.

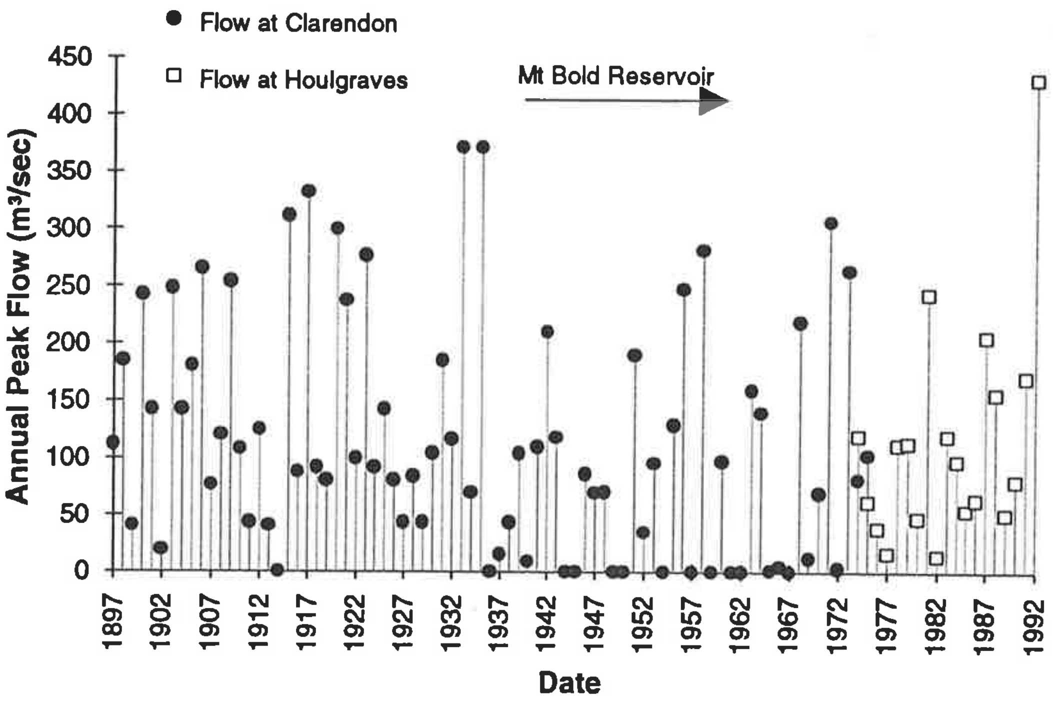


Figure 7.2 Annual Series of Peak flows in the Onkaparinga River

The flows measured at Clarendon Weir were not directly applicable to the inflows to Mt Bold Reservoir. The effect of the different catchment areas had to be considered with the measured peak flows at Clarendon Weir being converted to equivalent peak inflows to Mt Bold Reservoir.

The flow at one location can be approximated from the flow at another location if the respective catchment areas are known. The relationship is shown in Equation 7.4.

$$Q_1 = Q_2\left(\frac{A_1}{A_2}\right)^b \qquad (7.4)$$

From Akter (1992), an exponent of 0.64 was chosen for use in Equation 7.4. Equations 7.5 and 7.6 were therefore used to calculate corresponding inflows to Mt Bold Reservoir.

Flow at Houlgraves Weir to Inflow to Mt Bold $\quad Q_{mb}=1.13\ Q_h \quad$ (7.5)

Flow at Clarendon Weir to Inflow to Mt Bold $\qquad Q_{mb}=0.921Q_c \qquad$ (7.6)

The recorded peak flows at Clarendon Weir from 1897 to 1935 were converted to corresponding peak inflows to Mt Bold Reservoir. The 39 years of modified data and the 19 years of records from Houlgraves Weir were then combined to produce a record of 58 years.

7.3.1. Choice of Distribution

The data and statistics of the full record, along with the fitted distributions calculated using WSO87 for the annual series are shown in Appendix H. Five of the distributions and the peak inflows are shown in Figure 7.3.

Great care must be taken in extrapolating the fitted theoretical distributions. Although WSO87 calculates values for the distribution up to an AEP of 1 in 10,000, rainfall based methods should be preferred for calculating flows which have a low AEP.

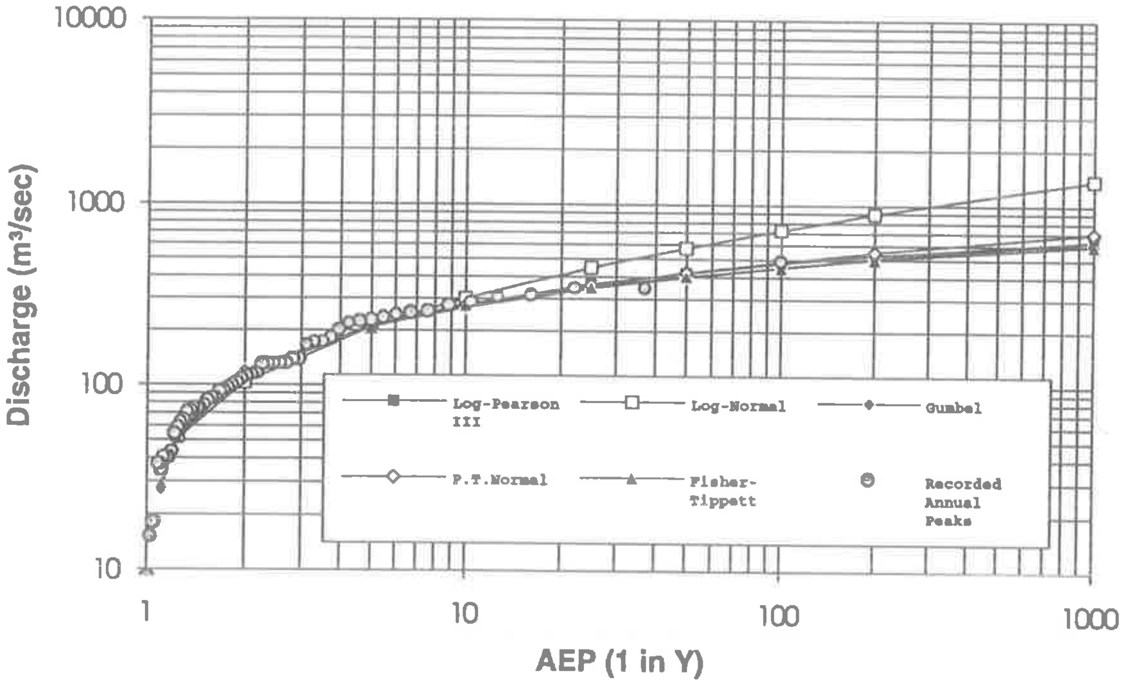


Figure 7.3 Five Different Theoretical Distributions for an Annual Series of Inflows to Mt Bold Reservoir (1897 - 1935, 1974 - 1992)

Following the procedures in IEAust (1987), a rainfall based method was preferred to a flood frequency based method for the inflows to Mt Bold Reservoir for floods having an AEP of less than 1 in 50 years.

The split record was examined to see whether there was any disparity between the early record 1897 to 1935 and the later record 1974 to 1992 (including August 1992). These results are shown in Figure 7.4. There is a small difference between the two log Pearson III distributions, but very little difference between the two log Normal distributions.

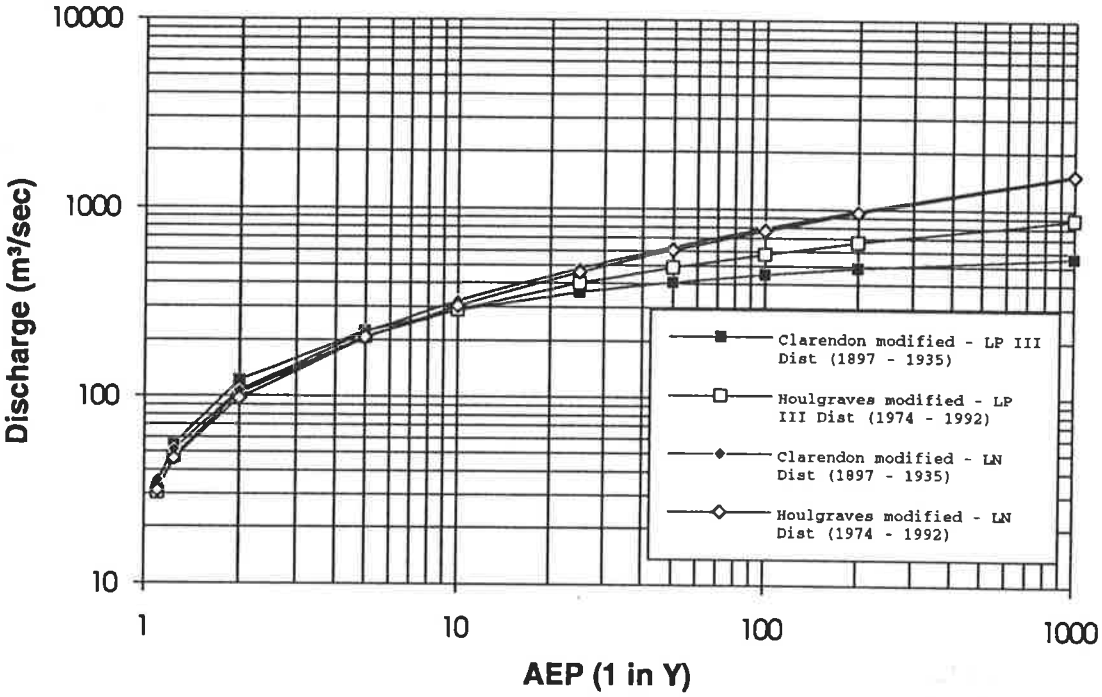


Figure 7.4 Comparison of Flood Frequency Analysis for Inflows to Mt Bold Reservoir for 1897 to 1935 and 1974 to 1992

7.3.1.1. Goodness of Fit Tests

The four different goodness of fit tests in WSO87 were used to determine the theoretical distribution which provided the best fit to the data. Unfortunately the tests indicated different distributions:

- Both the difference and modified difference tests indicated that the best theoretical distributions were the log Pearson III, the Power Transform Normal and the Fisher-Tippett distributions.
- The chi-squared and the modified chi-squared tests indicated that the best distribution was the log-Normal distribution.

From Figure 7.3 it is clear that all of the theoretical distributions were similar, except for the log Normal distribution which was above the others.

7.3.1.2. Inclusion of the April 1889 Flood

The annual series of inflows does not include the flood of April 1889. The peak flow in the Onkaparinga River was not accurately gauged, but has been estimated to have been 680 m³/sec at Clarendon. This results of modelling of this event are included in Daniell and Hill (1993c). This event is quite significant as Section 7.2.1 shows the dramatic effect that a single large event can have on the calculated flood frequency.

Unfortunately the record at Clarendon Weir only commenced in 1896 and therefore the flow in April 1889 cannot strictly be included in the annual series. It is however important to consider the effect of its inclusion in the annual series. The flow of 680 m³/sec at Clarendon was converted to a corresponding inflow to Mt Bold Reservoir of 626 m³/sec using Equation 7.6. The theoretical distributions were then fitted to an annual series which included the April 1889 flow.

The effect of the inclusion of the April 1889 flood is shown in Figure 7.5. The log Pearson III distribution is consistently below the recorded values, from an AEP of 1 in 25 onwards.

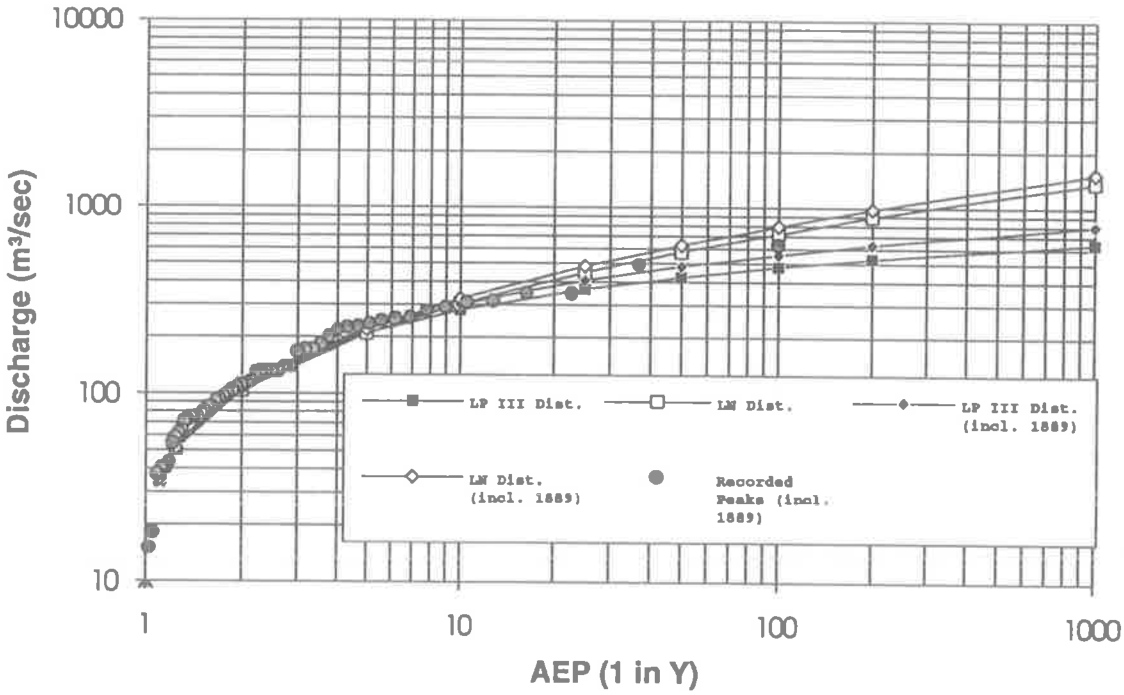


Figure 7.5 Flood Frequency Curves for Inflows to Mt Bold Reservoir Showing the Effect of the Inclusion of 1889 in the Annual Series

The log Normal distribution was chosen as the best distribution to represent inflows to Mt Bold Reservoir. This distribution was chosen because it performed best for the chi squared and modified chi squared test. Although the log Normal distribution plots above the other distributions, it is believed that this accounts for the April 1889 event.

The chosen distribution is shown in Figure 7.6. This distribution does not include the April 1889 event.

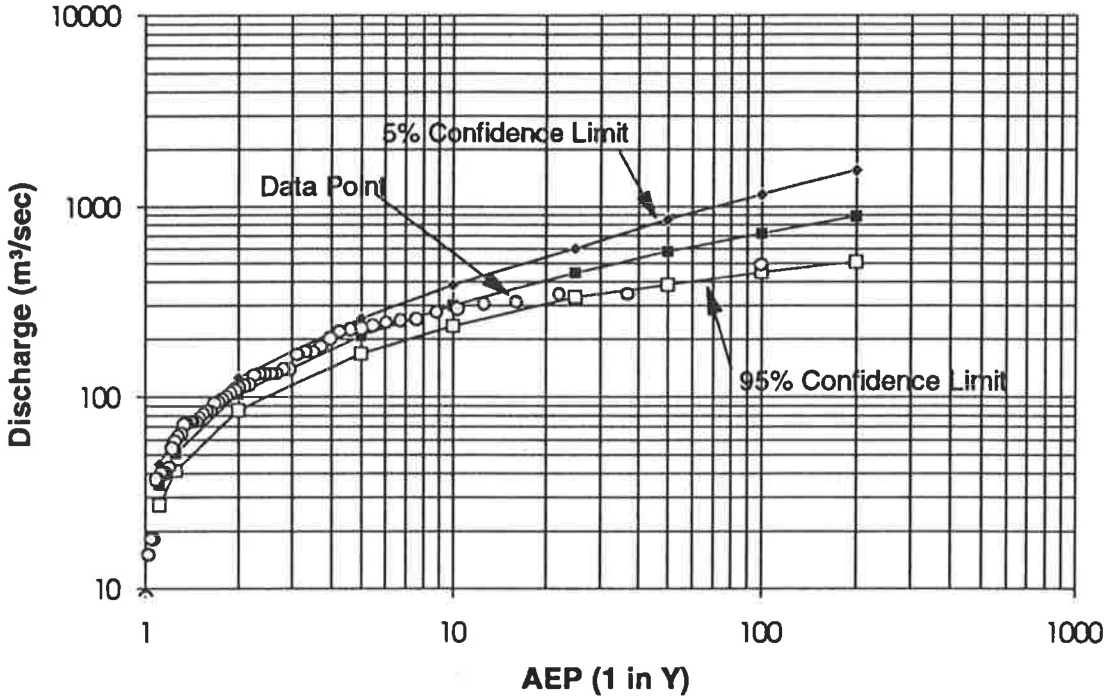


Figure 7.6 Fitted Log Normal Distribution for an Annual Series of Inflows to Mt Bold Reservoir (1897 - 1935, 1974 - 1992)

7.4. Design Inflows Calculated using Design Rainfalls

Design rainfalls were calculated for the Mt Bold Catchment using the guidelines included in IEAust (1987). The design rainfalls are discussed in more detail in Chapter 5. The design rainfall depths and the temporal patterns were used to determine design inflows for Mt Bold Reservoir.

The determination and calibration of the RORB model is discussed in Sections 6.3 and 6.4. The chosen optimal model parameters were used to rout the rainfall excess to determine the inflows into Mt Bold Reservoir for a range of ARIs and durations.

In order to determine the design flows using design rainfalls, it was necessary to calculate the critical storm duration. A range of storm durations and ARIs were tested. The results for the calculated inflows are shown in Figure 7.7.

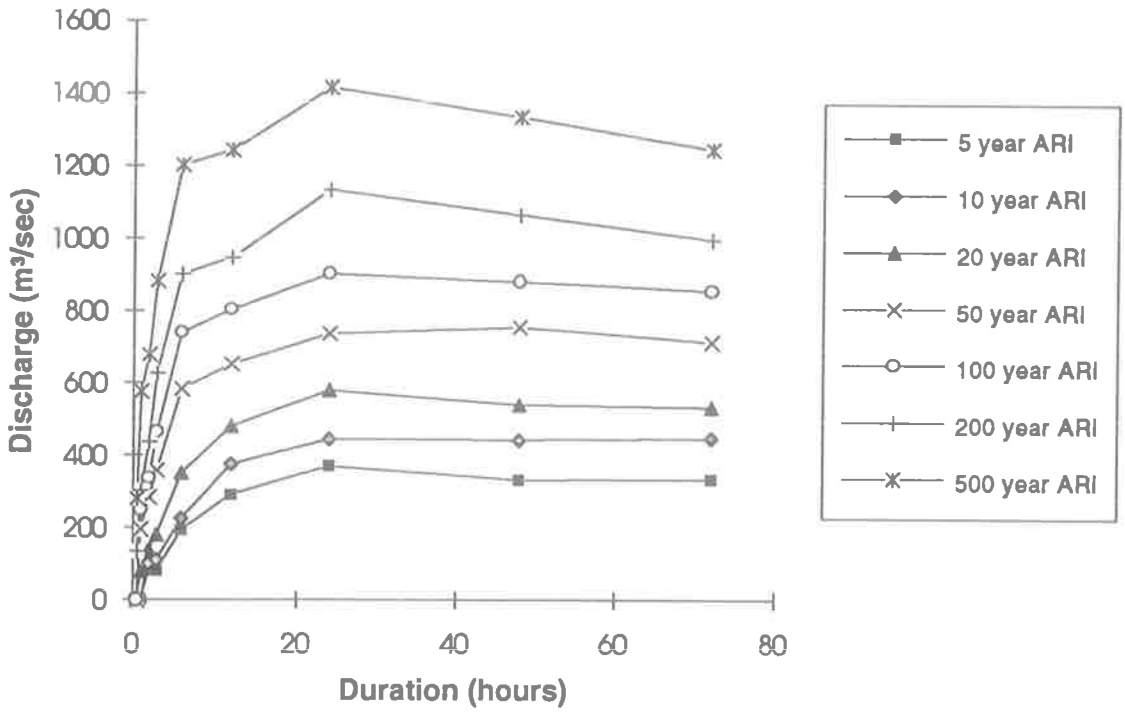


Figure 7.7 Design Inflows for Mt Bold Reservoir using Unmodified Design Rainfalls

It is clear from Figure 7.7 that the critical storm duration for determining design inflows was approximately 24 hours. The routing effect of the reservoir may however result in a longer critical duration for design outflows. The inflows were therefore routed through the reservoir assuming that the reservoir was at full supply level prior to the event and that the gates were operated according to E&WS official operating policy. The modelling of the gate operation is discussed in more detail in Section 6.3.2.1. The effect of the initial level of the reservoir on the calculated outflow is discussed in more detail in Chapter 8. The calculated outflow peaks are summarised in Figure 7.8.

From Figure 7.7 and Figure 7.8 it is clear that the critical duration for both inflow and outflow was 24 hours.

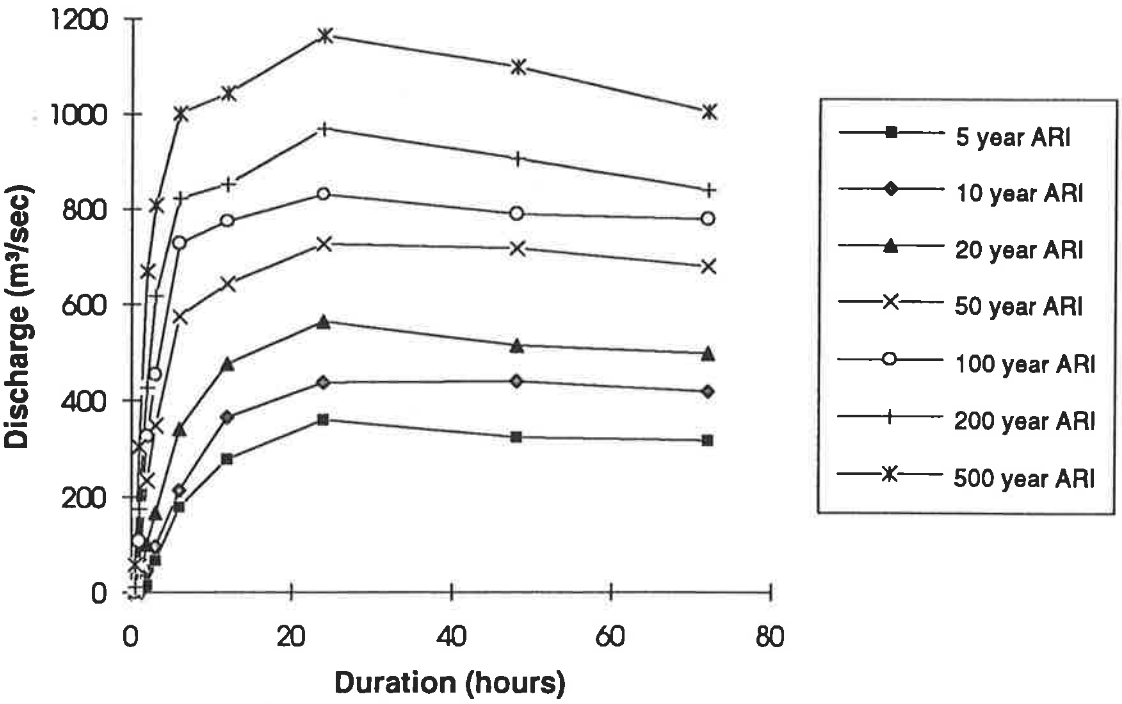


Figure 7.8 Design Outflows from Mt Bold Reservoir (using unmodified design rainfalls, reservoir initially at FSL and gates operated according to E&WS policy)

7.5. Comparison of Modelled and Historical Flood Frequencies

Design flows can be calculated using both rainfall and flood frequency based methods. It was therefore determined to check the consistency of the two methods by plotting the design flows obtained using the design rainfalls on a flood frequency curve. This is shown in Figure 7.9.

It is clear from Figure 7.9 that the flows obtained using design rainfalls exceeded those obtained from flood frequency analysis. This discrepancy could have resulted from many factors including:

1. Errors in the modelling of streamflow.
2. Errors in the flood frequency curve. This could have resulted from the incorrect choice of distribution or using an incorrect method of transposing peaks from one site to another.
3. Errors in the design rainfalls. The design rainfalls from IEAust (1987) may not have been applicable to the catchment.
4. Errors in the choice of losses used with the design rainfalls.

It is believed that the discrepancy was due to inadequacies in the use of losses with the design rainfalls. This is because of the great care that was exercised in the calibration and modelling of streamflow and the determination of the flood frequency curves.

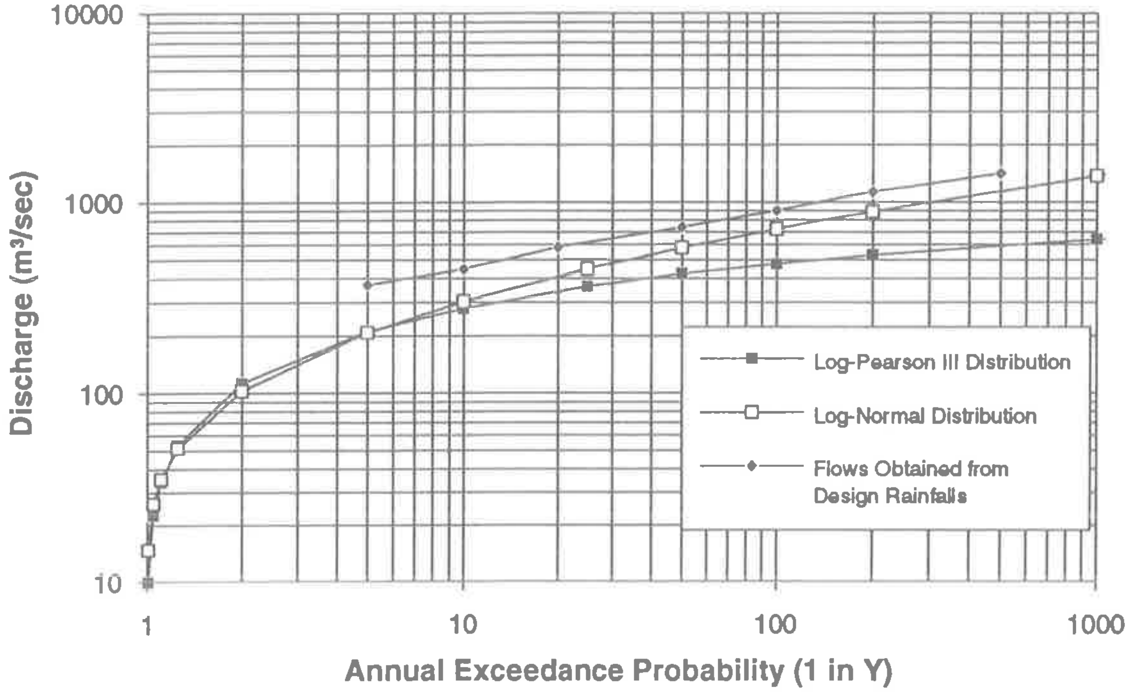


Figure 7.9 Comparison of Design Inflows for Mt Bold Reservoir Obtained from Design Rainfalls and from Flood Frequency Analysis

7.6. Final Design Rainfalls

The design flows derived from storms of different durations are compared to the log Normal distribution in Figure 7.10.

Although it was not expected that the design flows determined by the two different methods would coincide exactly, there should exist some correlation so that no discontinuity occurs at the point at which the rainfall method becomes preferred over the flood frequency method.

It was necessary to modify the flows obtained from design rainfalls so that they corresponded to the flows obtained from the flood frequency analysis. Two different methods were examined for modifying the design flows obtained from design rainfall; increasing the losses and reducing the rainfall depths.

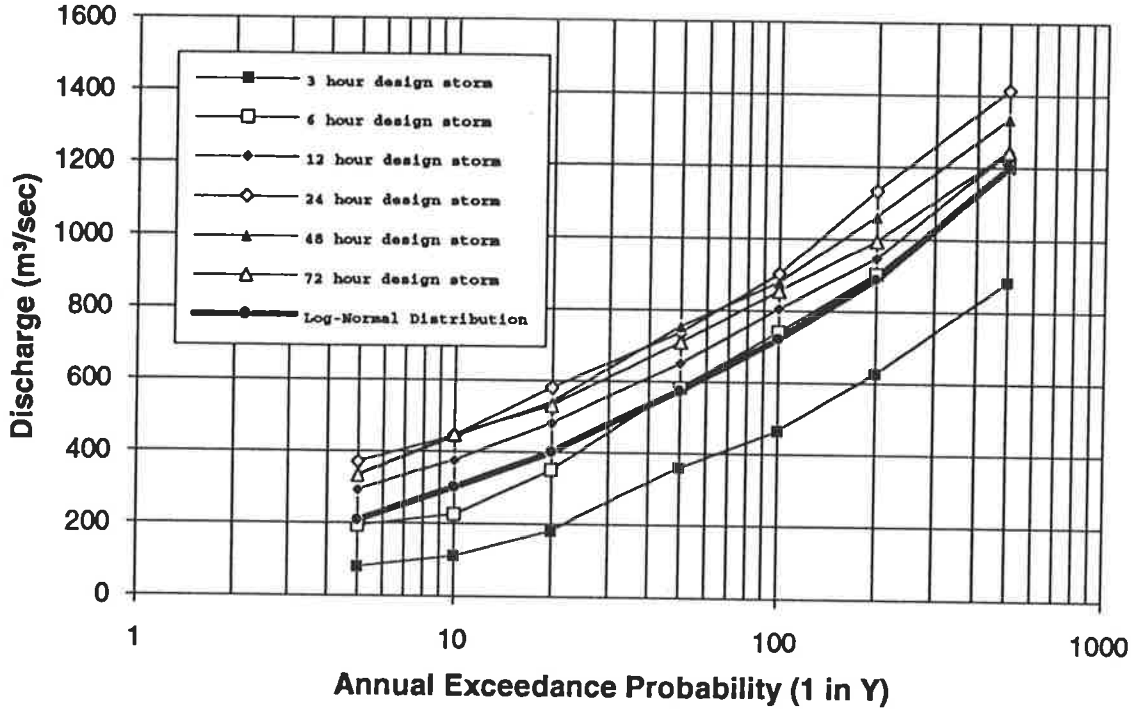


Figure 7.10 Comparison of Inflows to Mt Bold Reservoir Calculated using Different Duration Design Storms

7.6.1. Choice of Losses

The choice of loss parameters is important in determining the magnitude of design flows. It is important to note that the losses derived from calibration of the RORB model do not necessarily apply for use with design rainfalls. This is because the design rainfalls included in IEAust (1987) are bursts which are generally embedded in longer duration storms. It is recommended in IEAust (1987) that lower losses should be used to rout design rainfalls than those derived from the calibration of the model.

Waugh (1991) recommended however that higher losses should be used with the design rainfalls than those derived from the calibration of the model. This is because loss values are often based on the analysis of the largest floods, and such values are likely to be biased due to wet antecedent conditions. There may be some large storms which yield little runoff and therefore these are not included in the calibration process. It was therefore recommended by Waugh (1991) that losses are derived from the analysis of all major storms; not calculated using the sample of major floods.

The recommendations as to the suitable losses for use in design of Waugh (1991) and IEAust (1987) are contradictory. The losses used for design were the same as those obtained from the model calibration.

In a study of the hydrology of the Torrens Catchment (BC Tonkin and Associates, 1975) a similar problem was observed in the design flows obtained from rainfalls exceeding those obtained from a flood frequency analysis. The design flows obtained from design rainfalls were artificially lowered by increasing the losses. An initial loss of 30 mm and a continuing loss of 3 mm/hr were adopted. This decision was not based upon observed losses, but rather the choice was made in order to convert design rainfalls to design flows of the same ARI (Schalk, 1993, pers. comm.).

The models were re-run with the losses proposed by BC Tonkin and Associates (1975) to determine whether those loss parameters were applicable to the Onkaparinga Catchment. The resulting flows are compared to the log Normal distribution in Figure 7.11. It is clear that these losses are too high for the Onkaparinga Catchment and the adoption of these loss parameters would result in the design flows being under estimated.

Losses applicable to the Onkaparinga Catchment could have been obtained, but rather it was decided to reduce the design rainfalls, according to the procedures described in IEAust (1987).

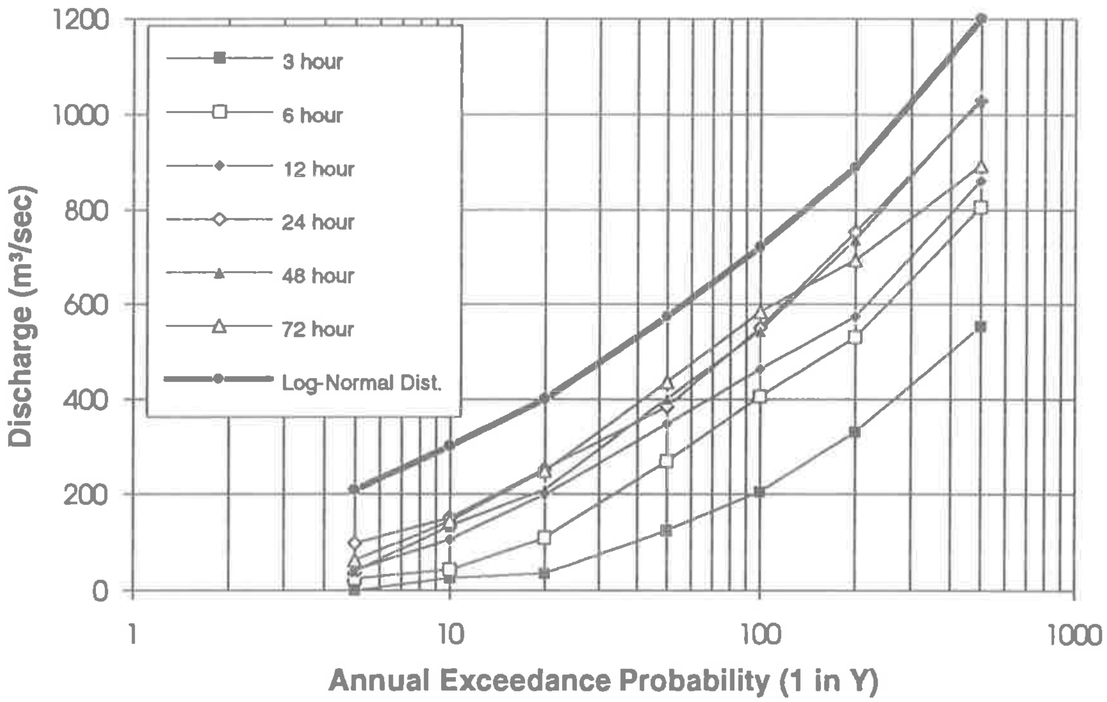


Figure 7.11 Flood Frequency Curves for Inflow to Mt Bold Reservoir Showing the Effect of the Adoption of an IL of 30 mm and an CL of 3 mm/hr for Different Duration Design Storms

7.6.2. Reducing Design Rainfall

IEAust (1987) describes a method of reducing the design rainfall until the design flows correspond to those obtained from flood frequency analysis. For a given ARI, the design flow Q_R is determined using the critical duration. The corresponding value obtained from flood frequency analysis is designated as Q_{FF}. The rainfall depth is then scaled by the ratio of Q_{FF}/Q_R.

The 24 hour design rainfalls were modified until the calculated flows corresponded to those obtained from flood frequency analysis. A rainfall depth of 87 percent of the design rainfalls resulted in the design flows approaching those obtained from the flood frequency analysis. This is shown in Figure 7.12.

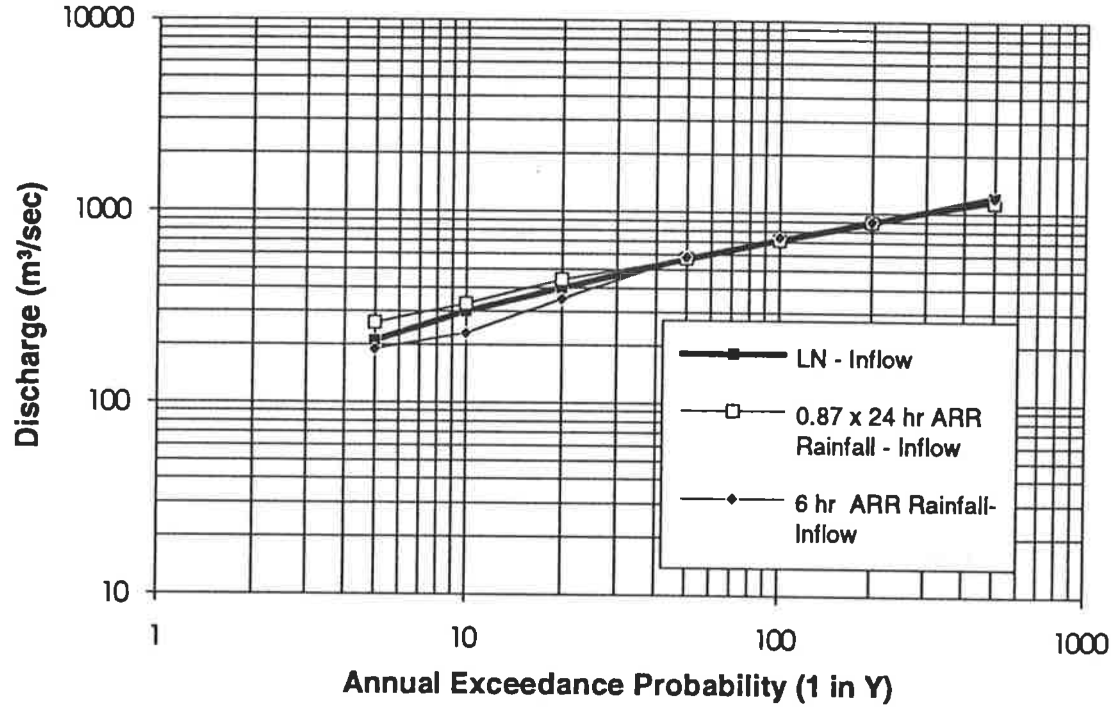


Figure 7.12 Comparison of Design Inflows to Mt Bold Reservoir Calculated using a Modified 24 hour Storm and the Flood Frequency Curve Derived from Recorded Peak Flows

In Figure 7.12 the design flows produced using a six hour storm are similar to the flows obtained from the modified 24 hour storm and the log Normal distribution.

The reduction of the design rainfalls might not have necessarily meant that the design rainfalls were in error, but rather it may have been the areal reduction factors which required adjusting.

7.6.3. Comparison of 6 and 24 hour Storms.

From Figure 7.12, it is clear that either the 6 hour storm or the 24 hour modified storm would have produced results which were consistent with the flood frequency analysis.

The outflow hydrographs were calculated for both the 6 and 24 hour modified duration storms to see which of the duration produced the largest outflow. The 24 hour modified storm produced the largest outflow. It was therefore decided to use the modified 24 hour rainfall as the basis for determining design flows into Mt Bold Reservoir.

There was some concern that the PMF critical duration was only 4 hours (Section 10.4). For the sake of consistency, a shorter duration storm may be appropriate across the whole range of frequencies.

7.7. Treatment of Baseflow

As explained in Section 6.1, the runoff routing model only applies to the surface runoff and not to baseflow. Prior to the calibration of the model, the baseflow had to be extracted.

The hydrographs resulting from design rainfalls only represent the surface runoff component of the flow. In most cases it is therefore necessary to add an estimate of the likely baseflow to the calculated surface runoff hydrograph to produce a design streamflow hydrograph.

In this case no adjustment of the design hydrographs for baseflow had to be made. By modifying the design rainfalls until they produced design flows which corresponded to the flows obtained from a flood frequency analysis, the baseflow was implicitly included. The streamflow records used to generate the flood frequency analysis included both surface runoff and baseflow, and therefore by producing design flows which matched these, the baseflow was included.

Chapter 8

Mt Bold Outflow Flood Frequency

8.1. Effect of Dams on Flood Frequency

The presence of a dam reduces the magnitude of floods downstream. The magnitude of the effect depends on the storage of the reservoir. The effect of the reservoir on the flows will diminish further downstream as the area of uncontrolled catchment increases.

It is important to note the effect of dams on the frequency of floods. There is little effect for very large floods because the volume of the reservoir becomes less significant. The post dam flood frequency curve is below the pre-dam flood frequency curve in the lower flow ranges and approaches it more closely in the upper flow ranges.

8.1.1. Frequency of Flows Below Mt Bold Reservoir

The determination of the flood frequency curve for the flows below Mt Bold Reservoir was hindered by several factors peculiar to the Onkaparinga Catchment.

1. The poor data record of the reservoir level. The reservoir level is dependent upon the pumping policy of the E&WS from the River Murray. As explained in Section 8.2.1, this has recently been changed and therefore recorded storage levels were not applicable for use in determining design flows.
2. The flows downstream of Mt Bold Reservoir are dependent on the operation of the gates. The operation depends on human judgement and if errors of judgement occur this can result in larger outflows than inflows. The operation of the gates is described in more detail in Chapter 4.

3. The record of historical gate operations has inaccuracies and until recently inaccurate ratings for the spillway have been used.
4. The limited period of accurate records of flows in the Onkaparinga River below Mt Bold Reservoir. Although Clarendon Weir has a long period of record, it is primarily a diversion weir for water supply and only one gauging was recorded to verify a theoretical rating that has been used since the construction of the weir (Section 3.2.4.2).
5. The gauging station at Old Noarlunga. This gauging station low in the catchment has been closed. The record was required in order to undertake a frequency analysis and to determine the hydrological response of the river for calibration of the hydrological model.

The determination of the frequency of flows downstream of a reservoir is a complex task. The operation of a gated spillway adds to the complexity. The following reports describe the analysis of flows downstream of a reservoir:

- International Engineering Services Consortium (1969). This report dealt with the determination of the frequency of floods in the Namoi River downstream of Keepit Dam in New South Wales. The problem of determining flows downstream of the reservoir considered the joint probability of the peak inflows and the storage level.
- Laurenson (1973). This paper also studied the frequency of flows downstream of Keepit Dam. It was suggested that the flood frequency should be calculated at a number of downstream sites in order to examine the effect of the reservoir further downstream. A transformation matrix was developed which dealt with the joint probability of inflows and storage level. In order to do this a single probability distribution of storage levels was calculated which applied to the entire year.
- Ahern and Weinmann (1982). This paper described the hydrology of the Goulburn River and some of the general concepts of single event models. The design flows upstream and downstream of Lake Eildon were determined. The problem of joint probability was addressed using the transition matrix described by Laurenson (1973). The inflow and initial storage levels were considered to be independent.
- Water Resource Branch, E&WS (1987). In this study of the operation of the South Para Reservoir spillway gates, the frequency of outflows was determined. The method used followed the procedures described in International Engineering Services Consortium (1969).

8.1.2. Concept of Joint Probability

A particular downstream peak flow can result from different combinations of inflow and storage level. In order to determine the probability of a given outflow, it is necessary to sum the probabilities of the different combinations of storage and inflow which result in a particular outflow. This relationship can be described by Equation 8.1.

$$P(q_o) = \sum P(q_i)\ P(s) \tag{8.1}$$

where: q_i is a peak inflow to the reservoir (m³/sec);

s is a reservoir storage level (a gauge height in metres);

q_o is a peak outflow (m³/sec);

$P(q_o)$ is the probability of an outflow having a peak flow of q_o or greater;

$P(q_i)$ is the probability of a peak inflow of q_i, which is assumed to be equal to the probability of the design rainfall; and

$P(s)$ is the probability that the storage will be at or above a storage level s prior to an inflow of q_i.

The relationship described by Equation 8.1 is represented graphically in Figure 8.1. A line can be drawn which is the locus of storage and probability which results in a particular peak outflow. The probability of that peak outflow being exceeded is represented by the area under the curve.

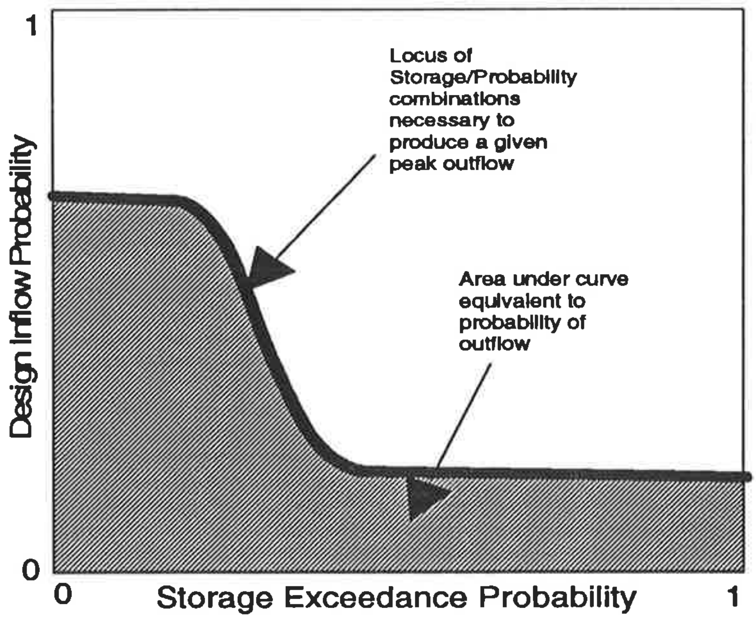


Figure 8.1 Relationship between Inflow Probability and Storage Exceedance Probability to Produce a Given Peak Outflow

8.1.3. Independence of Inflows and Storage Level

Both the transformation matrix proposed by Laurenson (1973) and Equation 8.1 assume that the inflow and the storage level are independent variables. Laurenson (1973) states that, "*this condition is not infrequently satisfied to a reasonable degree of accuracy*". In cases in which this is not the case; where the probability of the inflow and the storage level varies markedly from season to season, the year can be divided into a number of seasons and the partial probabilities calculated. The results are then summed to determine the probability of the downstream flows.

Independence cannot be assumed for Mt Bold Reservoir because there is a defined seasonal distribution of storages and inflows.

It is not possible to determine a season or time of year in which design rainfalls are assumed to occur. Most of the rainfall in the Onkaparinga Catchment occurs in the winter months, although extreme rainfall is more likely to occur in the warmer months when the atmosphere is warmer and therefore able to contain more moisture vapour (Section 3.5).

8.2. Reservoir Levels

The probability distribution of reservoir levels had to be determined in order to calculate the outflow flood frequency curve for Mt Bold Reservoir. The reservoir levels depend on the operating policy of the E&WS and the natural streamflow.

The reservoir level is measured at the dam wall by a continuous recorder which was installed in July 1986 by the Scientific Services Branch of the E&WS. Before 1986 the water level was recorded daily and every time an adjustment was made to the gates.

8.2.1. Operation of System

The water level in the Mt Bold Reservoir depends on the operation of the Metropolitan Adelaide Water Resource Supply System which comprises of a system of storages in the Mt Lofty Ranges and two pipelines from the River Murray. The system is divided into two components, with Mt Bold, Happy Valley and Myponga Reservoirs comprising thc southern system.

The prime objective of the operation of the system is to supply water at a minimum cost, subject to avoiding restrictions to consumers. The other objective is to improve the quality of the supplied water without significantly increasing the cost.

At the commencement of the financial year the operators determine target end of month storages for each reservoir. These are levels at which they aim to have the reservoir at or above at the end of the specified month and are dependent on predicted demand and inflows to the storages. The inflows are based upon 70% exceedance probabilities; that is 70 percent of the time the inflows will exceed this value. Linear programming is then used with the constraints of reservoir capacities and minimum operating levels in order to determine the most cost effective pumping program (Crawley and Dandy, 1992).

At the end of each month, the recorded inflow and storage levels are used to recalculate the target storages.

In order to assist the planning and operation of the system, the Headworks Optimisation Model - Adelaide (HOMA) has been developed (Crawley and Dandy, 1992). The program is designed to simulate the operation of the system and can be used in either operational or planning modes.

8.2.2. Synthetic Data

In order to test the reliability of the system under different operating policies, synthetic data sets were generated. The statistics of the observed inflows were used to generate a longer synthetic data set (Baker, 1991).

The synthetic data was used in conjunction with the HOMA model to generate twenty 100 year data sets, representing 2,000 years of synthetic storage level data.

The average end of month storages for each month were calculated using the synthetic data and these are compared to the historical end of month storages in Figure 8.2.

The synthetic and historical end of month storages are similar, although the effects of the changes in the operating policy are evident.

1. During autumn the synthetic reservoir levels are lower than the historical levels. This is because the new policy is based on 70% exceedance probability inflows whereas the system has in the past operated using the 90% exceedance probability inflows. This results in the storages being held at a lower level during the autumn months.

2. The second trend which becomes apparent is that the synthetic storages are greater than the historical in spring. This is also a result of the new pumping procedures. In the past, pumping from the River Murray was delayed as long as possible in the hope that it could be avoided. The current policy is to pump at a more consistent rate throughout the year and hence avoid the large costs associated with pumping at large rates. This results in pumping commencing earlier and hence the storages being at a higher level in spring.

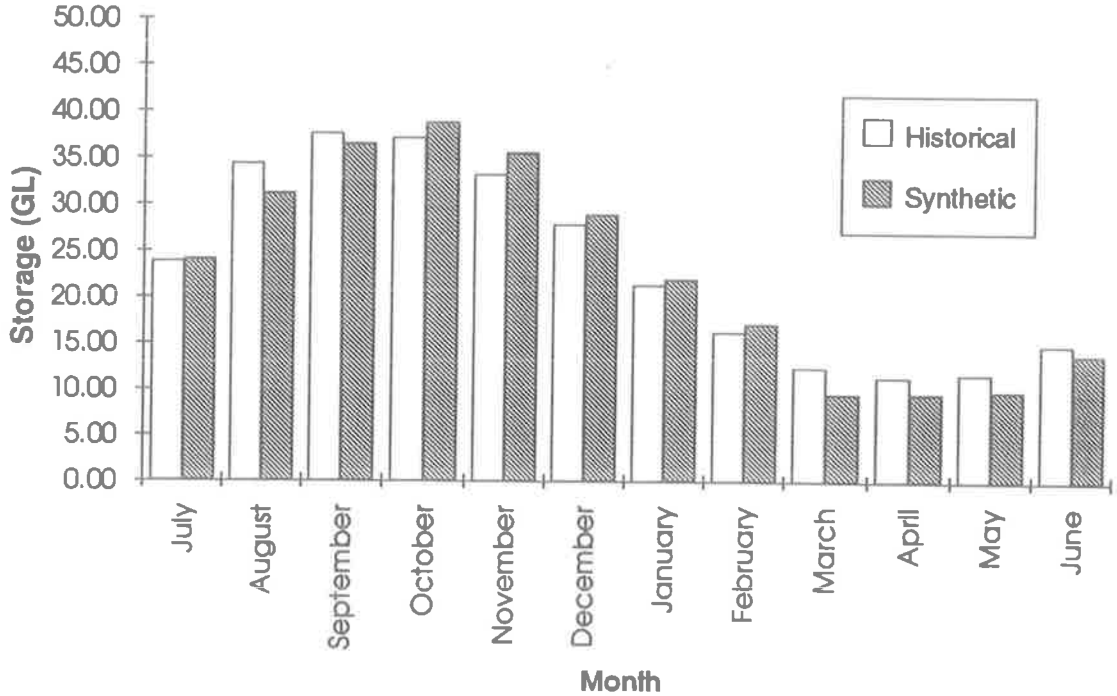


Figure 8.2 Comparison of Historical and Synthetic Average End of Month Storages for Mt Bold Reservoir

8.2.3. End of Month Storage Exceedance Probability

For each of the twenty sets of 100 years of synthetic data, individual end of month distributions were developed, that is 240 distributions for end of month storages. For each of these data sets the end of month storages were ranked and a plotting position assigned. The plotting position was calculated using Equation 8.2.

$$PP_m = \frac{m-0.4}{N+0.2} \tag{8.2}$$

where: m is the rank; and
N is the number of values (100).

The distributions of end of month storages for July are shown in Figure 8.3. Twenty different distributions were calculated from each data set. In Figure 8.3 only the average and the maximum and minimum values are shown of the different distributions.

The end of month storage distributions were calculated for every month and these are shown in Appendix I.

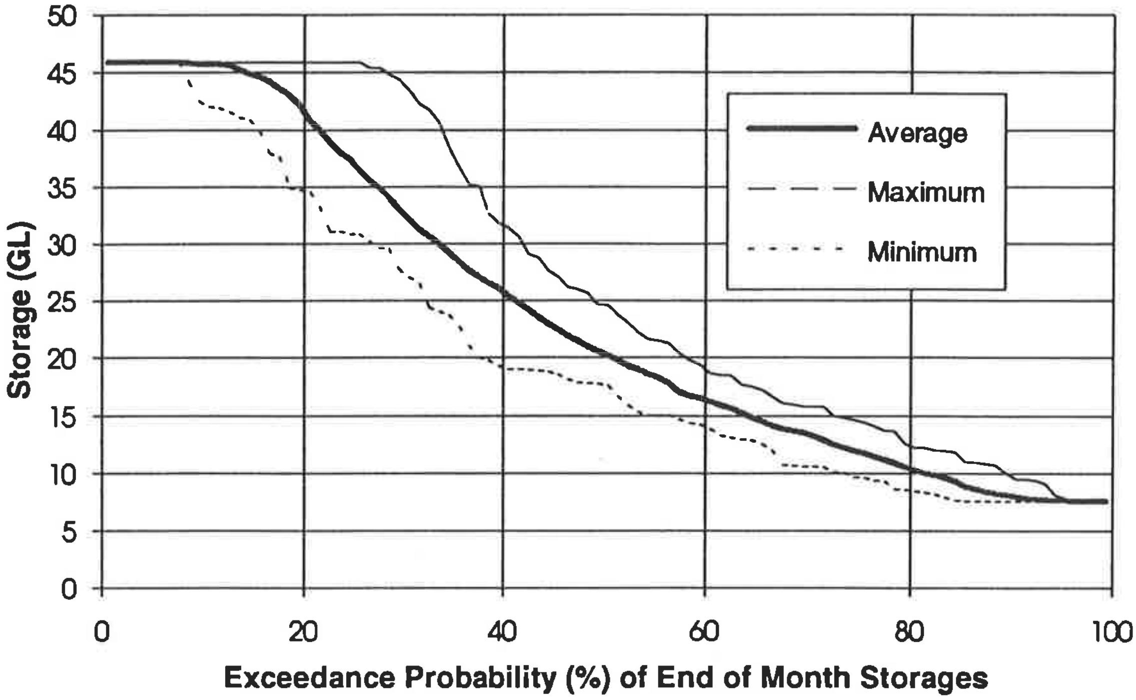


Figure 8.3 End of July Storage Exceedance Probability (based upon 2000 years of synthetic data)

Using the average exceedance probabilities of end of month storages it was possible to determine different storages for given exceedance probabilities and months. The 20, 50 and 80 percent exceedance probability storages are shown for each month in Figure 8.4.

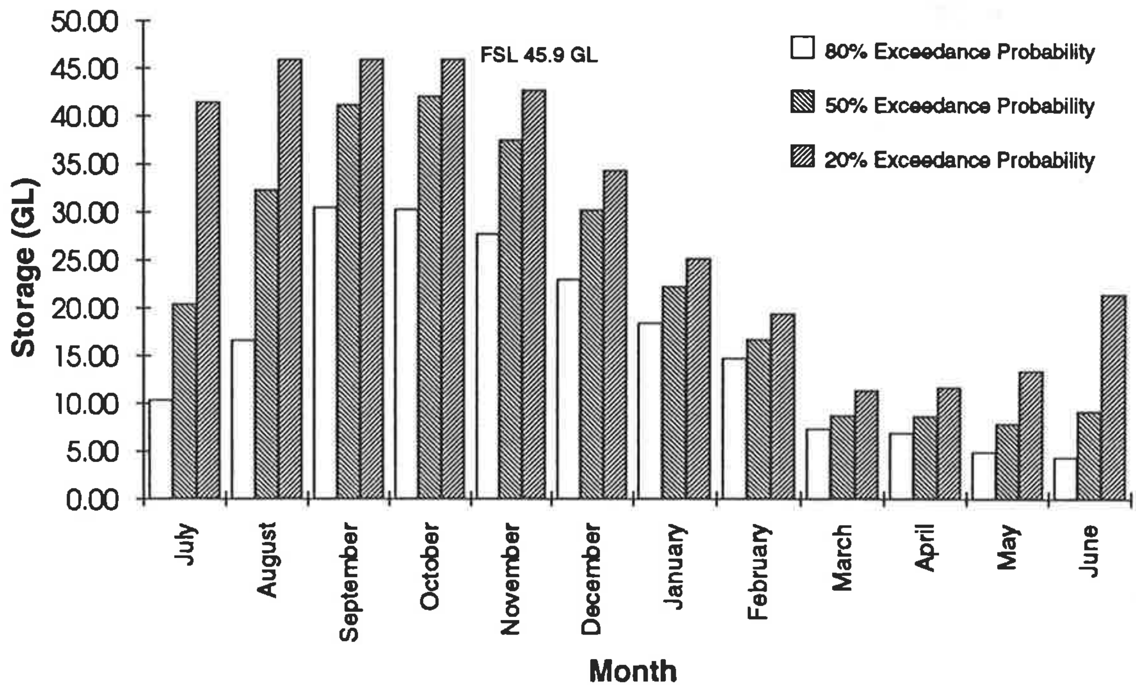


Figure 8.4 Summary of End of Month Exceedance Probabilities (based upon 2000 years of synthetic data)

8.3. Flood Frequency Analysis at Clarendon Weir and Old Noarlunga

The historical records at both Clarendon Weir and Old Noarlunga were used to undertake a flood frequency analysis to check the calculated design flows based upon the joint probability of the design rainfalls and the storage levels.

The recorded flows are not strictly applicable to the calculated flows because:

1. The flows in the Lower Onkaparinga River are dependent upon the level of Mt Bold Reservoir which is determined by the pumping policy of the E&WS. This has changed many times since the completion of the reservoir and therefore the recorded levels may be significantly different to those resulting from the current operating procedures.
2. Prior to 1961, the outflow from Mt Bold Reservoir was not controlled by gates. During this period there would have been more frequent and higher outflows from Mt Bold Reservoir. Because the intention of the current operating procedures is to match outflow with inflow, this is not expected to result in a large error.
3. The recorded flows downstream reflect current gate operations which are different to the current official E&WS policy.

8.3.1. Flood Frequency Analysis at Clarendon Weir

Although the record at Clarendon Weir commenced in 1897, only the record since the completion of Mt Bold Reservoir is appropriate. This results in a record of 45 years.

As stated in Section 3.2.4.2, the rating of Clarendon Weir is relatively unsubstantiated with only one recorded gauging.

Because an annual data series was used to determine the flood frequency of inflow to Mt Bold Reservoir, peak annual values were extracted from the record at Clarendon Weir. Both log Normal and log Pearson III distributions were then fitted to the recorded peaks using WSO87. This is shown in Figure 8.5.

It is clear from Figure 8.5 that neither of the tested distributions fitted the data very well. This was confirmed by the goodness of fit tests. The poor fit was a result of the unusual distribution of flows caused by the reservoir. The theoretical distributions tended to overestimate the flow for floods which have an AEP of less than about 1 in 5.

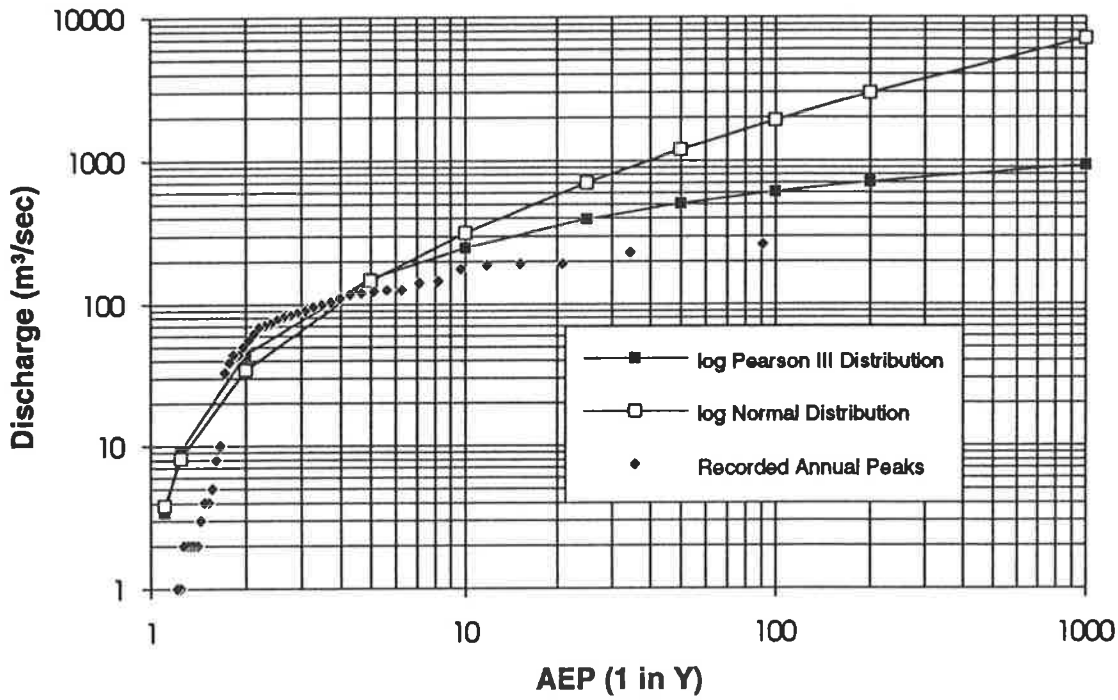


Figure 8.5 Flood Frequency Curves for an Annual Series at Clarendon Weir (1937 - 1993)

A flood frequency analysis was therefore undertaken using a partial data series at Clarendon Weir. Figure 8.6 shows both a log Normal and log Pearson III distribution fitted to the partial series at Clarendon Weir. From this figure it is evident that both distributions are better fits to the partial series than to the annual series. The log Pearson III distribution proved to have the best fit of the two distributions.

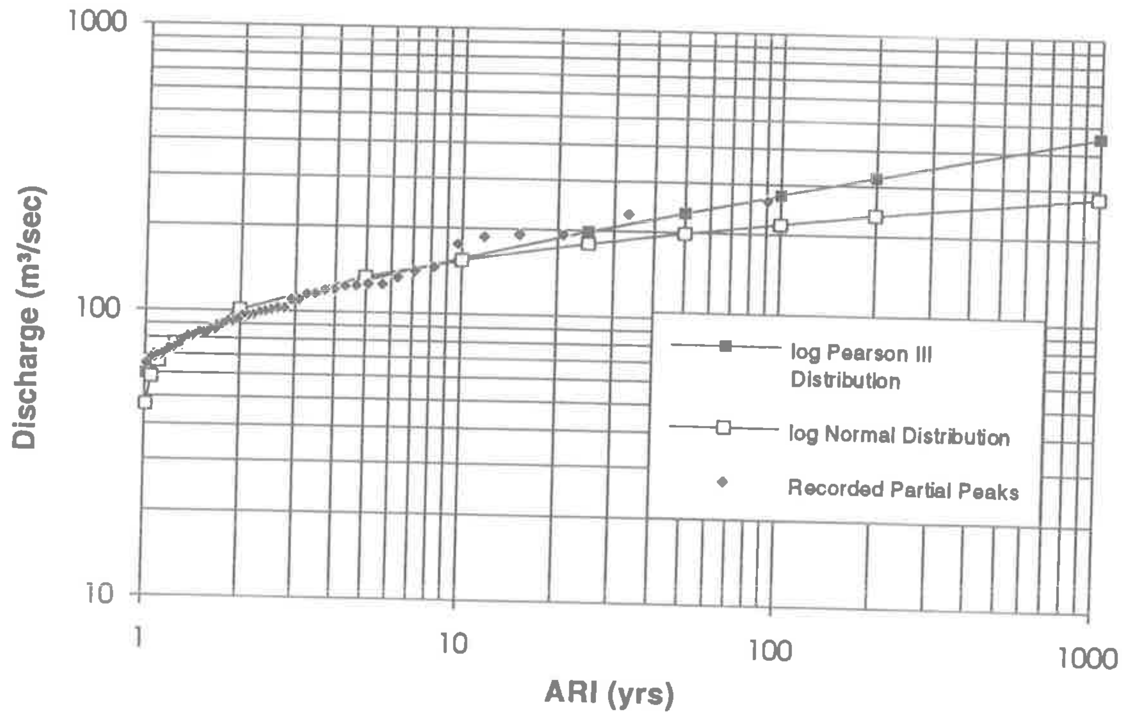


Figure 8.6 Flood Frequency Curves for a Partial Series at Clarendon Weir (1937 - 1993)

8.3.2. Flood Frequency Analysis at Old Noarlunga

A flood frequency analysis was also undertaken using the record at Old Noarlunga. The gauging station was operational from June 1973 to July 1988. The 14 years of data was significantly less than the period of record at Clarendon Weir.

Both the annual and partial series were extracted and the log Normal and log Pearson III distributions fitted to the data using WSO87. The two different series are shown in Figure 8.7 and Figure 8.8. As was the case with the records as Clarendon Weir, both distributions provided better fits for the partial series. The log Pearson III distribution provided the best fit.

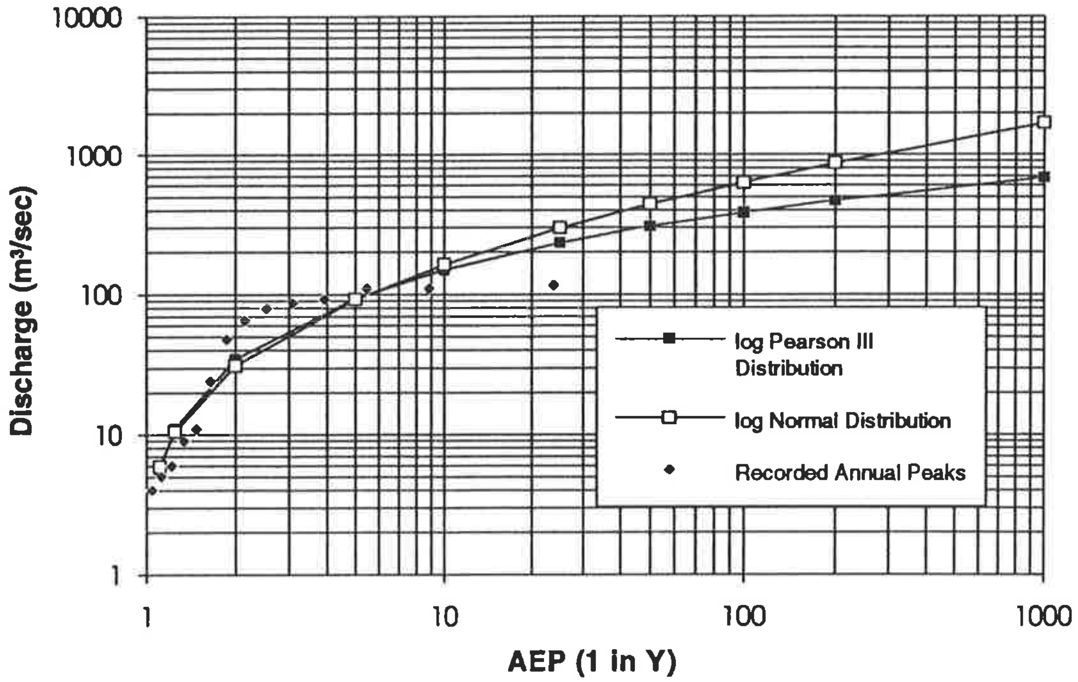


Figure 8.7 Flood Frequency Curves for an Annual Series at Old Noarlunga (1973 - 1988)

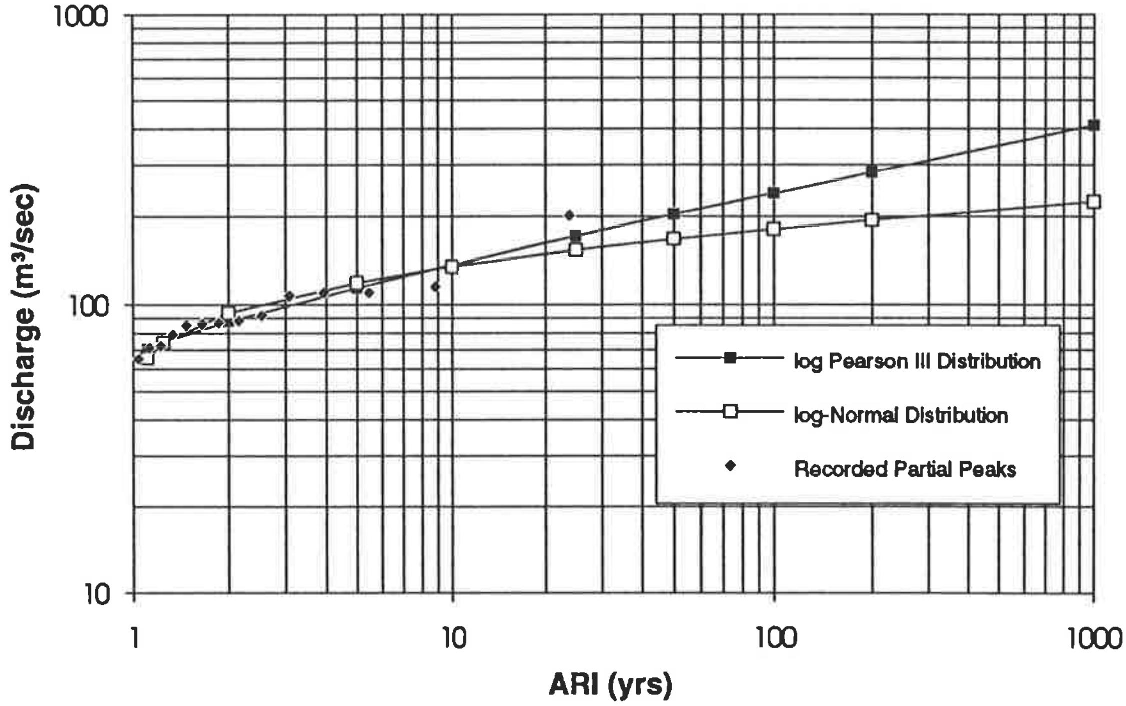


Figure 8.8 Flood Frequency Curves for a Partial Series at Old Noarlunga (1973 - 1988)

8.3.3. Comparison of Flood Frequency Curves for Clarendon Weir and Old Noarlunga

The flood frequency curves for Clarendon Weir and Old Noarlunga were calculated using a log Pearson III distribution fitted to a partial series are compared in Figure 8.9. The flood frequency curve for Clarendon Weir is above that for Old Noarlunga.

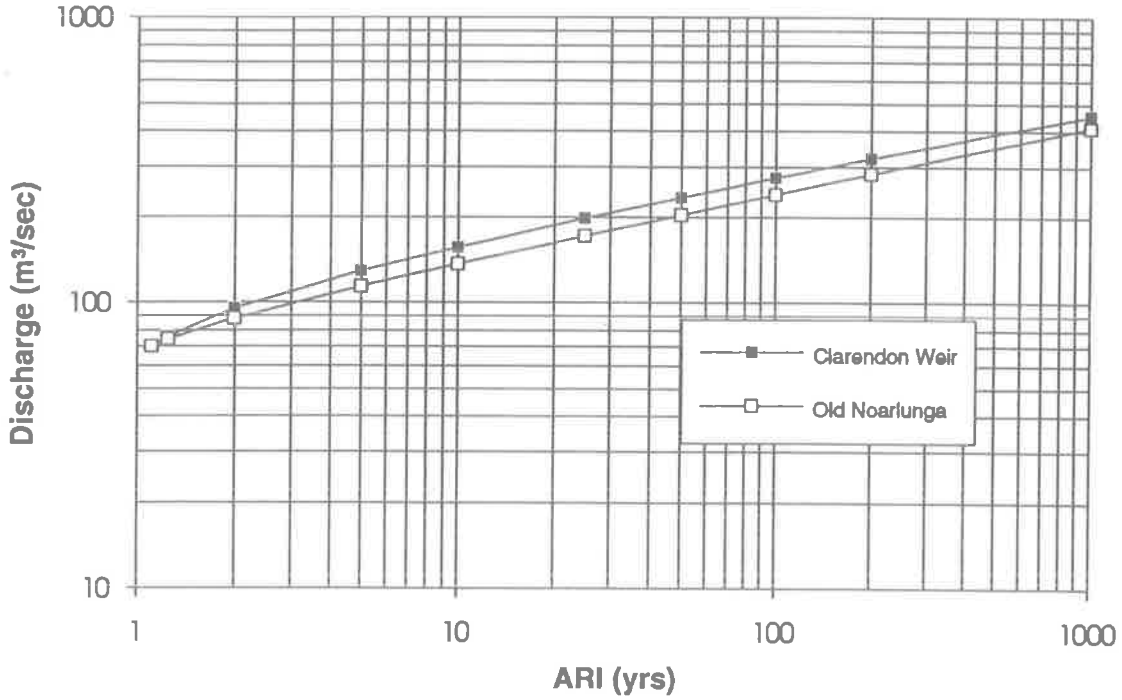


Figure 8.9 Comparison of Flood Frequency Curves for Clarendon Weir and Old Noarlunga for a log Pearson III distribution fitted to a Partial Series

A check was made to find out whether the short record at Old Noarlunga distorted the true distribution and consequent relationship with the flood frequency at Clarendon Weir. A partial series was extracted at Clarendon Weir for the same period as Old Noarlunga, and this was compared to the other calculated flood frequency curves using the full record in Figure 8.10.

It is clear that the calculated flood frequency curve at Clarendon Weir was not significantly affected by the shorter record.

The ratings for Clarendon Weir and Old Noarlunga were checked by examining the flow volumes. This proved to be inconclusive because there exists uncertainties in both ratings.

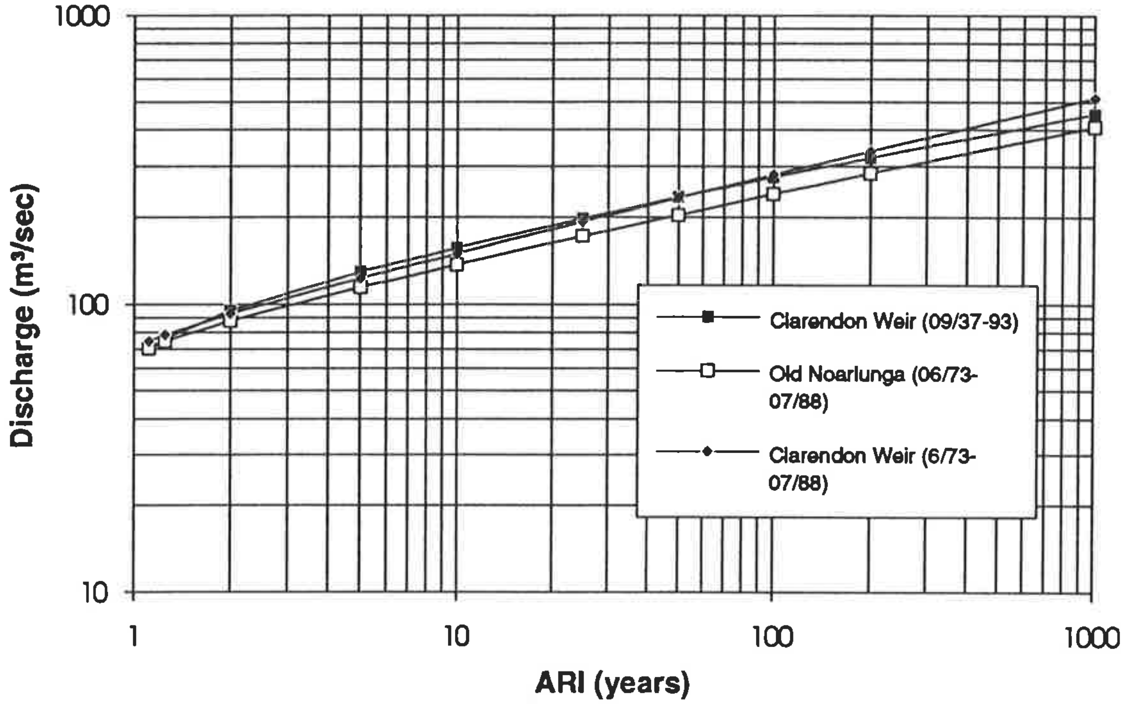


Figure 8.10 Comparison of Flood Frequency Curves for Clarendon Weir and Old Noarlunga for a log Pearson III distribution fitted to a Partial Series for a common period of record

8.4. Choice of Frequency Distribution for Flows Below Mt Bold Reservoir

8.4.1. Choice of Storage Probability Distribution

In order to rout the floods through the reservoir, a probability distribution of storages had to be chosen. It was decided to test four different storage probability distributions in order to see which one produced flows which were consistent with the log Pearson III distribution of floods at Clarendon Weir. The four storage probability distributions tested were:

1. The average storage distribution for the end of month storages for August, September, October and November. This period was chosen to represent the generally high storages in late winter and spring. Most of the events which were used to calibrate the RORB model occurred during this period.

2. The average storage distribution for the end of month storages for January, February, March and April. This period was chosen to observe the effect of a storage with a large drawdown in late summer and early autumn. The largest flow on record, in April 1889, occurred during this period.

3. The average storage distribution of end of month storages for the whole year. The choice of this distribution implies that the design inflows are equally likely to occur at any time throughout the year. This may slightly underestimate the flows downstream because large inflows are unlikely to occur during the middle of winter, while the storage is at its lowest, because the atmosphere is at its coolest and therefore able to hold less water vapour (Watson, 1993, pers. comm.).
4. The average storage distribution of end of month storages for December. This month was chosen as it was felt that it could possibly represent the best distribution of storages in order to reproduce the observed distribution of flows at Clarendon Weir.

The four different end of month storage probability distributions are shown in Figure 8.11.

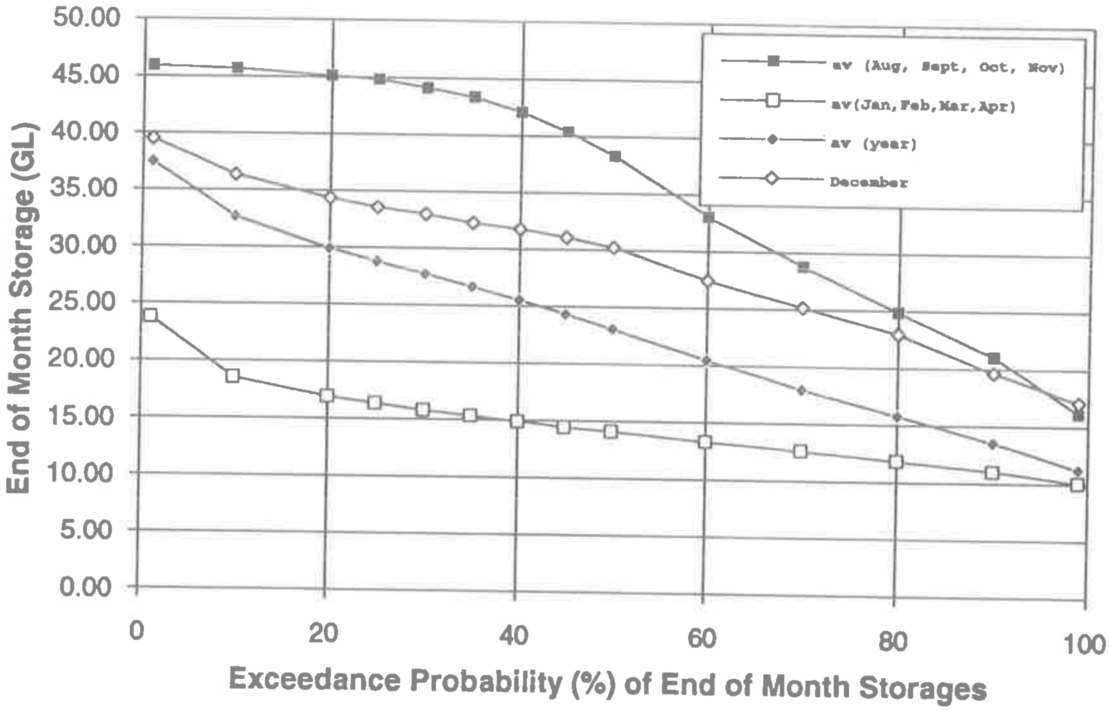


Figure 8.11 Four Different End of Month Storage Distributions

8.4.2. Construction of Joint Probability Diagram

The RORB models set up for the Onkaparinga Catchment to Old Noarlunga were re-run for a range of initial drawdowns. The calculated peak flows were recorded for the outflow from the reservoir and at Clarendon Weir.

The results of the RORB models were then plotted in a similar fashion to Figure 8.1. Lines of equal peak flows were then drawn. Figure 8.12 shows the joint probability of flows at Clarendon Weir for the average storage distribution for August, September, October and November. The joint probability relationship for each storage distribution is shown in Appendix J.

The area under the curves represent the probability of that peak flow occurring.

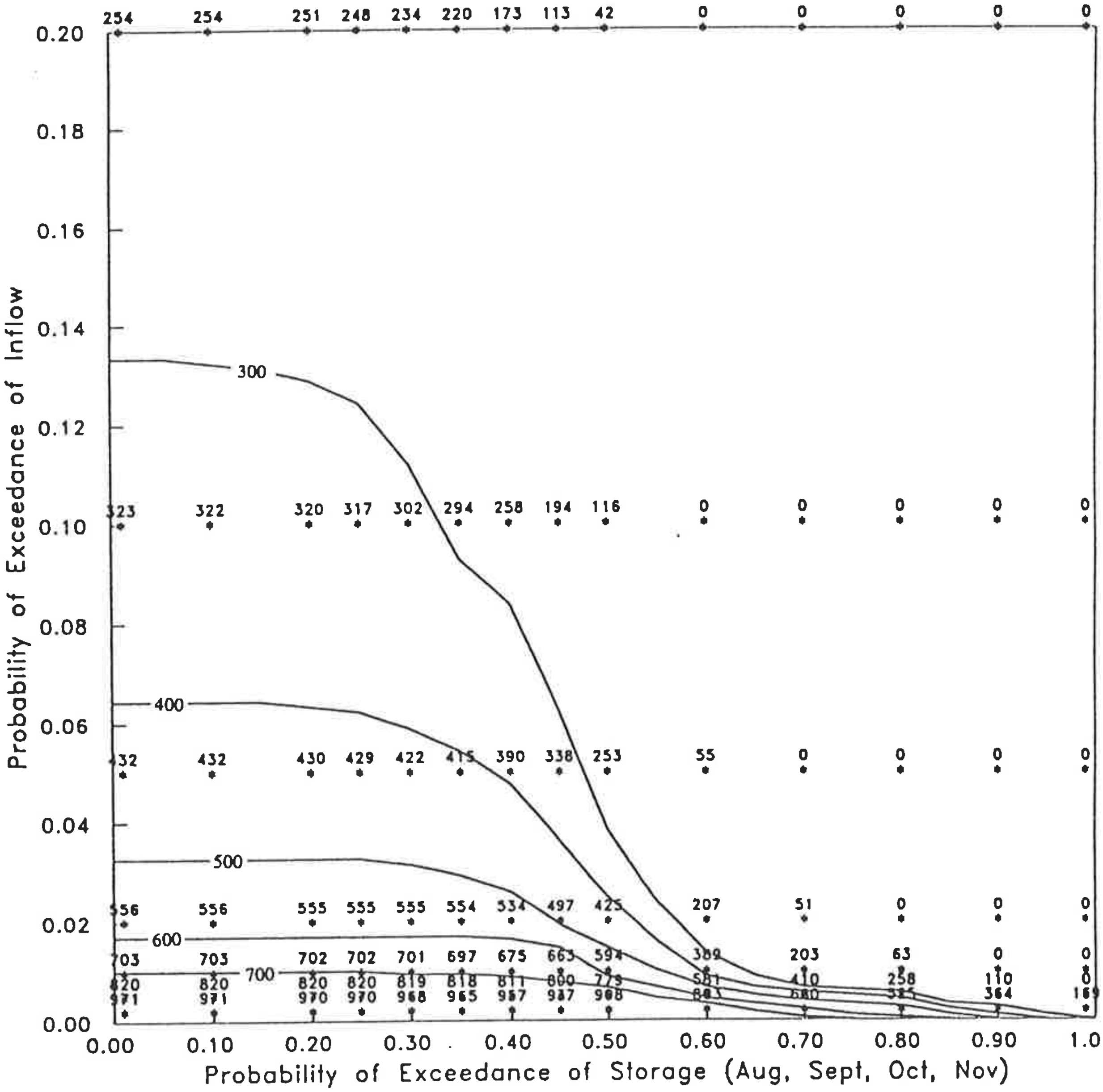


Figure 8.12 Joint Probability of Flows at Clarendon Weir for the Averages of the End of Month Storage Exceedance Probabilities for August, September, October and November

8.4.3. Choice of Frequency Distribution for Clarendon Weir

The calculated flood frequencies were compared to the log Pearson III distribution fitted to the partial series at Clarendon Weir.

Figure 8.13 shows the different frequency distributions at Clarendon Weir for each of the different storage probability distributions considered.

The flows calculated using the average end of month storage distribution for January, February, March and April were dramatically lower than those obtained using other distributions of storage. This confirmed the belief that floods in the Onkaparinga River below Mt Bold Reservoir are far more likely in early summer when the storage is near full supply level, than in late autumn when the storage is drawn down.

The flows calculated using the average end of month storage distribution for August, September, October and November produced flows that were above those obtained from the log Pearson III distribution at Clarendon Weir. The flows calculated from both the end of December storage distribution and the average yearly end of month storage distribution produced flows that were less than the log Pearson III distribution at Clarendon Weir for low ARIs, but greater than the log Pearson III distribution for larger ARIs.

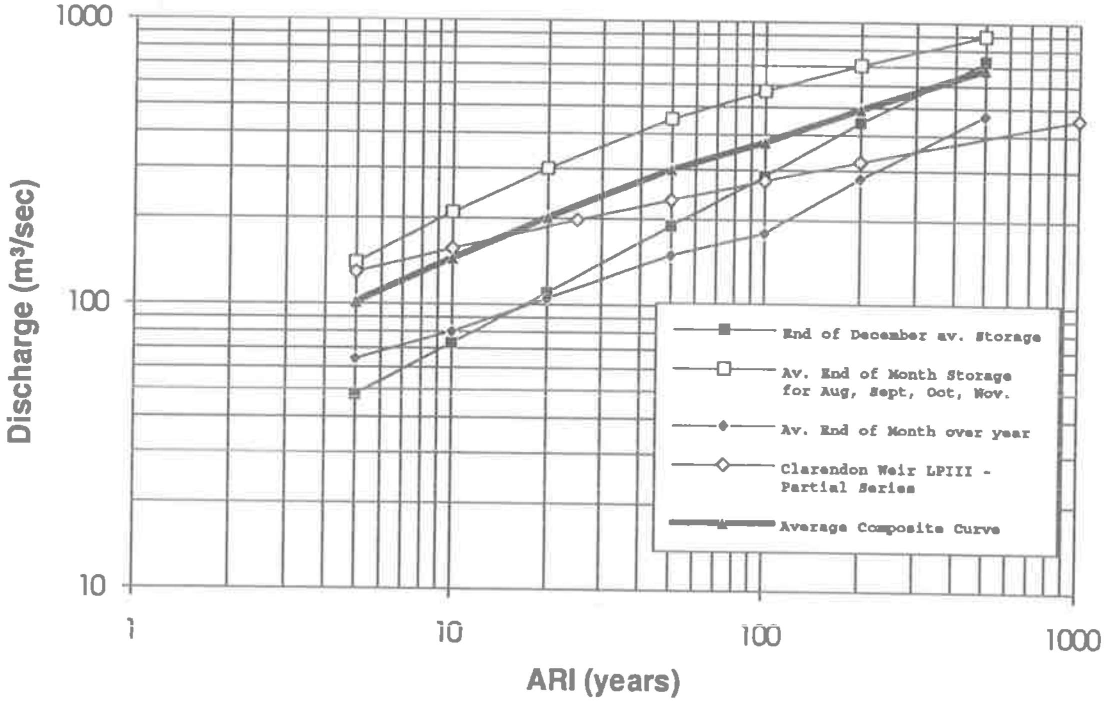


Figure 8.13 Different Frequency Distributions at Clarendon Weir

The chosen distribution was the average composite curve shown in Figure 8.13. It was the average of the flows calculated using the yearly end of month storage distribution and the flows calculated using the end of month storage distribution for August, September, October and November.

In Laurenson (1973) the average storage distribution over the entire year was chosen. From Figure 8.13 it is evident that this would have resulted in the flood frequency curve at Clarendon Weir being underestimated.

8.4.4. Choice of Frequency Distribution for Outflows from Mt Bold Reservoir

Figure 8.14 shows the outflows calculated for different storage distributions. The average composite curve was constructed as for Clarendon Weir.

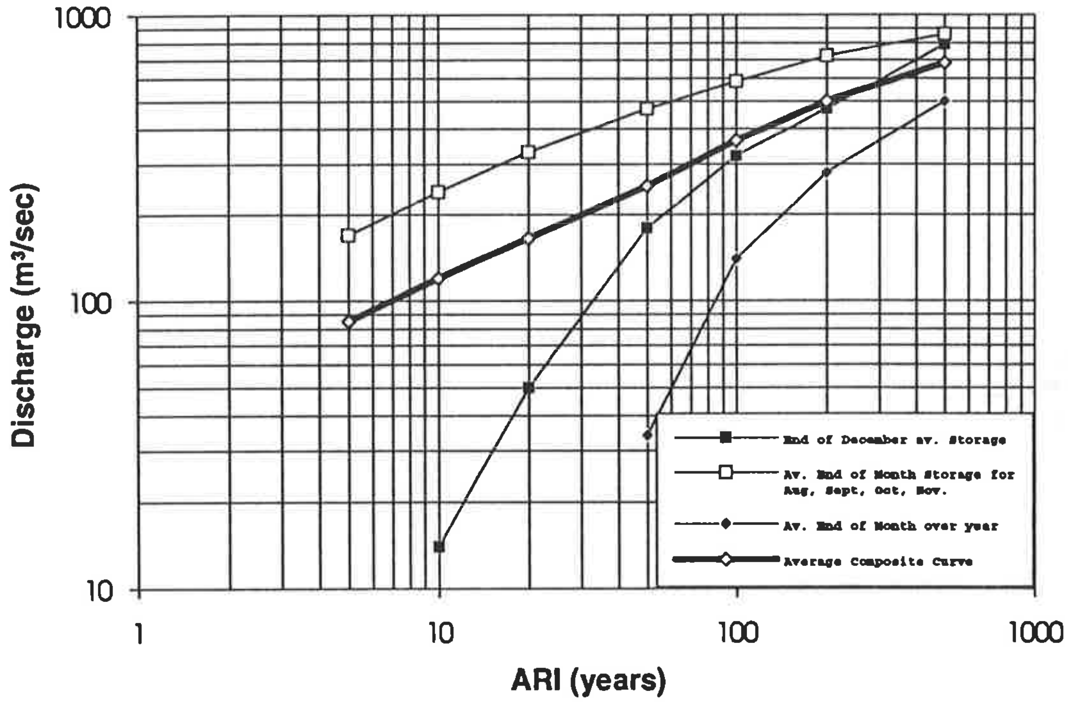


Figure 8.14 Different Frequency Distributions for Outflow from Mt Bold Reservoir

The outflow frequency curve shown in Figure 8.14 calculated using the average composite storage distribution is considered to be the most appropriate. Unless downstream gauging stations are installed which have accurate rating curves, it will not be possible to determine the outflow frequency with certainty.

Chapter 9

Probable Maximum Precipitation

9.1. Introduction

For the Onkaparinga Catchment, Bulletin 51 is applicable for PMP durations up to 3 hours. Estimates of PMP for longer durations should be obtained from the Bureau of Meteorology using the generalised south eastern Australia method (GSAM).

An approach was made to the Hydrology Branch of the Bureau of Meteorology Melbourne, inquiring about obtaining estimates of longer duration estimates of PMP using the GSAM. This branch was totally occupied with work from the Dam Safety Committees or Groups of NSW, Rural Water Corporation (Vic), Melbourne Water and the HEC from Tasmania. It would have been at least six months before a request could be fulfilled (Pearce, 1993, pers. comm.).

Because longer duration storms could not be obtained from the Bureau of Meteorology, Bulletin 51 was used to calculate PMP of durations from four to six hours. Longer duration PMPs that have been applied elsewhere in south eastern Australia were examined and this showed that the extrapolation from the shorter durations of Bulletin 51 seems to dominate in the formation of the curve for a range of catchment sizes. Estimates of longer duration PMP depths were therefore made from the extrapolation of the results from Bulletin 51.

9.2. Location of Isohyetal Pattern

The difficulty of defining procedures which produce a 'reasonable' maximum flood is illustrated by the guidelines included in IEAust (1987) for the location and orientation of the isohyetal pattern of the PMP. IEAust (1987) states that the areal pattern should be, *"centred over the catchment and not located in such a way as to give the greatest possible flood."*

IEAust (1987) proceeds to state that, *"The largest flood may result from location of the eye of the areal pattern close to the storage, even though this does not result in the greatest average depth of rainfall."* This indicates that it is indeed legitimate to orientate the areal pattern in order to maximise the PMF. In order to provide a reasonable degree of conservatism this would appear to be prudent.

It is the intention of the Bureau of Meteorology that the PMP isohyets derived from the generalised methods are centred over the catchments. Kennedy et al. (1988) stated regarding the spatial pattern included in the GSAM method,

> *"It is intended to be used without shifting the pattern, or the main rainfall centres, around the catchment, (in order to obtain the highest possible value of the PMF). This also applies to the design isohyetal pattern used with the method of adjusted United States data and those for use with generalised estimates of tropical storm PMP."*

This statement is in conflict with the early recommendations of Pilgrim (1986a) for use with the PMP derived using Bulletin 51, that the isohyetal map can be, *"located over the catchment to give the greatest average depth of rainfall"*.

9.2.1. US Practice

It is interesting to compare the recommendations of IEAust (1987) with the recommendations for the eastern United States (Ely & Peters, 1984; and Chow et al., 1988). The PMP estimate is converted to a probable maximum storm (PMS). The PMS has four variables:

1. location of the centre of the storm;
2. storm-area size;
3. storm orientation; and
4. temporal arrangement of precipitation amounts.

The four variables are chosen to produce the maximum peak discharge or runoff volume at the point of interest. A trial and error procedure is recommended to determine the critical values of the four variables. This procedure for locating and orienting the PMP differs dramatically with that contained in IEAust (1987).

An earlier paper (Riedel, 1977) stated that the National Weather Service used an elliptically shaped rainfall pattern centred on the drainage. Great attention was paid to the possibility and magnitude of an antecedent storm. A storm 30 percent of the PMP was assumed to occur several days prior to the PMP.

9.2.2. Adopted Philosophy of Locating the Isohyetal Pattern

For this study, a number of different locations of the isohyetal pattern were tested. This was done to test the sensitivity of the calculated peak flow to the isohyetal location (Daniell, 1987).

The locations which were examined are shown in Figure 9.1. These were all considered to be located approximately centrally in the catchment. Location 2 was excluded from the analysis because location 4 proved to be a better estimate of the isohyetal pattern located towards the reservoir.

9.3. Choice of Losses

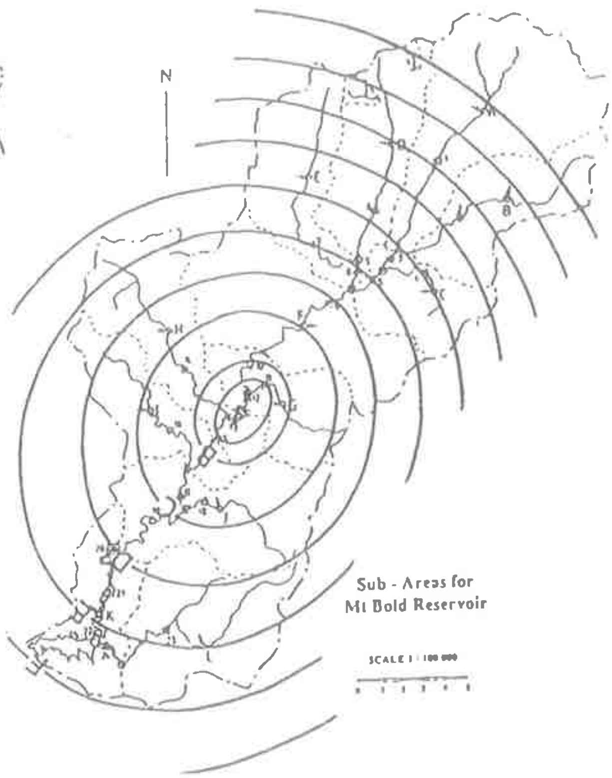
In small flood design, the losses are chosen to ensure that the calculated peak flows have the same probability of occurrence as the design rainfalls which produced them. The choice of losses for extreme flood estimation are not based on this premise.

The losses to be used in conjunction with the PMP are to be based on observed losses. IEAust (1987) recommends that the losses for the estimation of the PMF should be equal to or possibly a little less than the minimum value in large floods observed on the catchment.

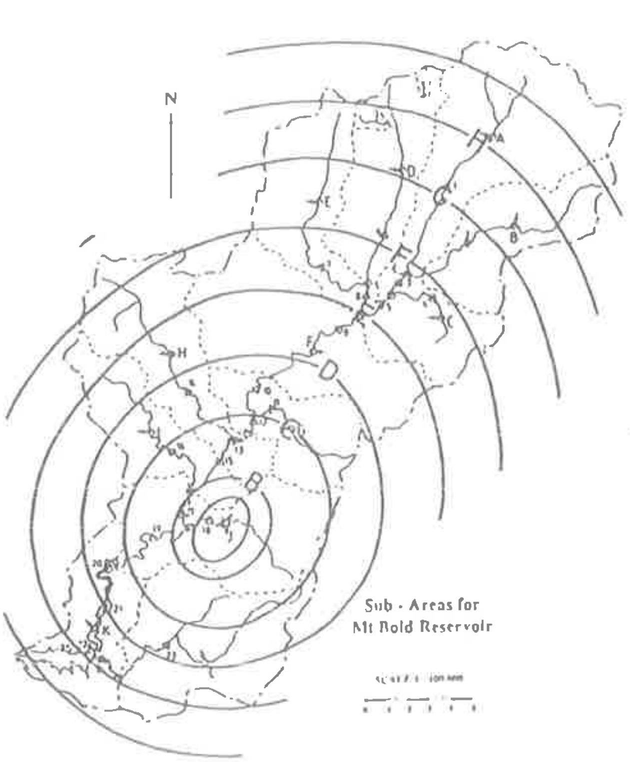
For short duration PMPs, the losses are generally very small when compared to the rainfall intensities, and therefore the estimation of losses is not considered to be critical. IEAust (1987) suggests that zero losses may be appropriate in this case. For long duration PMPs, the losses can reduce the estimated PMF by a larger amount (Brown, 1982).

It is interesting to note that for a 5 hour duration PMP a 2 mm/hr continuing loss only represents a total of 10 mm over the storm duration. This is seen as being relatively insignificant because generally PMP depths are rounded to the nearest 10 or 20 mm.

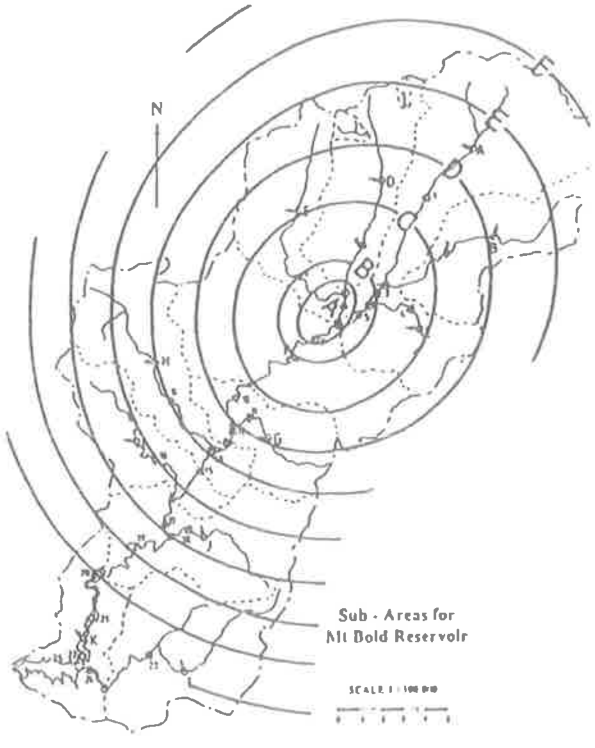
Mount Bold Catchment Location 1



Mount Bold Catchment Location 3



Mount Bold Catchment Location 4



Mount Bold Catchment Location 5

Figure 9.1 Locations of PMP Isohyetal Patterns

The matter of antecedent rainfall is considered to be very important in the United States. In Australia the occurrence of antecedent rainfall is not usually analysed directly but is inferred by the choice of zero initial loss. Brown (1982) analysed the effect of antecedent rainfall on the estimate of the PMF for a number of case studies.

9.3.1. The Effect of Changes in Land Use

Changes in the land use of a catchment can have a dramatic effect on the hydrological response of the catchment. Clearing of the natural vegetation from a catchment will increase the runoff because of the reduced losses. The likelihood of bushfires and agricultural clearing is therefore an important consideration in the estimation of model parameters for the estimation of the PMF (Brown, 1988).

Brown (1982) considered the possibility of a bushfire followed by a PMP. This paper gave two examples of PMF estimates which were adjusted for the possibility of bushfires. Brown (1982) noted that the joint probability of the catchment denuded by a fire and the occurrence of a PMP should be estimated.

In keeping with the desire to obtain a reasonable estimate of the PMF it is not considered necessary to consider the possibility of a bushfire followed closely by a PMP.

9.4. Temporal Pattern

It is generally believed that the temporal patterns of extreme storms are more uniform than those of more frequent storms (Pilgrim, 1986a). The choice of temporal pattern is very important and can have a significant effect on the magnitude of the PMF.

Bulletin 51 contains a recommended temporal pattern for short duration PMP estimates. This is based upon the temporal patterns recorded during the Woden Valley storm of 1971 and the Melbourne storm of 1972. Both Wood and Alvarez (1982) and Brown (1982) recommended that the temporal patterns be rearranged to maximise the surface runoff. The recommendation to rearrange the temporal patterns is in line with the recommendations in the United States and is based on the large differences in the calculated PMF that can occur.

Brown (1982) recommended that the effect of rearrangement be taken into account,

> *"It is recommended that the effect of varying patterns be examined and the feasibility of occurrence of the most severe patterns be studied in conjunction with observed temporal patterns and then discussed with meteorologists. A subjective decision must finally be made but this should be made with a background knowledge of the effects of different alternatives in terms of their reality and effect on costs."*

Wood and Alvarez (1982) described the policy of rearrangement of temporal patterns which was used to determine PMFs by the Water Resources Commission in New South Wales. The temporal patterns were completely rearranged to give "*the most critical condition of runoff*". This procedure was adopted despite a recommendation by the Bureau of Meteorology that the design temporal patterns should not be rearranged. The Bureau of Meteorology made three recommendations (Wood and Alvarez, 1982):

> *"1. temporal patterns for design storms should not be modified indiscriminately;*
>
> *2. any modifications should only be carried out after a thorough analysis of meteorological factors;*
>
> *3. the modified pattern must be consistent with that of at least one observed major storm."*

These recommendations were noted by Wood and Alvarez (1982) but the temporal patterns were still modified because of the practice in the US and the catastrophic effect of a failure of a dam. This reflects the desire to determine an estimate of the PMF which is an absolute maximum.

The rearrangement of the temporal pattern is not considered to yield a reasonable estimate of the PMF.

> *"Rearrangement of patterns to give the maximum possible flood peak is at variance with the design objective determining a limiting value to floods that could reasonably occur."* (Nathan, 1992)

Nathan (1992) described the development of the design temporal patterns to be used as part of the GSAM. The average variability method was used to derive the temporal patterns from a large database of extreme storms. These temporal patterns were then smoothed in keeping with the belief that the temporal patterns of extreme storms are more uniform than for more frequent storms. The aim of the procedure is to develop temporal patterns that, "*satisfy the combined requirements of engineering conservatism and notions of physical reality*" (Nathan, 1992).

9.4.1. Adopted Temporal Pattern

The design temporal pattern appropriate for all durations up to 6 hours is shown in Figure 9.2 (Bulletin 51). Although it is based on storms from the ACT and Melbourne, it was considered to be suitable for use in this catchment. The second temporal pattern Figure 9.2 is a typical 24 hour temporal pattern used in the GSAM.

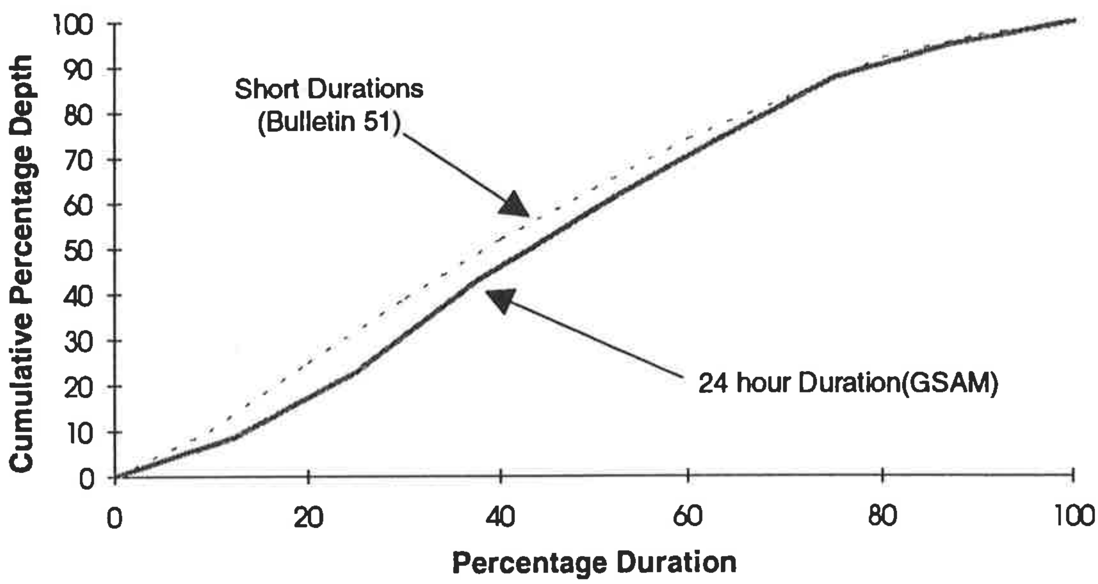


Figure 9.2 Temporal Patterns for PMPs

9.5. Application of Bulletin 51

The procedures used in Bulletin 51 derive standard rainfall depths from a set of depth-duration-area curves for the area of the catchment. These depths are then adjusted by various factors reflecting the individual characteristics of the catchment: geographical location, moisture potential, elevation and topography.

The depth-duration-area curves are adjusted by the ratio of the highest atmospheric moisture content possible in the catchment to that at 28°C. The values of the percent reduction is obtained from Figure 3 of Bulletin 51. From this figure, a value of 0.64 was adopted for the Onkaparinga Catchment. This compares with the value of 0.63 that was used by the Bureau for their earlier estimation of PMP for the Onkaparinga Catchment (BoM, 1984).

Recorded storms indicate that storms of less than 1 hour duration are not influenced by the underlying topography (Pierrehumbert and Kennedy, 1982). For storm durations greater than one hour, Bulletin 51 distinguishes between smooth and rough terrain and two separate sets of rainfall depth/duration/area curves are given. This is because rough terrain can trigger thunderstorms and can hold them in place (Bulletin 51).

The rough terrain is defined as catchments where elevation changes of 50 metres or more within 400 metres are common. Appendix K includes a map of the Onkaparinga Catchment showing the slopes. It can be seen that a significant part of the catchment has a slope greater than 12.5 percent. The catchment was therefore considered to be rough.

Earlier PMP estimation procedures included an adjustment for elevation. Bulletin 51 includes no adjustment for elevations up to 1,500 metres and a reduction in PMP of 5 percent per 300 metres for elevations above 1,500 metres (Pierrehumbert and Kennedy, 1982). No reduction for elevation was necessary as none of the catchment is above 1,500 metres.

The PMP estimates derived using Bulletin 51 are assumed to occur in summer or early autumn. This is because extreme rainfalls are more likely during the warmer months when the atmosphere is able to hold more moisture vapour. It is noted in Bulletin 51 that this period coincides with the period of greatest rainfall losses as the soil is drier than in winter. PMP estimates for other times of the year are therefore included in Bulletin 51. The limiting area for winter PMP estimates is 500 km².

No allowance for seasonal variations was made. This is because the storage level of Mt Bold Reservoir is at its highest level during late spring and summer and the choice of losses is insignificant when compared to the depth of rainfall (Section 9.3).

9.6. Preliminary PMP Results

The calculations required to determine the PMP depths for the various subareas for any one location can take a significant amount of time. The procedures adopted were based on those outlined in Bulletin 51, Department of Territories (1985) and in a subsequent PMP report, BoM (1991).

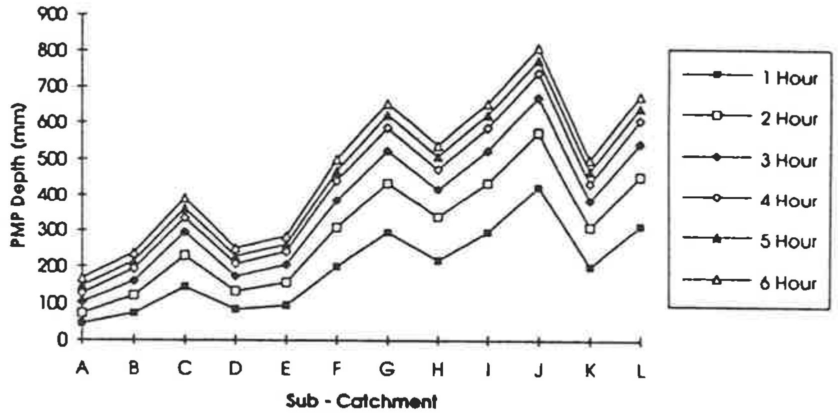
The estimation of the PMP involved two steps. In order to obtain a relatively quick estimate of the PMP, the average rainfall for each subcatchment was initially estimated by eye from the isohyetal pattern. The PMPs for different locations were then routed through the catchment using the RORB model to produce outflow PMFs (Section 10.4). Once the critical locations were determined, a full analysis of the rainfall depth on each subcatchment was made using a planimeter. The preliminary and final PMP depths were then compared.

The calculation of PMPs was performed using a spreadsheet. The preliminary PMP values are summarised in Appendix K for each duration and location.

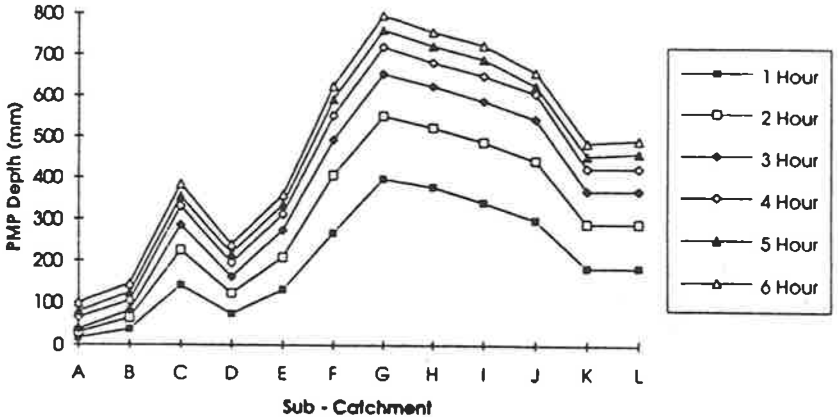
The PMP depths have been plotted on Figure 9.3 to show the relationship between the durations, subcatchments and locations. From this figure it is possible to quickly determine whether there are any large errors in the estimation of PMP depth.

PMP Depths for Mount Bold Catchment Location 1 - Centre of Catchment (100 % rough)

PMP Depths for Mount Bold Catchment Location 4 - Towards Reservoir (100 % rough)



PMP Depths for Mount Bold Catchment Location 3 - Reverse Centre of Catchment (100 % rough)



PMP Depths for Mount Bold Catchment Location 5 - High in Catchment (100 % rough)

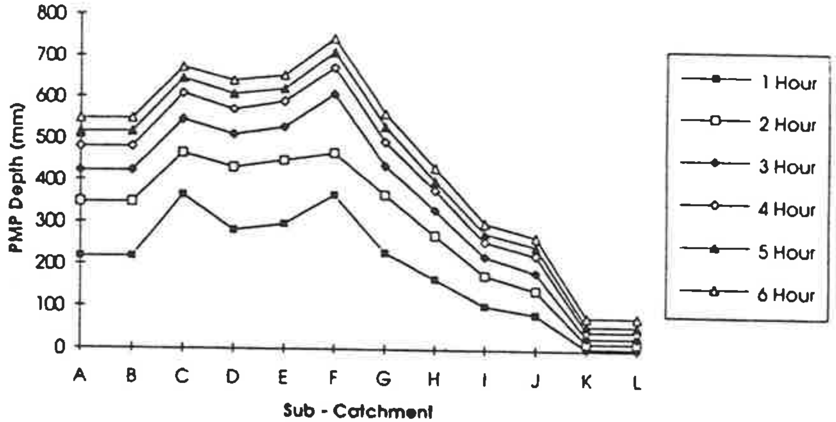


Figure 9.3 PMP Depths for Various Subcatchments and Durations

9.6.1. PMP for Durations Greater than 3 Hours

For durations greater than 3 hours, PMP estimates should be calculated using the GSAM (Pearce et al., 1993; Kennedy, et al., 1988). Unfortunately the undertaking of a GSAM by the Bureau of Meteorology was beyond the scope of the research in terms of both cost and time constraints (Section 9.1).

It is recommended that a GSAM be undertaken by the Bureau of Meteorology for the Onkaparinga Catchment. If further research is to continue into other South Australian catchments it is recommended that a GSAM be undertaken.

In order to estimate longer duration PMP depths, PMP estimates from other catchments in south eastern Australia were examined. Figure 9.4 shows a compilation of the PMP estimates in south eastern Australia for a number of catchments that have estimates calculated using the GSAM.

The PMP estimates from other catchments suggest that the short duration PMP estimates obtained from Bulletin 51 can be extrapolated and provide useful estimates of the longer duration PMP for the Onkaparinga Catchment.

It is recommended that the PMF be recalculated once the GSAM is completed. It is not expected that this will result in substantially different results because the critical duration of the PMP is approximately 3 to 4 hours (Section 10.4).

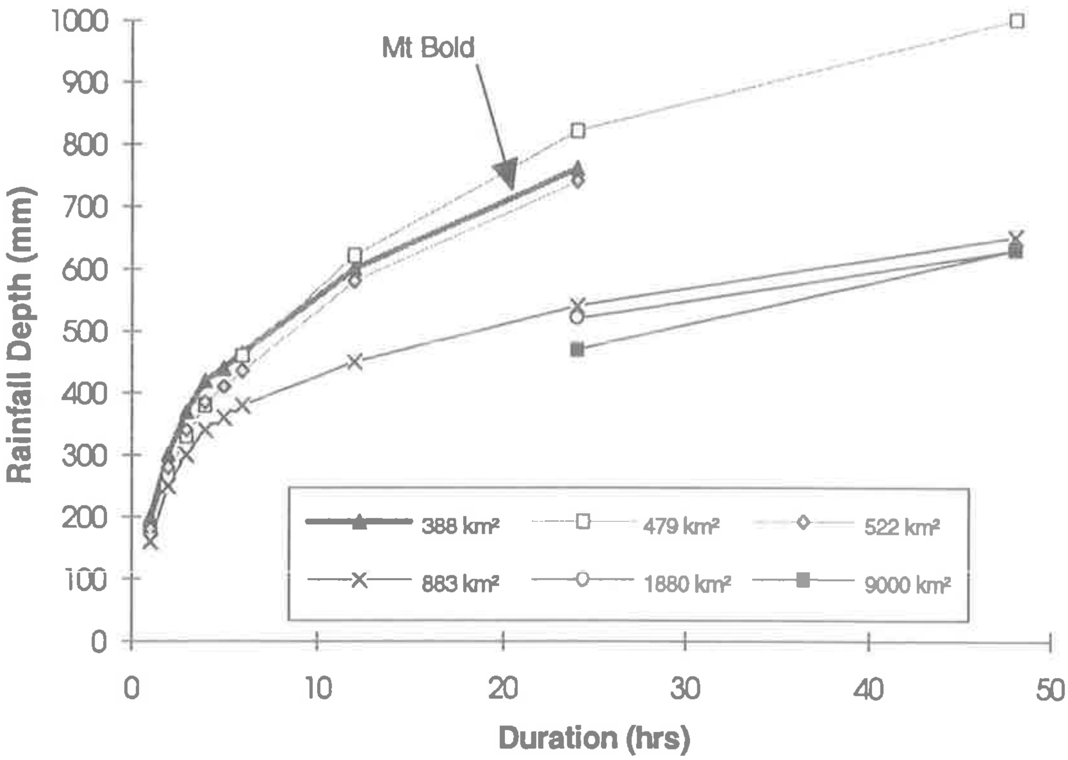


Figure 9.4 PMP Estimates for South-Eastern Australia

A number of estimates for the extension of the PMP were tested for analysis of the PMF. These are shown in Figure 9.4. The adopted PMP estimates for the longer durations are slightly below those shown from other catchments. This is reasonable as the other estimates are from catchments which are located in regions that have a greater chance of tropical storms occurring.

The temporal pattern from the GSAM was used for the longer duration PMPs. This temporal pattern has been derived using the average variability method of Pilgrim, et al. (1969) and smoothed for consistency using the method of Nathan (1992).

In BoM (1991) it is recommended that for the 12 hour duration PMP, a 24 hour temporal pattern and the Bulletin 51 temporal pattern should both be used with the resulting PMFs being averaged. The two temporal patterns used are shown in Figure 9.2.

9.7. Full PMP Analysis

From the routing of the preliminary PMP estimates (Section 10.4), it was evident that the critical PMP was location 3 with a duration of approximately 4 hours.

A full PMP analysis, as outlined in Bulletin 51, was performed for this storm event. The area and hence the rainfall on each subcatchment was determined from the isohyets using a planimeter. The results of the full PMP analysis are summarised in Table 9.1 for durations of 3, 4 and 5 hours.

Table 9.1 Full PMP Analysis for 3, 4 and 5 hour Duration at Location 3

Total Precipitation in mm

Subcatchment	Area (km^2)	3 hour	4 hour	5 hour
A	51.93	42	52	60
B	27.83	86	106	122
C	19.56	304	350	377
D	30.85	145	174	194
E	29.34	236	274	299
F	46.89	504	567	605
G	22.18	692	773	813
H	35.09	533	601	637
I	23.19	574	642	681
J	38.01	569	636	676
K	23.09	377	429	461
L	40.13	395	449	482

The difference between the final PMP and the preliminary PMP analysis are shown in Table 9.2. Although there are significant differences in the estimation of rainfall depth for individual subcatchments, the PMF calculated using the final estimate of PMP was very similar to the PMF calculated using the preliminary estimate of PMP (Table 10.2). This tends to validate the procedure of the preliminary estimation of the PMP depths.

Table 9.2 Differences between Preliminary and Final PMPs for Location 3

		3 hour			4 hour		
Sub catchment	Area (km^2)	Prelim (mm)	Full (mm)	Difference (%)	Prelim (mm)	Full (mm)	Difference (%)
A	51.93	37	42	-11.9	65	52	25.0
B	27.83	81	86	-5.8	103	106	-2.8
C	19.56	285	304	-6.3	330	350	-5.7
D	30.85	161	145	11.0	194	174	11.5
E	29.34	273	236	15.7	310	274	13.1
F	46.89	490	504	-2.8	550	567	-3.0
G	22.18	651	692	-5.9	718	773	-7.1
H	35.09	620	533	16.3	679	601	13.0
I	23.19	583	574	1.6	647	642	0.8
J	38.01	540	569	-5.1	601	636	-5.5
K	23.09	366	377	-2.9	420	429	-2.1
L	40.13	366	395	-7.3	420	449	-6.5

9.8. PMP for Catchment to Old Noarlunga

The PMPs for the catchment to Old Noarlunga were established by using the locations that were deemed to be the critical locations of the isohyetal pattern for the Mt Bold Catchment. The isohyetal patterns for location 3 and 4 are shown in Figure 9.5 for the catchment to Old Noarlunga.

The PMPs were then calculated for the larger area. This resulted in a lower total depth for each of the various durations examined. The distributed rainfall depths over the whole catchment were calculated using the preliminary method as results obtained from analysis of the Mt Bold Catchment (Section 10.5) showed that there was little effect in using the accurate analysis when it came to calculating the PMF.

The results of the analysis are shown in Appendix K for Locations 3 and 4.

The temporal patterns that were established for the various durations for the Mt Bold Catchment were deemed to be suitable for the Old Noarlunga Catchment.

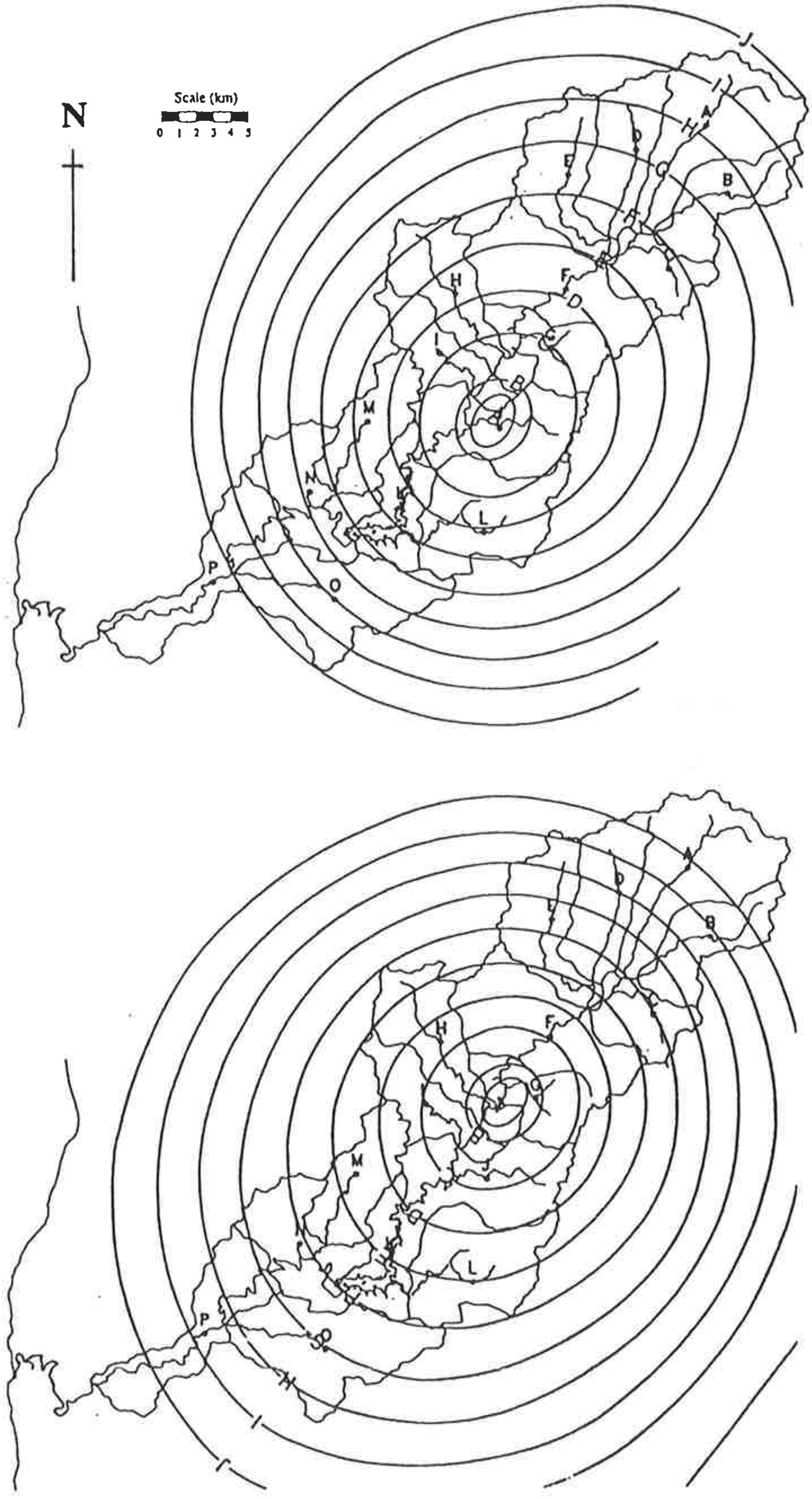


Figure 9.5 PMP Isohyetal Pattern Location for Old Noarlunga Catchment

9.9. Summary of PMP Results

The PMP depths for the Mt Bold Catchment for the various durations are shown in Figure 9.4. PMP estimates for longer durations were extrapolated figures matched to the shorter durations from Bulletin 51.

These PMPs were distributed according to the spatial distribution recommended in Bulletin 51 in order to determine preliminary results for the various locations of the PMP isohyetal pattern. These PMPs were then routed as described in Chapter 10 to determine the critical location of the isohyetal pattern. A full analysis of the PMP was then undertaken for the critical location.

The preliminary PMPs for the Onkaparinga Catchment to Old Noarlunga were estimated in a similar fashion but included the accurate analysis results of the Mt Bold analysis. Because the PMF results from the Mt Bold Catchment indicated that the preliminary PMP estimates produced similar values of PMF as the full PMP analysis, a full PMP analysis was not done for the four subcatchments downstream of Mt Bold Reservoir.

Chapter 10

Probable Maximum Flood

10.1. Introduction

The concept of the probable maximum flood (PMF) is discussed in Section 2.3. Guidelines for calculating the PMF are included in ANCOLD (1986) and IEAust (1987). The intention of these two guidelines is that the PMF should represent a 'reasonable' estimate of the maximum flood.

It is therefore important that the modelling is undertaken to ensure a reasonable estimate of the PMF. The magnitude of the resultant PMF is affected by the adopted spillway rating curve, the chosen model parameters and the choice of PMP.

The derivation of the PMP is discussed in Chapter 9. The PMP was converted to a rainfall excess by the subtraction of losses and routed to produce both inflow and outflow hydrographs for Mt Bold Reservoir using the RORB package described in Chapter 6.

10.2. Extrapolation of the Spillway Rating Curve

10.2.1. Introduction

From preliminary estimates of design flows it became evident that the spillway capacity will be exceeded by floods of low annual exceedance probabilities. In order to estimate the outflow PMF it was necessary to extend relationship between flow and gauge height.

The gauge height can be converted to Australian Height Datum using Equation 10.1 for the Mt Bold location.

$$EL(m) = GH + 205.5 \tag{10.1}$$

Figure 10.1 shows a diagrammatical cross section of the top portion of the dam wall indicating the relevant elevations and corresponding gauge heights for the case where all gates are open 3.8 metres.

10.2.2. Maximum Gate Opening

The capacity of the gated spillway is limited by the maximum gate settings. From a field trip by the author in August 1992, it was found that the maximum gate openings differed greatly from those in the operator's notes based on a test conducted in November 1985 (Section 4.2.2.2). The maximum gate openings need to be reset to allow the gates to open to at least 3.8 metres. This has important repercussions on the calculated rating. For a gauge height of 42.92 metres (dam crest), the discharge over the spillway for all gates open to their current maximum settings, as shown in Table 4.1, is approximately 950 m^3/sec. If the gates are reset to allow them to open to 3.8 metres, this would be increased to 1,170 m^3/sec.

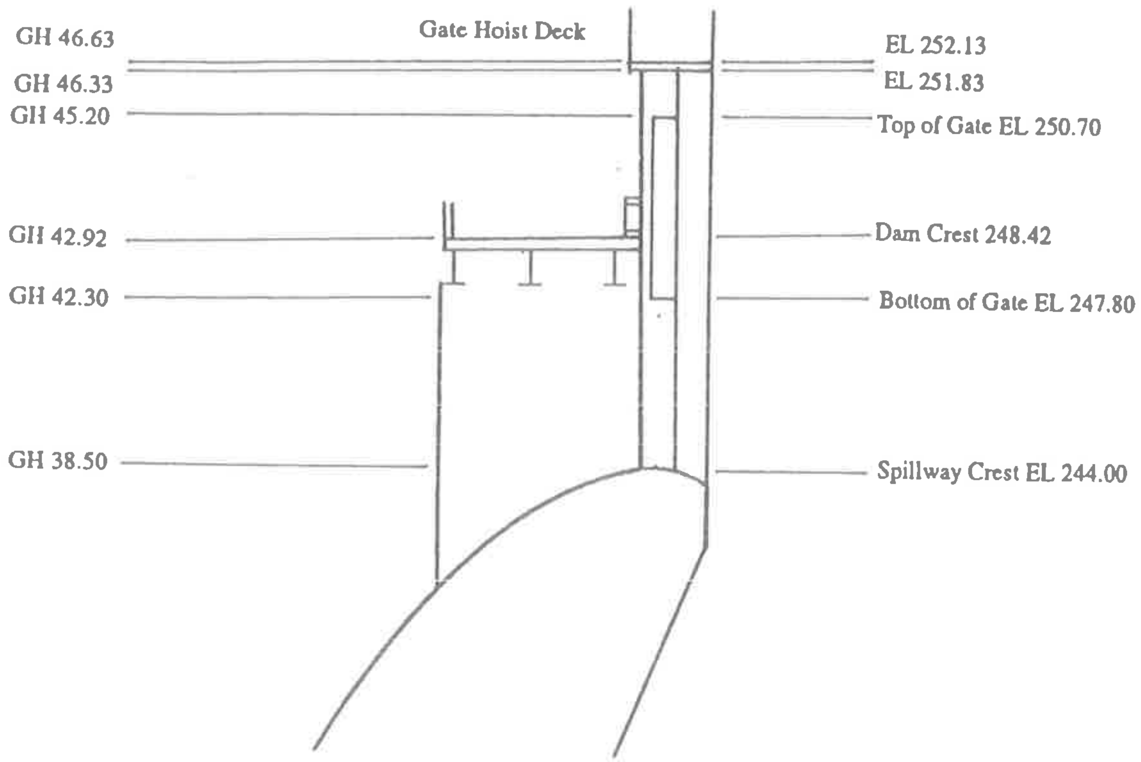


Figure 10.1 Cross Section of Mt Bold Dam showing Elevations

10.2.3. Components of the Rating Curve for Mt Bold Dam

In order to calculate the rating curve to rout the PMF through the reservoir it was assumed that all of the gates were opened to 3.8 metres. This meant that the rating curve did not depend on gate operations but rather only on the reservoir level. It was also assumed that the bridge hoist deck structure remained intact for all flood levels.

ANCOLD (1986) recommends that for routing floods through a gated spillway that consideration be given to malfunction of some of the gates. This could be a result of operator error or equipment failure. There is also the possibility that one or more of the gates becoming partially blocked by debris. Because of the large uncertainties associated with the determination of the rating curve for routing of the PMF, the consideration of gate failure was not considered to be warranted.

The different component flows of the extended rating curve are listed below:

Spillway Flow

Assuming the gates are initially raised, water spills for a gauge height greater than 38.50 metres. The rating curve for the spillway was calculated using the equations proposed in Hager and Bremen (1988). These equations are discussed in Chapter 4. Initially the flow was calculated as flow over an ungated spillway. As the head increases the gates become submerged and the flow is controlled by the gated spillway.

As discussed in Section 4.3.1.3, the design head for Mt Bold Reservoir's gated spillway has been estimated to be 3.8 metres. This means that the maximum head which can be used in the equations proposed by Hager and Bremen is 7.6 metres, which corresponds to a gauge height of 46.1 metres. The equations were used up to a gauge height of 51 metres. Although this exceeds the limit imposed by Hager and Bremen (1988), these sets of equations represent the best method of calculating the rating.

Spill over the Dam Crest

When the reservoir level reaches a gauge height of 42.92 metres, water spills over the dam crest. The length of the dam crest from the abutment to the gate structure on the north side is 58.52 metres and on the south side is 95.61 metres (E&WS drawing number E59 388). The flow is assumed to act as a broad crested weir. The discharge was calculated using Equation 10.2.

$$Q = 1.5 b_{ef} H^{1.5} \tag{10.2}$$

where: Q is the discharge (m³/sec);

b_{ef} is the effective width (m); and
H is the head (m).

Spill over the Gates and Below the Gate Hoist Bridge

When the reservoir level reaches a gauge height of 45.20 metres, the flow passes over the raised gates. This has been approximated as flow over a broad crested weir. When the reservoir level is greater than 46.63 metres, the flow over the gates was considered as orifice flow. The rectangular orifice has a width of 58.52 metres and a height of 1.13 metres, when the gates are open 3.8 metres.

Spill over the Gate Hoist Bridge

Once the reservoir level reaches 46.63 metres the water will flow over the gate hoist bridge. The rating was calculated as a broad crested weir, where the effective width was assumed to be 73 metres.

Spill over the Saddle

Analysis of the topography revealed that at a gauge height of 43.8 metres (EL 249.3m), water begins to flow through a saddle located approximately 650 metres north-west of the dam wall. The lowest point of the saddle is only 880 mm above the dam crest and 5.3 metres above the spillway crest. Unfortunately there is no accurate survey of the area. An approximate cross section of the saddle was determined from E&WS drawing number E59 399, *Mt Bold Dam - Raising Additions to Clay Blanket Area*. This drawing shows the installation of a clay blanket and includes a contour plan of the saddle region. Additional information was obtained from a 1:10,000 map.

In order to determine an approximate rating of the saddle, it was assumed that the saddle was cleared of all vegetation and re-contoured. The adopted cross section is shown in Figure 10.2.

The approximate rating for the saddle was calculated using Equation 10.2, which applies for a broad crested weir. The width of the saddle varies from 55 metres to a maximum of 210 metres (Figure 10.2). If the saddle is to be used as an auxiliary spillway it is important that an accurate survey of the area be undertaken and a revised rating be established.

The use of the saddle area as an auxiliary spillway has some ramifications which should be addressed. The current access road to the reservoir passes across the saddle and this may need to be realigned. Water flowing over the saddle may also impinge on some of the existing buildings.

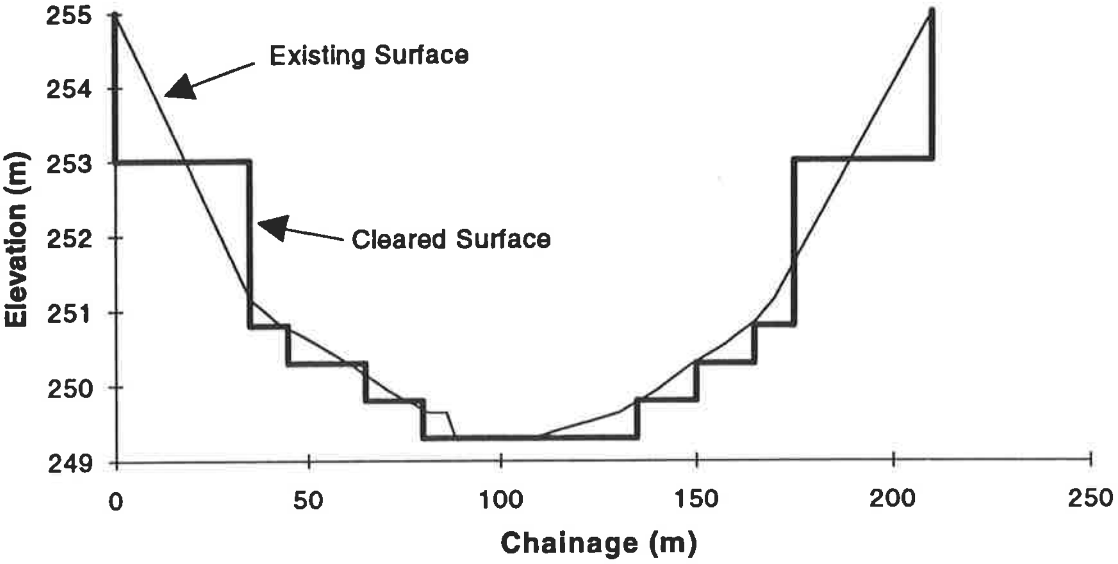


Figure 10.2 Elevation of Saddle Showing Assumed Cleared Cross Section

10.2.4. Composite Rating Curve for Mt Bold Dam

The components of the rating curve were combined to produce a composite rating for the spillway and this is shown in Figure 10.3. It is important to note that for high gauge heights the discharge over the saddle is a significant portion of the total discharge.

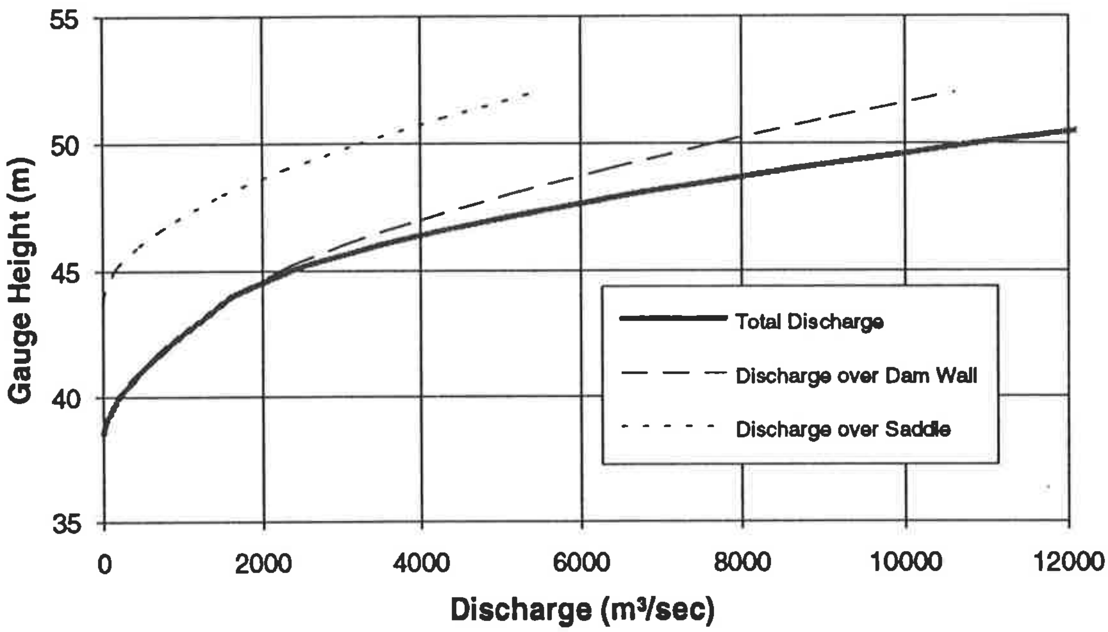


Figure 10.3 Extended Rating Curve for Mt Bold Dam

10.3. Model Parameters

10.3.1. Introduction

A unit hydrograph or runoff routing model can be used to rout the rainfall excess. It is stated in IEAust (1987) that because of non-linearity, "*the use of an average unit hydrograph probably would lead to an underestimate of the PMF*".

Following the discussion in Section 6.2, the RORB model was chosen. The RORB runoff routing model is described in Chapter 6.

10.3.2. Model Non-Linearity

The choice of runoff routing model used to rout the excess rainfall to produce a PMF is very important. Most Australian runoff routing programs have a relationship between the temporary storage and the discharge of the form of Equation 10.3.

$$S \propto kQ^m \tag{10.3}$$

where: S is the storage;
Q is the discharge; and
k and m are constants.

The model is considered to be linear if the exponent m is one. Non-linear runoff routing models use a different value of m which is usually less than one. The use of non-linear runoff routing models has contributed to increases in the estimates of PMF.

The choice of the degree of linearity is very important. Daniell (1987) concluded that the choice of model linearity affected the estimated PMF by up to 30 percent. This was confirmed in Section 10.6.4.

Another factor to be considered when comparing PMF estimates derived using unit hydrographs and runoff routing models is that uniform rainfall is generally used with the unit hydrograph whereas spatially varied rainfall is used with the runoff routing models (Daniell, 1987). The use of spatially varied rainfall also increases the estimate of the PMF.

IEAust (1987) makes the following recommendation regarding the choice of model linearity;

> *"Where most of the valleys in the catchment are V-shaped with only small flood plains:*
> - *if the value of m is found by calibration and is > 0.8, adopt this value;*
> - *if the peak of the calibration flood is > 0.4PMF, and the calibrated m is less than 0.8, adopt the calibrated value;*
> - *otherwise use m= 0.8".*

From the calibration undertaken in Chapter 6, the preferred value of *m* was 0.7. The largest calibrated event had a peak flow of 432 m³/sec. From initial estimates of the PMF, it was obvious that this flow was significantly less than 40 percent of the PMF. Following the recommendations of IEAust (1987), a value of *m* of 0.8 was adopted for routing the PMF.

The sensitivity of the PMF to the choice of model non-linearity is discussed in Section 10.6.4.

10.3.3. Adopted Model Parameters

The calibration of the model is discussed in Section 6.4. From the discussion included in Section 10.3.3, a value of *m* of 0.8 was adopted. For this value of *m* the best estimate of k_c was 16.3 (Section 6.4).

The choice of losses is discussed in Section 9.3. The parameters chosen for analysis of the PMF for the Mt Bold Catchment are shown in Table 10.1.

Table 10.1 Model Parameters for Routing the PMF for the Mt Bold Catchment

IL	0 mm
CL	0 mm/hr
m	0.8
k_c	16.3

10.3.4. Initial Storage Level

The storage elevation curve was extended to incorporate the elevations reached with the estimated PMF. The extrapolation of the storage elevation curve is discussed in Section 4.7.

ANCOLD (1986) recommends that unless normal operating procedures indicate otherwise, a reservoir should assumed to be initially full for routing the PMF through the storage.

In Section 8.2 the probability of storage levels for different months was analysed. This indicated that it is likely that the reservoir will be close to full prior to the PMF inflow. It is therefore not considered to be unduly conservative to assume that the reservoir is initially full.

The sensitivity of the calculated PMF to the initial storage level was undertaken using a range of storage levels and the results are shown in Section 10.6.

10.3.5. Treatment of Baseflow

The PMF calculated from the PMP does not include a baseflow component and therefore the baseflow component should strictly be added. In most circumstances the baseflow is only a small proportion of large floods and is therefore ignored. If the baseflow is to be included, IEAust (1987) recommends a value 20 to 50 percent greater than the maximum value estimated in design floods.

In the calibration of the rainfall runoff model for the Mt Bold Catchment, the peak baseflows were generally 10 percent of the peak discharge. The largest estimated peak baseflow was 34 m^3/sec (Table 6.1). A baseflow of approximately 50 m^3/sec could therefore be added to the surface runoff component of the PMF.

In the calculation of the PMF a continuing loss of zero was adopted and the reservoir was assumed to be initially full. The addition of baseflow was considered to be unduly conservative. The effect of the choice of continuing loss was tested by calculating the PMF using a continuing loss of 1 mm/hr. This tended to reduce the peak flows by between 50 to 100 m^3/sec. This was of a similar magnitude to the baseflow component of the PMF.

In an attempt to obtain a reasonable estimate of the PMF it was decided not to add the baseflow. This was because conservative choices of both initial reservoir level and continuing loss had been adopted (Section 10.6.2).

10.4. Results of Preliminary PMF Analysis

The preliminary PMP depths using Bulletin 51 and the estimated longer duration PMPs that were developed in Section 9.6 were routed using the calibrated RORB model. The parameters shown in Table 10.1 were used in the analysis. The storage at Mt Bold was assumed to be initially full for these analyses. The extended spillway rating developed in Section 10.2 was used.

The resulting PMFs are summarised in Table 10.2. It is clear from this table that the attenuating effect of the storage is lower with the longer duration storms. The 6 hour peak flow value for isohyetal location 3 is attenuated by only 3.4 percent whereas the 3 hour peak flow value for isohyetal location 3 is attenuated by 11.9 percent.

Table 10.2 Preliminary PMFs using Bulletin 51 PMPs

Location	1		3		4		5		Uniform Rainfall	
Duration	Inflow	Outflows	Inflow	Outflows	Inflow	Outflows	Inflow	Outflows	Inflow	Outflows
(hrs)	(m^3/sec)	(m^3/sec)	(m^3/sec)	(m^3/sec)	(m^3/sec)	(m^3/sec)	(m^3/sec)	(m^3/sec)	(m^3/sec)	(m^3/sec)
1	5,700	4,000	6,400	4,500	6,100	4,200	5,500	4,100	4,200	3,500
2	8,400	6,900	9,100	7,500	8,800	7,100	8,300	7,000	6,700	6,000
3	9,500	8,300	10,100	8,900	9,600	8,500	9,700	8,500	8,200	7,500
4	9,600	8,800	10,000	9,200	9,700	8,900	9,900	9,100	8,900	8,300
5	9,100	8,600	9,300	8,800	9,000	8,600	9,400	8,700	8,700	8,300
6	8,600	8,300	8,700	8,400	8,500	8,200	8,900	8,400	8,500	8,100
12	-	-	-	-	-	-	-	-	6,800	6,600
24	-	-	-	-	-	-	-	-	4,900	4,700

The peak outflows for different duration PMPs are plotted in Figure 10.4 for different isohyetal locations. The largest peak outflow PMF occurs for a 4 hour PMP situated at location 3. The four different locations tested resulted in similar outflow peaks.

The highest inflow was produced by the 3 hour duration storm.

The use of uniform rainfall resulted in lower outflow PMF estimates. The PMF estimates calculated using uniform PMP tended to approach those for non-uniform PMP for longer durations.

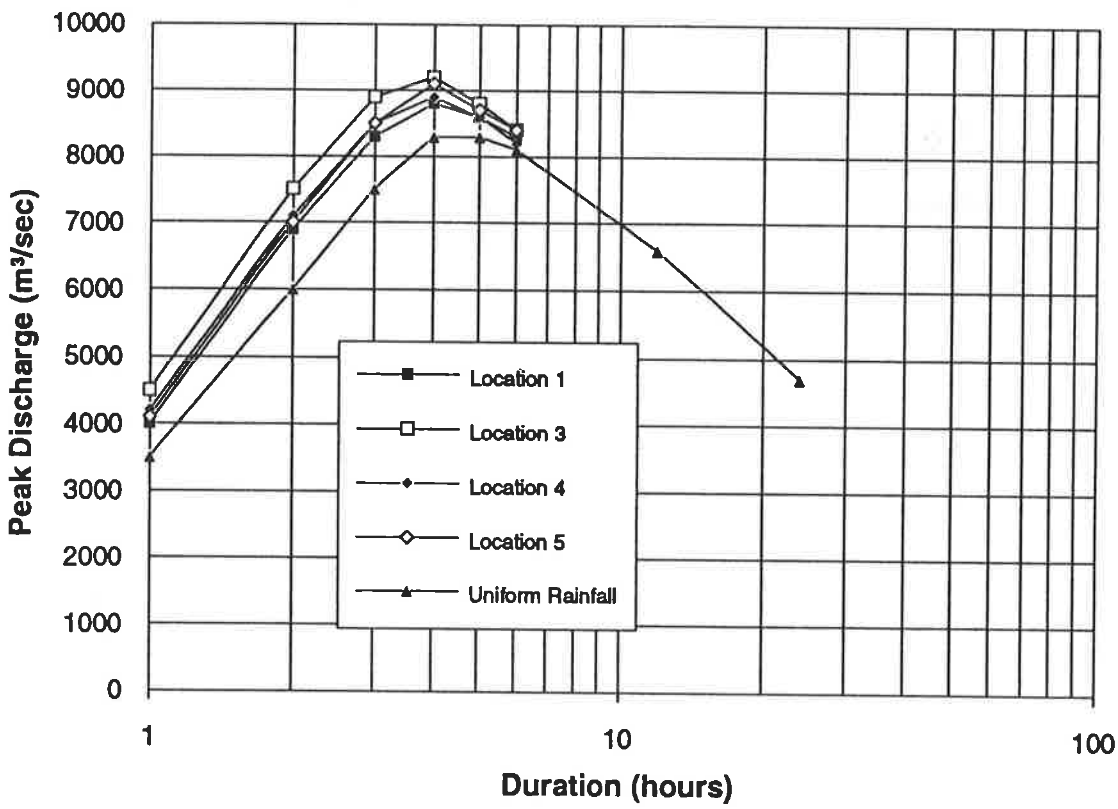


Figure 10.4 Mt Bold Outflow PMFs (Gates open 3.8 metres) for durations up to 24 hours

The sensitivity of the extrapolation of the estimates of the PMP for durations longer than 6 hours was tested by analysing PMP depths that were larger than those predicted by the extrapolation. It was concluded that none of these were critical to the estimate of both the inflow and outflow PMFs. A 5.5 percent change in precipitation estimate gave a 5.6 percent change in inflow and a 6.2 percent change in outflow.

Following the discussion included in Section 9.6.1, the PMF calculated from the 12 hour duration was the average of the PMFs produced by two different temporal patterns. The peak flow values obtained using the 24 hour duration pattern were higher than those obtained from the Bulletin 51 pattern.

10.5. Final PMF Analysis

Once the critical storm location and duration had been identified, a final PMP analysis was undertaken. This is described in Section 9.7. These estimates were then routed to produce outflow PMFs.

In Table 10.3 the PMF values are compared to those based upon the preliminary estimates of the PMP for isohyetal location 3. Although there was a difference in the estimates of the rainfall depth on individual subcatchments for the preliminary and accurate PMP estimates (Table 9.2), there was very little effect on the estimate of the outflow PMF.

Table 10.3 Differences between Preliminary and Final PMF Analysis for Location 3

	Inflow PMF			Outflow PMF		
Duration (hours)	Preliminary (m^3/sec)	Final (m^3/sec)	Difference (%)	Preliminary (m^3/sec)	Final (m^3/sec)	Difference (%)
3	10,100	10,200	-0.9	8,900	8,900	0.0
4	10,000	10,100	-1.0	9,200	9,300	-1.1
5	9,300	9,300	0.0	8,800	8,900	-1.1

The final values for the inflow and outflow PMFs are shown in Table 10.4 for a range of durations for isohyetal location 3. An interesting feature of Table 10.4 is that there is only a difference of 400 mm in the peak elevations reached for durations from 3 to 6 hours.

Table 10.4 Final PMF Analysis for Location 3 for various Durations

Duration (hours)	Inflow (m^3/sec)	Outflow (m^3/sec)	Peak Elevation (m)
1	6,400	4,500	46.75
2	9,100	7,500	48.45
3	10,200	8,900	49.14
4	10,100	9,300	49.30
5	9,300	8,900	49.14
6	8,700	8,400	48.90

The final inflow and outflow PMF hydrographs for Mt Bold are shown in Figure 10.5.

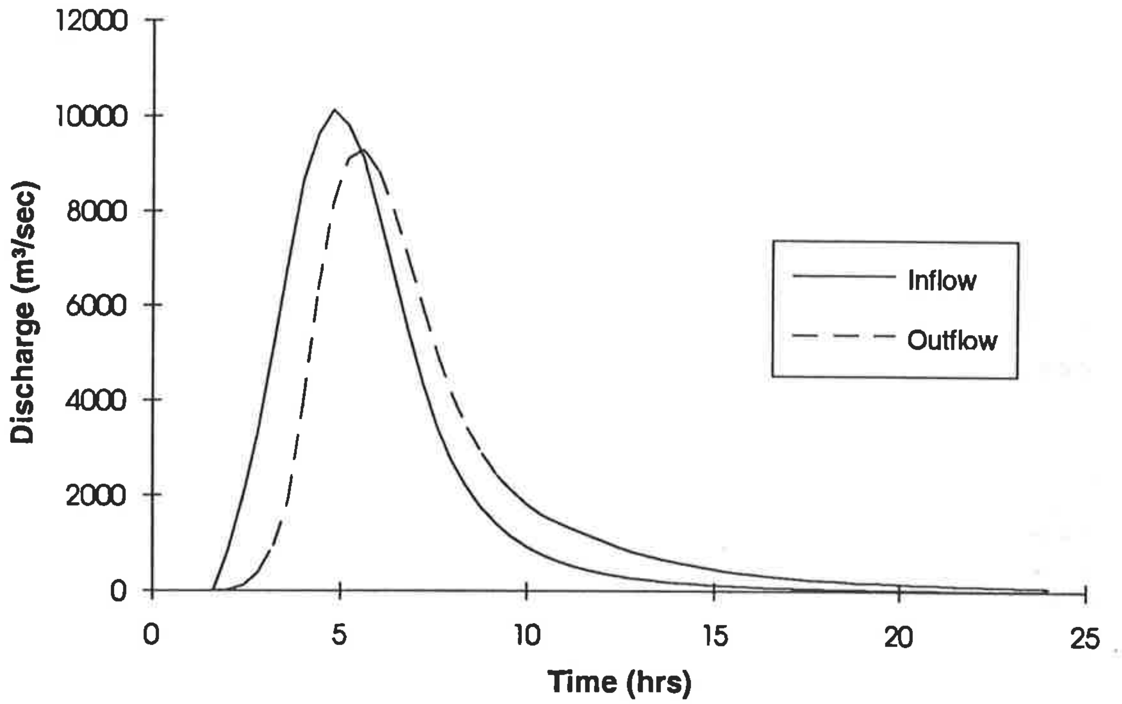


Figure 10.5 Final Inflow/Outflow PMF Hydrographs for Mt Bold Reservoir (4 hr duration and PMP location 3)

10.6. PMF Sensitivity

In order to test the sensitivity of the estimate of the PMF, various parameters were varied in order to give a high and low bound for the PMF. The sensitivity of the estimated value of the PMF to changes of various parameters compared favourably with those listed by Daniell (1987).

10.6.1. Isohyetal Location

It is recommended by Bulletin 51 and IEAust (1987) that the choice of the location of the isohyetal pattern for the PMP should be a central location, however following the discussion in Section 9.2, the sensitivity of the PMF to the location of the PMP was analysed.

Five different locations were tested and isohyetal location 3 was found to be critical (Section 9.2 and 10.4).

The PMF calculated using isohyetal location 3 is compared with those produced using other isohyetal locations in Table 10.5.

Table 10.5 Comparison of Outflow PMFs Calculated from Different Isohyetal Locations

Duration (hrs)	Location 3 (m^3/sec)	Difference			
		Location 1	Location 4	Location 5	Uniform Rainfall
1	4,500	-11%	-7%	-9%	-22%
2	7,500	-8%	-5%	-7%	-20%
3	8,900	-7%	-4%	-4%	-16%
4	9,200	-4%	-3%	-1%	-10%
5	8,800	-2%	-2%	-1%	-6%
6	8,400	-1%	-2%	0%	-4%

Note: Location 2 was discarded (Section 9.2.2)

It is clear from Table 10.5 that the differences between the PMF calculated using isohyetal location 3 and the PMFs calculated using the other isohyetal locations are greatest for the shorter durations. As the PMP duration increases the difference between the outflow PMF estimates decreases. At the critical location, the difference is approximately 5 percent.

In the previous PMF study of the Mt Bold Catchment (Kotwicki, 1984) a uniformly distributed storm was used. The PMF analysis has shown that this would have resulted in a lower estimate of the PMF by approximately 10 percent at the critical durations but there is little difference at the 6 hour duration. Spatially distributed storms are recommended by the Bureau of Meteorology and in IEAust (1987) primarily because that is the mechanism by which these extreme events occur.

10.6.2. Baseflow

From the calibration of the RORB model, a baseflow of up to 50 m³/sec could be added to the calculated surface runoff PMF. This represents 0.5 percent of the peak inflow PMF.

It was concluded that the baseflow should not be added if the continuing loss is zero (Section 10.3.5).

10.6.3. Initial Storage Level

Although the Mt Bold Reservoir was assumed to be full for the 4 hour duration PMP storm event, a thorough analysis was undertaken using the distribution of storage levels likely for different seasons.

Figure 10.6 shows the PMF outflow hydrographs for different initial reservoir levels for the period during autumn and late summer when the reservoir is drawn down. This was calculated using the average end of month storages for January, February, March and April.

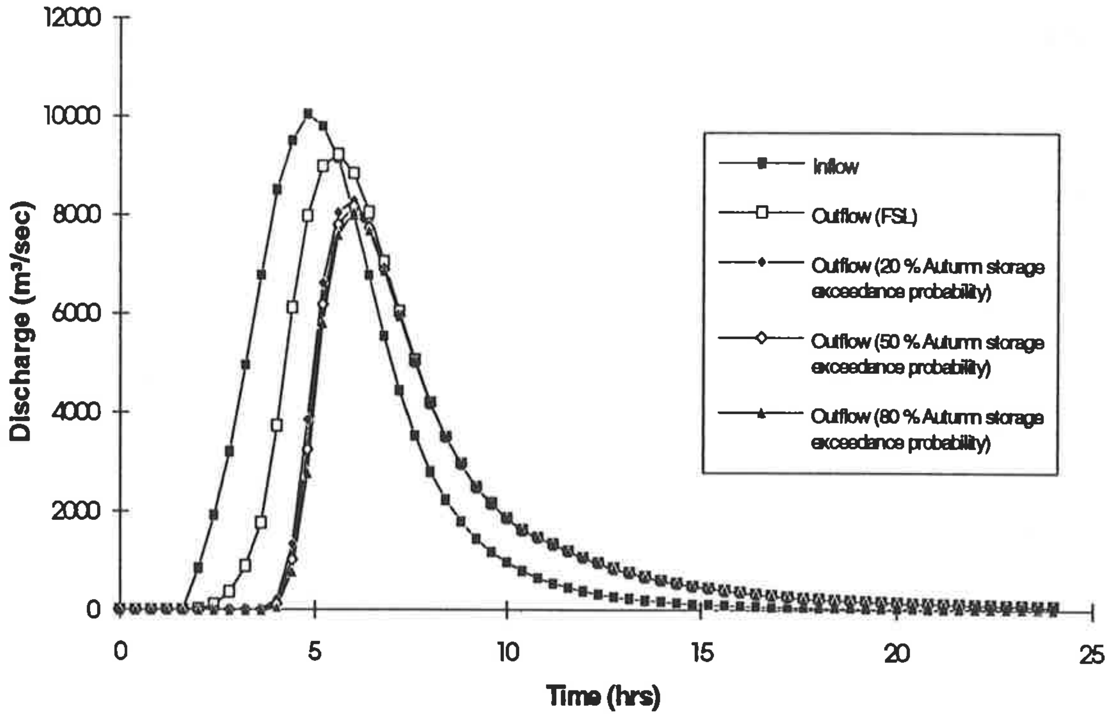


Figure 10.6 Effect of Initial Storage Level on PMF Hydrographs for a 4 hour Duration Storm and PMP Location 3

The likelihood of the storage being full due to rainfall preceding the PMP was considered to be sufficiently high to choose the full storage option for analysis. This option is also recommended by ANCOLD (1986).

10.6.4. Model Non-Linearity

The sensitivity of parameters for the runoff model was investigated with a range of values being examined. Figure 10.7 shows the different hydrographs resulting from different choices of non-linearity.

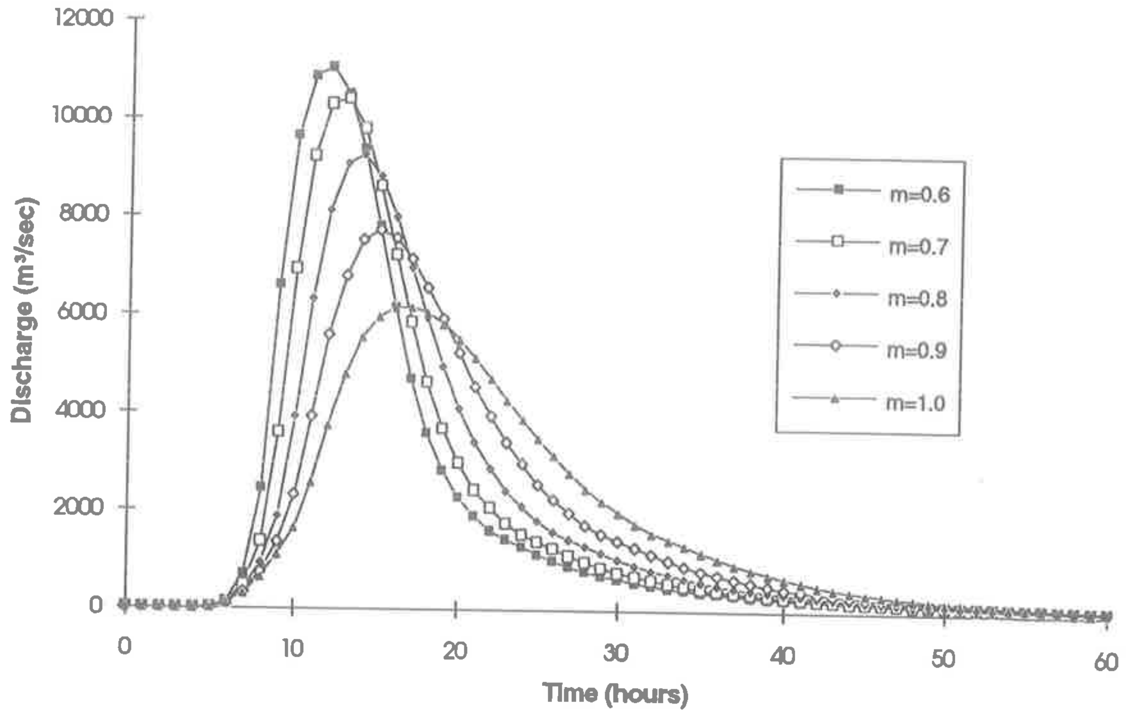


Figure 10.7 Effect of Model Non-linearity on Calculated Outflow PMF (zero losses)

Table 10.6 compares the peak flows obtained from different values of the parameter *m*. It is clear that as linearity increases the PMF peak diminishes. The choice of extreme non-linearity results in overestimating the inflow PMF by 26 percent and the outflow PMF by 19 percent. The choice of a linear model would result in underestimating the inflow PMF by 35 percent and the outflow PMF by 33 percent.

Table 10.6 Effect of Model Non-Linearity on the PMF

		Inflow PMF		Outflow PMF	
m	k_c	Peak (m³/sec)	Comparison with m=0.8	Peak (m³/sec)	Comparison with m=0.8
0.6	46.4	12,700	+26%	11,100	+19%
0.7	27.4	11,400	+13%	10,400	+12%
0.8	16.3	10,100	-	9,300	-
0.9	9.5	8,200	-19%	7,700	-17%
1.0	5.5	6,500	-35%	6,200	-33%

10.6.5. Choice of Losses

The PMF was calculated using zero losses. The sensitivity of the PMF to the choice of initial and continuing loss was tested by calculating the PMF using different losses. The results are shown in Table 10.7.

Table 10.7 Effect of Losses on the PMF

		Inflow PMF		Outflow PMF	
IL (mm)	CL (mm/hr)	Peak (m³/sec)	Comparison with zero losses	Peak (m³/sec)	Comparison with zero losses
0	0	10,100	-	9,300	-
0	1	10,000	-1%	9,200	-1%
0	2	10,000	-1%	9,100	-2%
10	0	9,900	-2%	9,100	-2%
20	0	9,700	-4%	8,800	-5%

From Table 10.7 it is evident that the choice of losses, within reasonable bounds, does not have a dramatic effect on the calculated PMF. The adoption of a CL of 1 mm/hr would only decrease the outflow PMF by 1 percent.

10.6.6. Extrapolated Spillway Rating Curve

The calculation of the spillway rating curve effects the estimated outflow PMF. The calculated rating was approximately 15 percent less than the existing spillway rating. The adopted spillway rating curve calculated using the gated spillway equations in Chapter 4 has not been verified for the Mt Bold Dam. The actual rating is expected to vary slightly from the theoretical because of errors in the estimation of the design head, contraction coefficients, velocity head and coefficient of discharge.

The routing of the PMF through the storage, necessitated the extrapolation of the rating curve. This involved extrapolation of the gated spillway equations beyond their recommended limits. Several assumptions were also made regarding the validity of the broad crested weir formula to adequately represent the flow over the dam crest and through the saddle.

For large flows the flow through the saddle represents a large proportion of the total flow (Section 10.2.4). An accurate survey and hydraulic analysis of the saddle should therefore be undertaken to reduce the large uncertainty existing as to the extended rating.

10.7. Reasonable Bounds of the PMF

The determination of the PMF is but one important element in the design or evaluation of a spillway. The reasonableness of the design will not only depend on the estimation of the PMF but also on the probability assigned to the PMF and also to the freeboard and other parameters adopted. It is important that all parameters represent reasonable estimates. Not withstanding this, it was thought necessary to estimate the upper and lower bounds of a reasonable estimate of the PMF.

The calculation of reasonable bounds of the PMF necessitated judgement as to what was considered to be 'reasonable'. The bounds calculated were not intended to be physically based bounds which could not be exceeded. The intention was to determine bounds which provided reasonable limits to the estimate of the PMF.

The parameters adopted to represent the upper and lower bounds of a reasonable estimate of the PMF are shown in Table 10.8.

Table 10.8 Bounds of Reasonable Estimates of the PMF for Mt Bold Reservoir

	High Bound	Low Bound
Isohyetal location	Location 3	Location 1
Baseflow	+ 100 m^3/sec	0
Initial reservoir level	Full Supply Level	15 GL drawdown
Model parameters	m=0.7, k_c=27.4	m=1.0, k_c=5.5
Initial loss	0	20 mm
Continuing loss	0	2 mm/hr
PMF Inflow	12,400 m^3/sec	5,600 m^3/sec
Difference from adopted (10,200 m^3/sec)	+ 22 %	- 45 %
PMF Outflow	10,800 m^3/sec	5,200 m^3/sec
Difference from adopted (9,300 m^3/sec)	+ 16 %	- 44 %

The partial area effect discussed in Section 10.9 was not considered to be a reasonable estimate of the PMF. The case of spatially uniform PMP was not considered to represent a reasonable lower bound of the PMF. This is because although spatially uniform PMPs have been used in the past, Bulletin 51 recommends the use of spatially non-uniform PMPs.

The initial storage level was assumed to be at full supply level (Section 10.6.3). This gives the upper bound. A drawdown of 15 GL was considered a reasonable lower bound for the initial storage level (Figure 8.11).

The choice of model non-linearity has a large effect on the calculated PMF (Section 10.6.4). A value of m of 0.7 was chosen for the lower bound and a value of m of 1.0 was chosen for the upper bound. The choice of a linear model represents the use of a unit hydrograph. Although a value of m of 0.6 was shown in Figure 10.7, it was considered to be overly conservative.

It is evident from Table 10.8 that the adopted PMF estimates are well within the reasonable bounds. The estimates are slightly on the high side. The adopted inflow and outflow PMFs are considered to be the best estimate.

Although the upper bound is 22 percent above the adopted value, it should be noted that this only represents a difference in the peak stage height of 650 mm. The lower bound represents a decrease in the peak stage height of 2.1 metres.

10.8. Extension of the Flood Frequency Curve to the PMF

It is not theoretically possible to assign an annual exceedance probability (AEP) to the PMF (Section 2.3.5). It is however necessary to estimate a probability for the PMF for economic analysis and in order to determine the probabilities of flows between the PMF and the 1 in 100 AEP discharge. A methodology of calculating the probability of the PMF and lesser events is outlined in IEAust (1987), based on Rowbottom et al. (1986b).

10.8.1. Assignment of a Probability to the Mt Bold PMF

The procedures in IEAust (1987) for assigning a probability to the PMF are based upon two different criteria. The largest probability is assigned to the PMF. The first method depends upon the effective transposition area used to calculate the PMP. For use of adjusted US data a probability of approximately 1 in 10^8 is recommended.

The second method depends on the shape of the flood frequency curve. The AEP of the PMF is determined by the value of the ratio in Equation 10.4. Australia is divided into two regions, with the south west of Australia designated as zone B and the remainder of Australia as zone A. Different probabilities are given for zone A and zone B.

$$Ratio = \frac{\log\left(\frac{X_{PMF}}{X_{100}}\right)}{\log\left(\frac{X_{100}}{X_{50}}\right)} \tag{10.4}$$

where: X_Y is the discharge for an AEP of 1 in Y; and
X_{PMF} is the PMF discharge

The expression in Equation 10.4 was evaluated for the inflow and the outflow frequency curves. These were derived in Chapters 7 and 8 respectively. The value of the ratio and the consequent AEP of the PMF are shown in Table 10.9.

Table 10.9 AEP of Mt Bold PMF

	Inflow	Outflow
1 in 50 AEP Event	555 m³/sec	250 m³/sec
1 in 100 EP Event	700 m³/sec	365 m³/sec
PMF	10,200 m³/sec	9,300 m³/sec
Value of Ratio (Equation 10.9)	11.5	8.6
AEP of PMF	1 in 10^7	1 in 10^6

Table 10.9 indicates that a probability of either 1 in 10^6 or 1 in 10^7 would be appropriate for the PMF (IEAust, 1987). In keeping with the intention to obtain a reasonable degree of conservatism, an AEP of 1 in 10^6 was adopted as the probability of the PMF.

Selection of the discharge to be used for the PMF plotting position has not been rigorously defined in the literature.

The largest outflow PMF (9,300 m³/sec) was obtained from a 4 hour duration PMP, whereas the largest inflow PMF (10,200 m³/sec) was obtained from a 3 hour duration PMP. The inflow PMF corresponding to the outflow of 9,300 m³/sec was 10,100 m³/sec. The largest calculated inflow PMF of 10,200 m³/sec was adopted as the inflow PMF because the frequency curve was considered to be an envelope of frequency curves for different durations.

10.8.2. Flood Frequency Curve Between the 1 in 100 AEP event and the PMF

The curve fitting procedures included in IEAust (1987) were used to determine the flood frequency curve between the 1 in 100 AEP event and the PMF. The first requirement was to identify the lower end of the frequency curve by the estimation of the 1 in 50 AEP and 1 in 100 AEP discharges.

The flood frequency analysis for this catchment described in Chapter 7 concluded that for inflows to Mt Bold Reservoir, both a log Normal or log Pearson III distribution were considered acceptable under various fit criteria. In a study to set up a regional flood estimation technique (Akter, 1992), it was found that both of these distributions were appropriate for many catchments in the Mt Lofty Ranges. The log Normal distribution was adopted (Section 7.3.1).

The outflow flood frequency curve was calculated in Chapter 8. This involved consideration of the joint probability of inflow and initial storage level. The outflow frequency curve is very important as it is used to assign an AEP to the imminent failure flood (Chapter 11). The average composite distribution was chosen as the most appropriate outflow distribution.

The assigned AEP of the PMF and the intermediate discharges based on the methodology of IEAust (1987) are shown in Table 10.10.

Table 10.10 Discharges between 1 in 20 AEP and the PMF

AEP	Mt Bold Inflow		Mt Bold Outflow	Clarendon Weir
(1 in Y)	(log Normal)	(log Pearson III)	(Average Composite Curve)	(log Pearson III)
20	430	340	165	190
50	555	420	250	235
100	700	475	365	275
2,000	1,810	1,160	1,400	760
50,000	5,420	4,740	4,560	3,850
1,000,000	10,200	10,200	9,300	9,300

The flood frequency curves for Mt Bold Reservoir extended to the PMF are shown in Figure 10.8. The effect of the choice of either the recommended log Normal distribution or the log Pearson III distribution for the inflow frequency distribution is also shown.

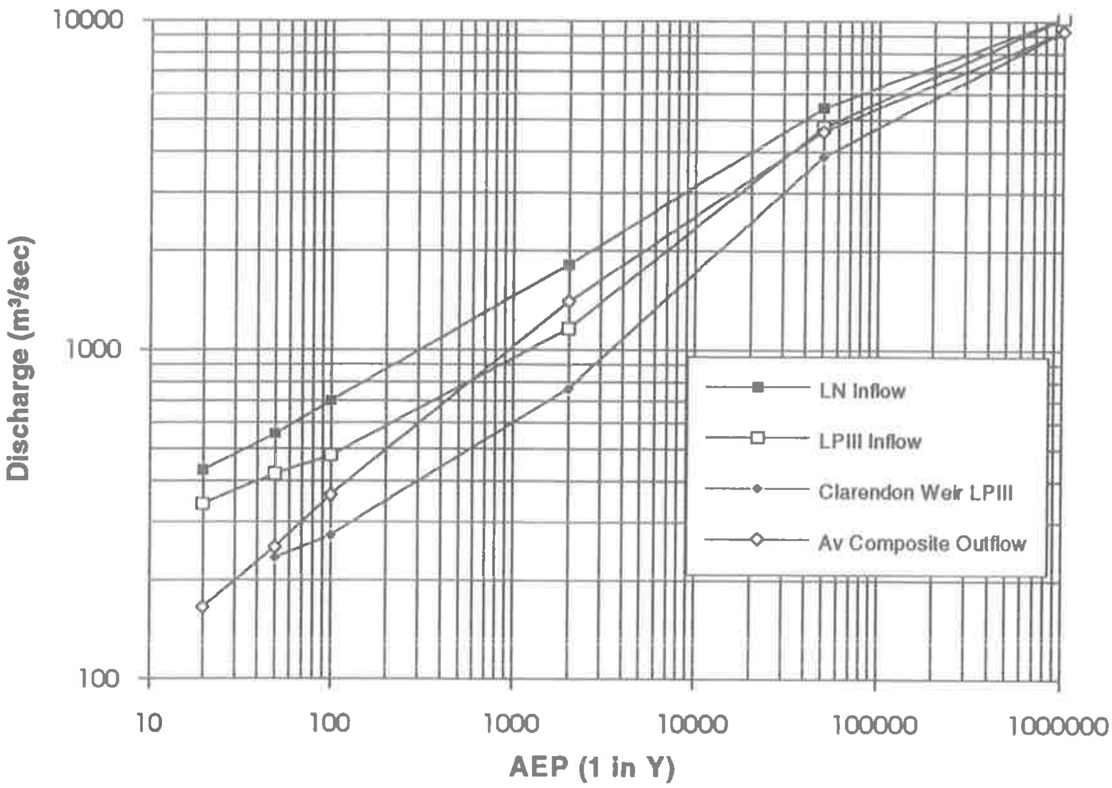


Figure 10.8 Mt Bold PMF Flood Frequency Curves

10.9. PMF for Clarendon Weir and Old Noarlunga

The PMFs for Clarendon Weir and Old Noarlunga were determined using the calibrated RORB model for the Old Noarlunga Catchment. The PMP was calculated for the 522 km² catchment using isohyetal locations 3 and 4 for the Mt Bold Catchment (Section 9.8).

The PMF estimates for Mt Bold, Clarendon and Old Noarlunga are shown in Table 10.11.

The PMFs calculated at Clarendon Weir and Old Noarlunga using isohyetal locations 3 and 4 were very similar.

From Table 10.11 it is interesting to note that the PMF inflow and outflow for Mt Bold Catchment calculated using the PMP for the Old Noarlunga Catchment are higher than those calculated using the PMP for the Mt Bold Catchment. This is due to a partial area effect. Although the average depth of rainfall over the Old Noarlunga Catchment is less than that for the Mt Bold Catchment the isohyetal pattern results in a greater volume of rainfall being situated in the catchment to Mt Bold Reservoir.

Table 10.11 PMF (m³/sec) from Old Noarlunga PMP, Old Noarlunga Model

	Duration	1 hr	2 hr	3 hr	4 hr	5 hr	6 hr	12 hr	24 hr
Isohyetal Location 3	Mt Bold Inflow	7,050	10,300	11,100	10,800	10,000	9,190	-	-
	Mt Bold Outflow	5,150	8,650	9,850	10,000	9,450	8,900	-	-
	Clarendon Weir	4,700	8,000	9,300	9,600	9,200	8,800	-	-
	Old Noarlunga	3,400	5,950	7,150	7,800	7,800	7,700	-	-
Isohyetal Location 4	Mt Bold Inflow	7,000	10,400	10,900	10,800	10,000	9,250	-	-
	Mt Bold Outflow	4,950	8,500	9,800	10,100	9,600	9,000	-	-
	Clarendon Weir	4,600	7,950	9,300	9,750	9,300	8,900	-	-
	Old Noarlunga	3,400	5,950	7,200	7,900	7,950	7,800	-	-
Uniform Rainfall	Mt Bold Inflow	3,700	6,300	7,600	8,300	8,100	7,900	6,500	4,700
	Mt Bold Outflow	3,000	5,600	7,000	7,700	7,700	7,500	6,400	4,500
	Clarendon Weir	2,900	5,400	6,700	7,400	7,400	7,300	6,400	4,700
	Old Noarlunga	2,400	4,400	5,500	6,200	6,300	6,400	6,100	4,700

Note: Values for Mt Bold inflow and outflows were not adopted (Section 10.9)

This partial area flow was not considered to be a reasonable estimate for the PMF for Mt Bold Dam even though it was 800 m³/sec greater than the critical value determined by applying the PMP over the Mt Bold Catchment. The best estimate for the PMF was deemed to be 9,200 m³/sec at Clarendon Weir and 7,900 m³/sec at Old Noarlunga. The PMF hydrographs are shown in Figure 10.9 for a 4 hour PMP situated at location 4.

The sensitivity of the PMF estimate to the choice of PMP for the longer durations of 12 and 24 hours was undertaken by varying the PMP depth. The resulting changes in PMF peak flows were not significant.

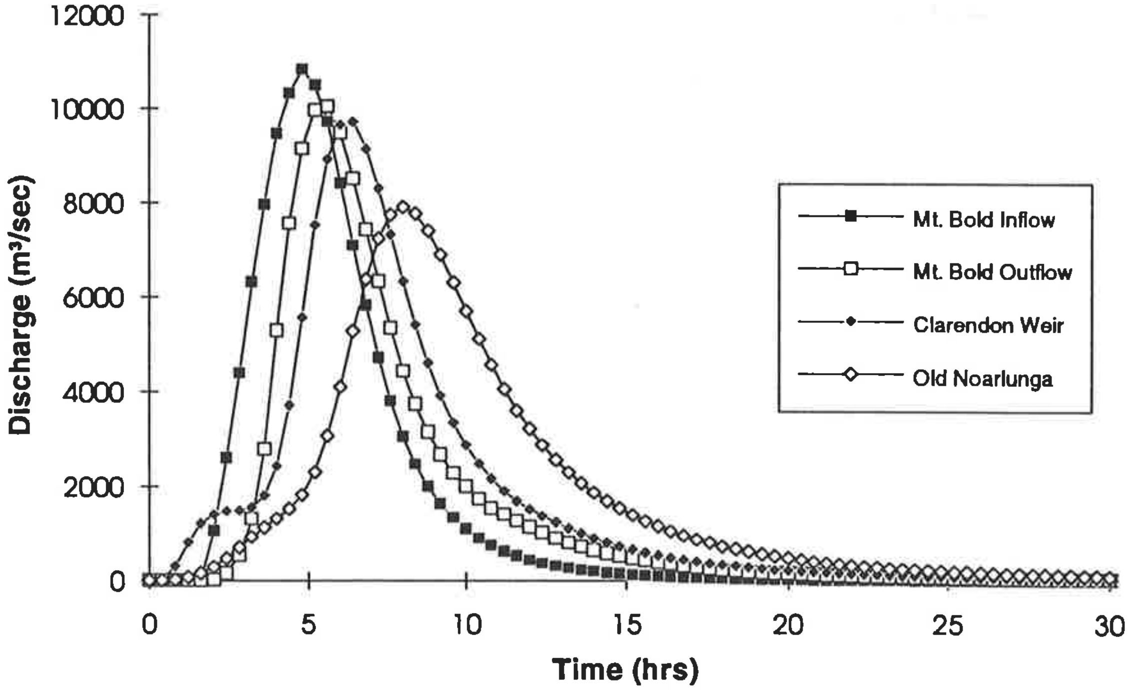


Figure 10.9 PMF Hydrographs For Onkaparinga River Downstream of Mt Bold (Old Noarlunga Model, PMP Location 4, Duration 4 hours)

10.9.1. Frequency Curve for the Lower Onkaparinga River

The flood frequency curves for the Onkaparinga River below Mt Bold Reservoir were extended up to the PMF using the method described in Section 10.8.2. The results are shown in Table 10.12.

Table 10.12 Peak Flows for the Onkaparinga River below Mt Bold Dam

Peak Flows in m³/sec

AEP(1 in Y)	20	50	100	2,000	50,000	1,000,000
Hist Clarendon LPIII	190	235	275	760	3,820	9,200
Ave. Composite Mt Bold Outflow	165	250	365	1,400	4,560	9,200
Ave. Composite Clarendon Weir	205	300	375	1,060	4,210	9,200
Ave. Composite Old Noarlunga	175	260	325	920	3,630	7,900

The frequency distributions up to the PMF for four different locations for the Lower Onkaparinga River are shown in Figure 10.10. It is clear from Figure 10.10 that the use of the log Pearson III distribution fitted to the historical records at Clarendon Weir would result in lower estimates of the peak flows than those calculated using the average composite curves.

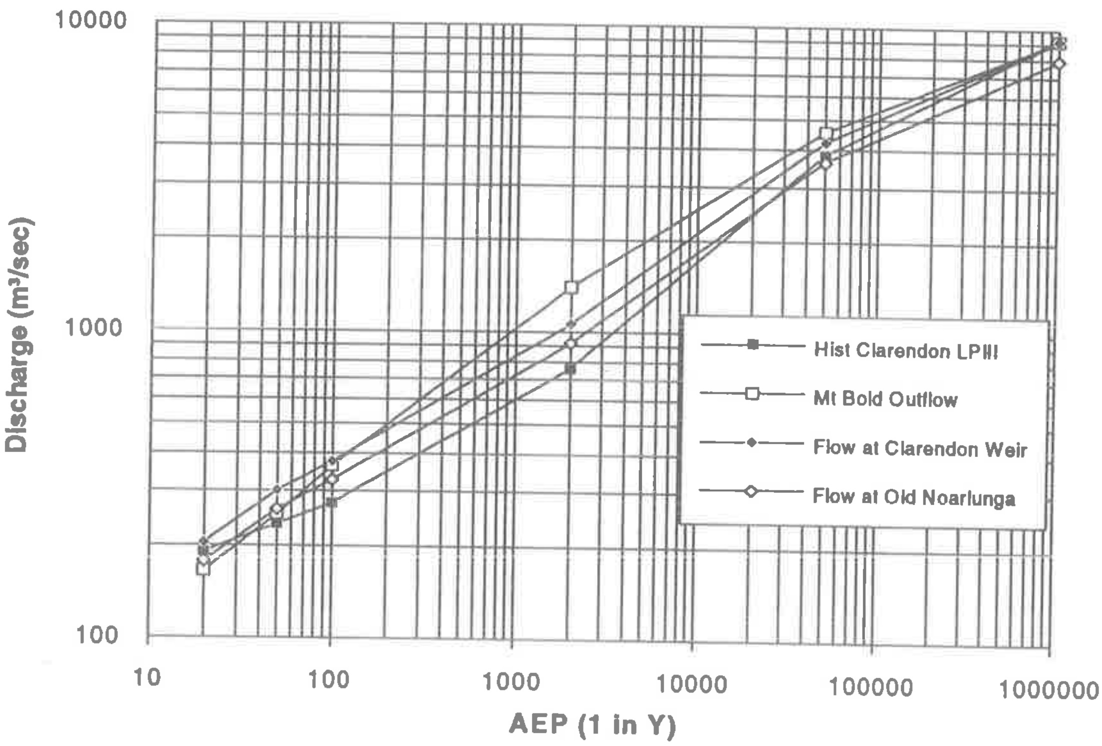


Figure 10.10 PMF Flood Frequency Curves Downstream of Mt Bold Dam

10.10. Summary

The inflow PMF for Mt Bold Dam has been determined at 10,200 m^3/sec for a 3 hour duration storm. The peak outflow PMF, using the current spillway and gate configuration of Mt Bold Dam is 9,300 m^3/sec for a 4 hour duration event. This outflow PMF would overtop the dam crest by approximately 6.4 metres.

This flood brings into operation a side discharge through a saddle which takes approximately 25 percent of the PMF and hence reduces the level by which the PMF overtops the dam by more than 2 metres.

The peak outflow of 9,300 m^3/sec resulted from a 4 hour duration storm with an inflow peak of 10,100 m^3/sec. The PMF at Old Noarlunga downstream of Mt Bold Dam was 7,900 m^3/sec. The PMF at Clarendon Weir was 9,200 m^3/sec.

The sensitivity of the PMF estimate to the choice of various parameters was tested. It was found that the PMF estimate was particularly sensitive to the model non-linearity and the choice of uniform or spatially varying rainfall.

Both high and low bounds of a reasonable estimate of the PMF were calculated. The high and low bounds of the inflow PMF were 12,400 m^3/sec and 5,600 m^3/sec respectively. The high and low bounds of the outflow PMF were 10,800 m^3/sec and 5,200 m^3/sec respectively.

Chapter 11

Imminent Failure Flood

11.1. Introduction

The PMF and frequency curves for Mt Bold Reservoir were determined in Chapter 10. The structural capacity of the dam was then considered by examining previous calculations by members of the E&WS. The imminent failure flood (IFF), which is the peak flood that initiates failure of the dam, was then determined.

Once the IFF was found the frequency curve was used to assign an AEP to the IFF.

11.2. IFF for Mt Bold Dam

The modifications to Mt Bold Reservoir which took place between 1961 and 1962 involved the addition of vertical lift gates and the raising of the dam wall. Because of concerns over the additional weight of the raised portion, the raised section of the dam was a hollow portal frame. Above the outlet valves a double portal frame was constructed because of the greater dam width.

Upon the completion of the dam modifications and subsequent filling of the reservoir, water was noted spurting from a few points on the upper dayplanes (discontinuities between different concrete pours). In 1973 the top four dayplanes were post tensioned (Design Services, 1982).

The IFF for Mt Bold Dam was difficult to determine because of the unusual geometry of the dam wall. Possible modes of failure include:

- overturning of the single portal frame;
- overturning of the double portal frame;
- uplift of one of the post tensioned dayplanes; or
- uplift of the highest non post tensioned day plane.

Two different IFF levels were considered in this study. The first is the dam crest at an elevation of 248.42 metres (GH of 42.92 metres). The second is 1 metre above the dam crest at an elevation of 249.42 metres (GH 43.92 metres), (Parsons, 1993, pers. comm.; Daniell and Hill, 1993c).

11.3. Mt Bold Spillway Rating

The extended rating curve for the Mt Bold Dam was calculated in Section 10.2. It was assumed that the gates were in a fully raised position; corresponding to a gate opening of 3.8 metres. The rating is shown in Figure 10.3.

A discontinuity in the rating curve was examined in Section 4.3.3. As the water passes the foot of the gate and head develops on the upstream side a critical level is reached where the water passing under the gate sheds from the upstream edge rather than the downstream edge of the bottom of the gate, causing a reduction in the discharge. This effect is shown in Figure 11.1 for three different gate openings.

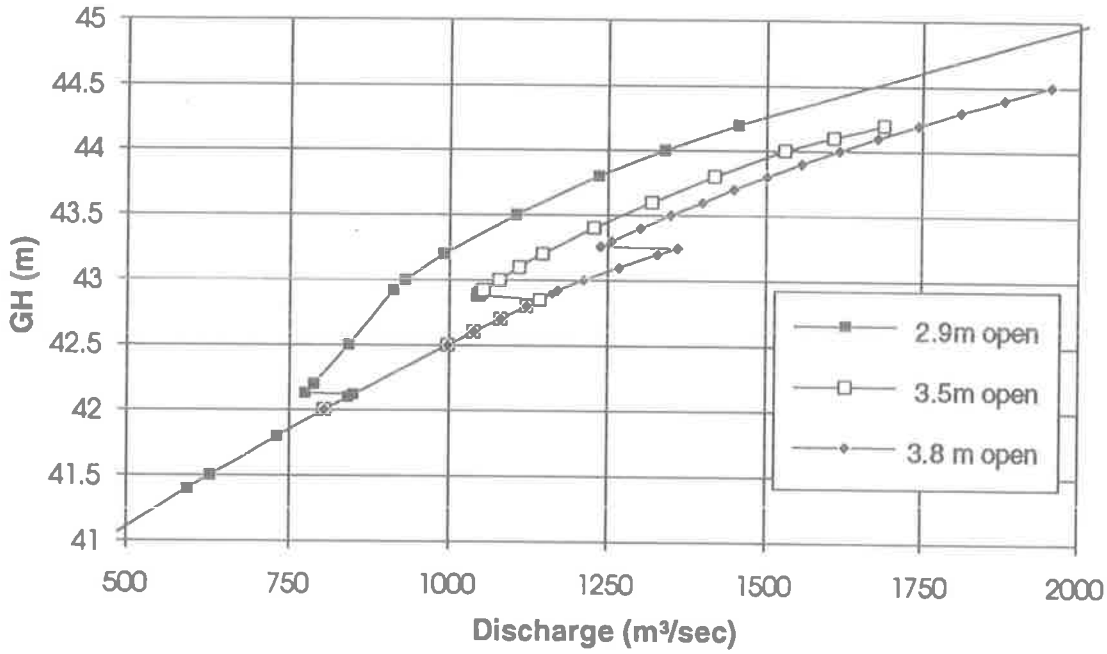


Figure 11.1 Rating Curve for Mt Bold Dam Near the IFF

At the point of discontinuity, the reduction in the coefficient of discharge results in a sudden reduction in the spillway capacity as shown in Figure 11.1. Depending upon the maximum gate opening, this effect occurs at a reservoir level of approximately the dam crest (GH 42.92).

The current maximum gate openings are discussed in Section 4.2.2. It is recommended that the maximum gate opening be increased to 3.8 metres. The rating near the IFF was calculated for maximum gate openings of 2.9, 3.5 and 3.8 metres.

11.4. AEP of Imminent Failure Flood

Once the storage water level which initiates failure was determined a corresponding flow could be determined from Figure 11.1 for a given maximum gate opening. The AEP of the IFF could then be determined. Table 11.1 shows the outflows and consequent probabilities for the two critical reservoir levels and three different maximum gate openings.

Table 11.1 AEPs of IFF Levels at Dam Crest and 1m above Dam Crest.

G.H. (m)	EL (m)	Gate Opening (m)	Outflow (m^3/sec)	AEP	
				Clarendon LPIII	Av Composite Outflow
42.92	248.42	2.9	915	1 in 2,800	1 in 750
		3.5	1,055	1 in 3,800	1 in 1,100
		3.8	1,170	1 in 4,600	1 in 1,300
43.92	249.42	2.9	1,300	1 in 6,000	1 in 1,800
		3.5	1,500	1 in 8,000	1 in 2,000
		3.8	1,580	1 in 8,500	1 in 2,400

It is clear from Table 11.1 that the probability assigned to the IFF is dependent upon the choice of outflow flood frequency distribution. In Section 8.4 the average composite outflow was deemed to be the most suitable distribution. The log Pearson III distribution at Clarendon Weir is included in Table 11.1 for comparison. The log Pearson III distribution is not considered to be appropriate because of changes in the pumping and gate operating policies of the E&WS (Section 8.1.1).

The methods of constructing the flood frequency curve between the 1 in 100 AEP event and the PMF are discussed in Section 10.8.

11.4.1. Risk of Failure

It is important that the annual probability of failure is recognised as a consequent probability of failure during the design life of the structure (Section 7.1.3). Table 11.2

shows the relationship between the probability of an annual event, the design life and the probability of failure during that design life (Equation 7.1).

Table 11.2 assumes that no modifications are made to the structure during its design life that affect its probability of failure. It is recognised that throughout the design life of the dam, modifications and safety reviews will be undertaken.

Table 11.2 Design Life and Probability of Event Occurring

AEP	Design Life in Years					
(1 in Y)	10	20	50	100	200	500
5	89.26%	98.85%	99.999%	100.00%	100.00%	100.00%
10	65.13%	87.84%	99.48%	99.9973%	100.00%	100.00%
20	40.13%	64.15%	92.31%	99.41%	99.997%	100.00%
50	18.29%	33.24%	63.58%	86.74%	98.24%	99.996%
100	9.56%	18.21%	39.50%	63.40%	86.60%	99.34%
200	4.89%	9.54%	22.17%	39.42%	63.30%	91.84%
500	1.98%	3.92%	9.53%	18.14%	32.99%	63.25%
1000	1.00%	1.98%	4.88%	9.52%	18.14%	39.36%
2000	0.50%	1.00%	2.47%	4.88%	9.52%	22.12%
10000	0.10%	0.20%	0.50%	1.00%	1.98%	4.88%

11.5. IFF Hydrographs for Mt Bold Dam

Following the identification of the IFF, the IFF outflow hydrograph was calculated. It was assumed that the reservoir was initially full and that the maximum gate openings had been reset to 3.8 metres. Hydrographs were required for peak flows of 1,170 m³/sec and 1,580 m³/sec (Table 11.1).

Following the discussion in Section 2.5.3, the IFF hydrograph was not obtained by scaling the ordinates of the PMF. Rather the design rainfall was scaled so that the required peak outflow was obtained (Green, 1991).

The critical storm duration for the design outflows for AEPs greater than 1 in 500 was 24 hours (Section 7.4), however the critical storm duration for the outflow PMF was 4 hours (Section 10.4). It was therefore decided to calculate IFF outflow hydrographs for different storm durations.

The rainfall depth, which produced the required peak outflows (1,170 and 1,580 m³/sec), was determined for each duration.

The outflow hydrographs for the IFF for Mt Bold Dam were estimated by firstly establishing the outflow flood frequency curves for the 3, 6 and 24 hour duration storms. These are shown in Figure 11.2. The curves were obtained by assigning an AEP of 1 in

10^6 to the PMF and then finding the shape of the curve from the procedures outlined in Section 10.8 and IEAust (1987).

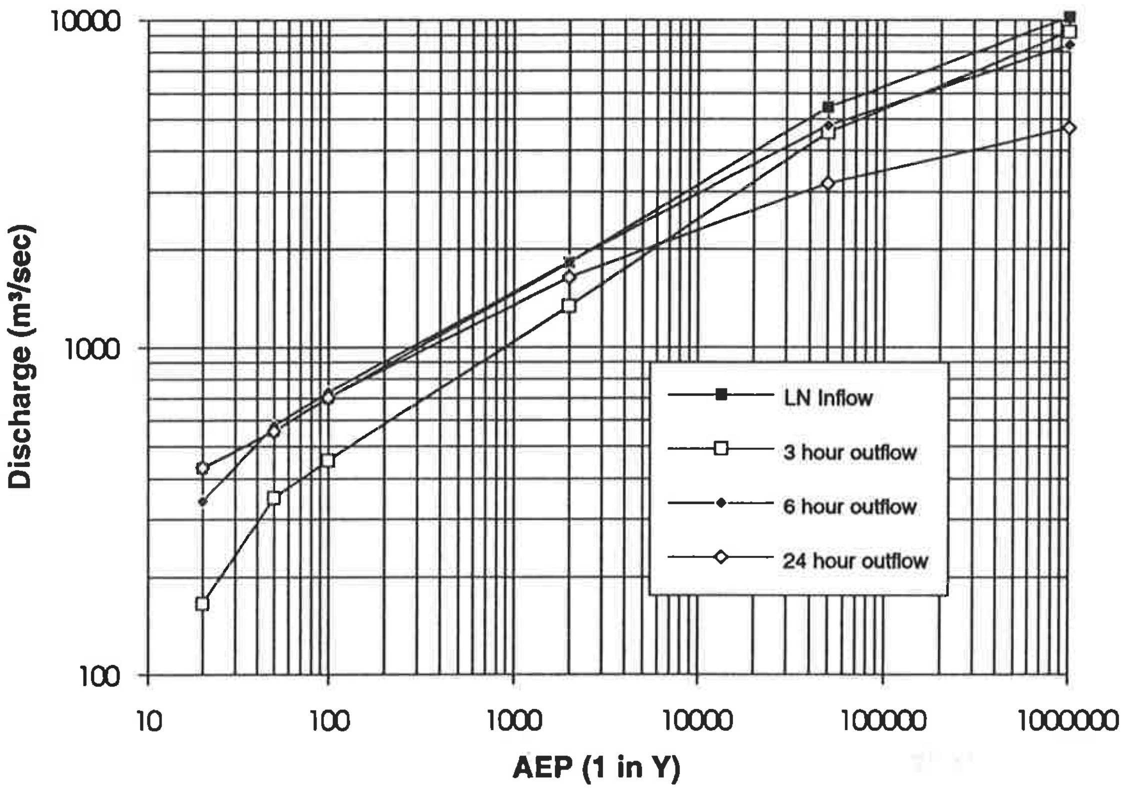


Figure 11.2 Flow Frequency Curves for Different Duration Storms Extended to the PMF

From Figure 11.2 a probability was assigned for the two required outflow peaks for each duration storm.

Figure 11.3 shows the rainfall frequency distributions for different durations. The PMP was assigned a probability of 1 in 10^6. The curves were derived using the procedures outlined IEAust (1987). A rainfall depth was then determined for a given duration and probability.

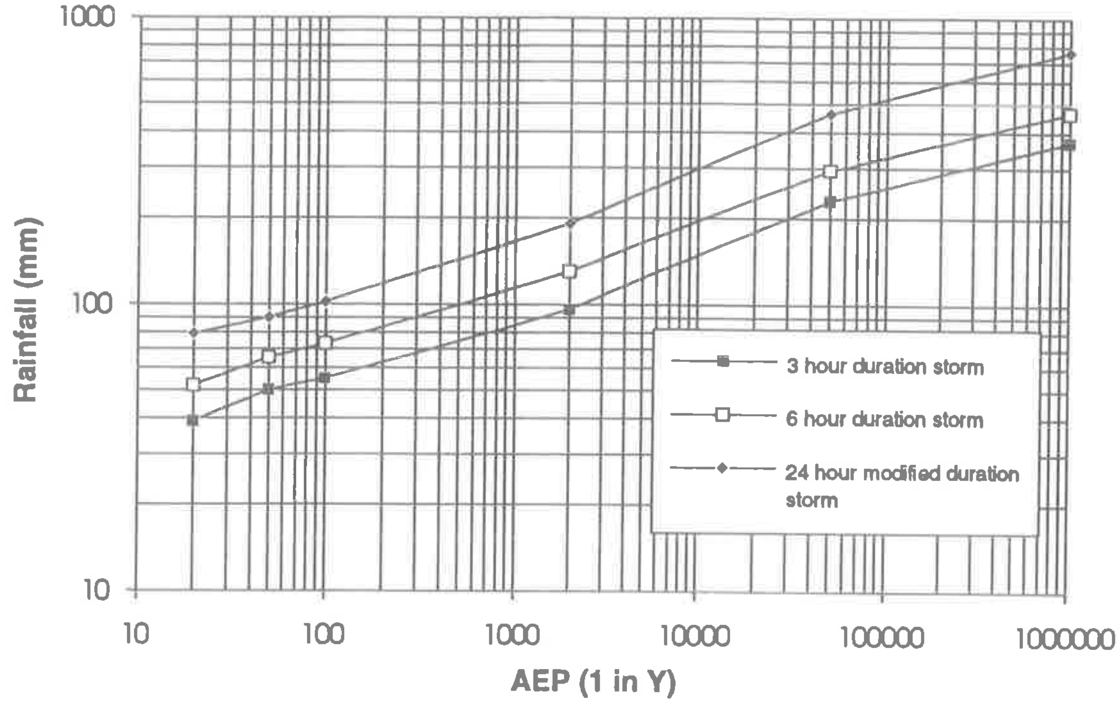


Figure 11.3 Mt Bold Catchment Extended Rainfall Frequency Curves

Estimates of rainfall were obtained using Figure 11.3 which were then converted to flows using the RORB model. The rainfall depth was varied until the desired peak outflow was obtained. The required design rainfalls for different durations that produced peak outflows of 1,170 m³/sec and 1,580 m³/sec are shown in Table 11.3.

Table 11.3 Inflow Peaks to give IFF Outflows and Required Design Rainfalls

Duration (hrs)	ARI (yrs)	Precipitation Depth (mm)	Peak Inflow (m³/sec)	Peak Outflow (m³/sec)
3	1,500	97.5	1,480	1,170
6	500	105.6	1,450	1,170
24	650	154.0	1,420	1,170
3	3,000	121.0	2,170	1,580
6	1,200	131.7	2,070	1,580
24	1,800	194.0	1,950	1,580

The resultant flood hydrographs are shown in Figure 11.4 and 11.5 for both IFF conditions.

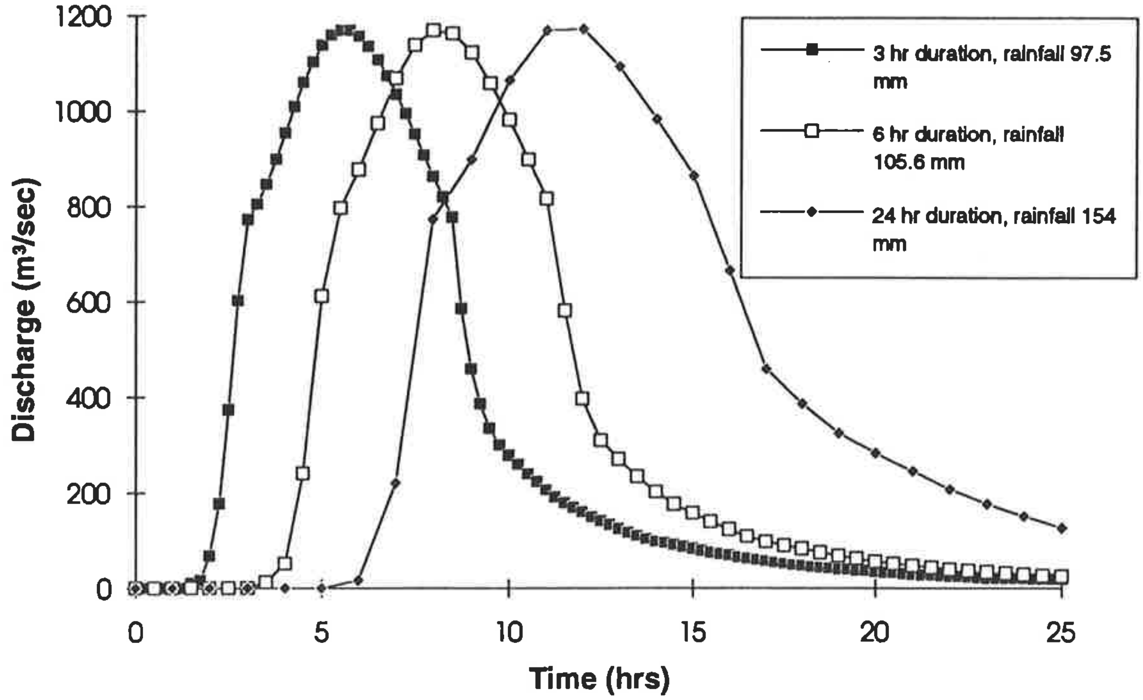


Figure 11.4 Outflow Hydrographs for Peak Discharge of 1,170 m³/sec

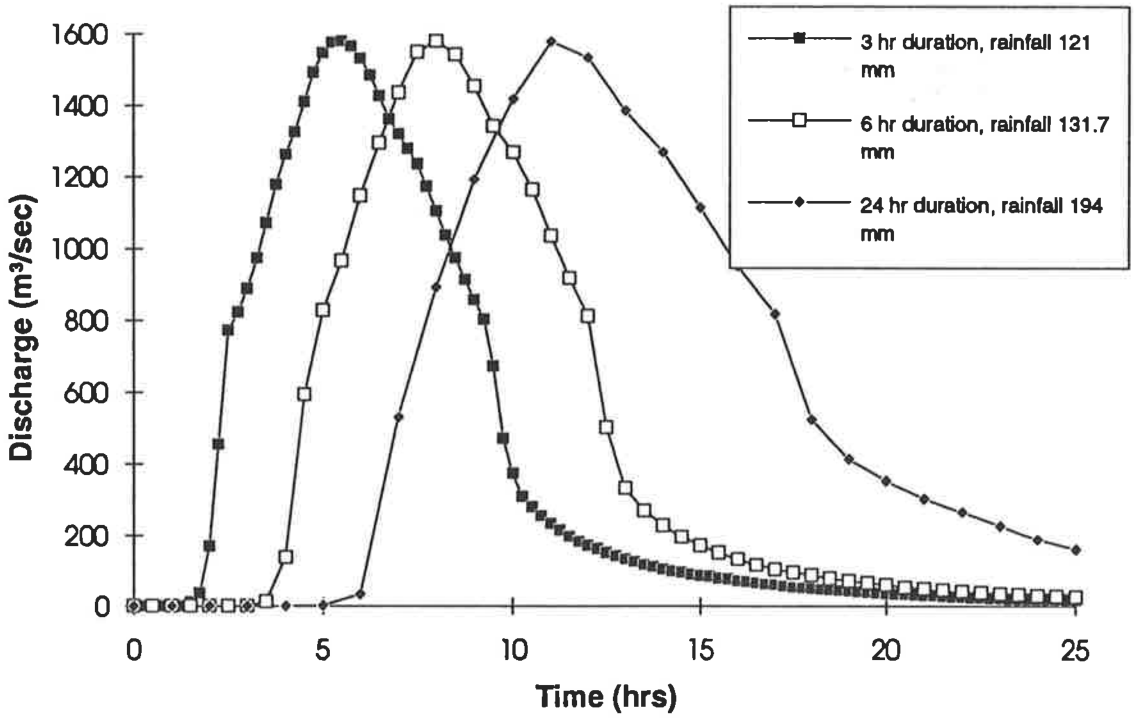


Figure 11.5 Outflow Hydrographs for Peak Discharge of 1,580 m³/sec

It was clear from deriving the IFF hydrographs that different combinations of storm duration and storm probabilities resulted in the same outflow peak. Green (1991) recommends that the most likely hydrograph should be adopted as the IFF hydrograph.

The shape of the hydrographs in Figures 11.4 and 11.5 reflect the operations of the gates established in the model. The adoption of different gate operating policies would have a significant effect on the shape of the IFF hydrographs.

11.5.1. Study of Mt Bold Spillway Gates

The Mt Bold Reservoir gate operations are critical to the determination of the IFF hydrograph. It is therefore recommended that a study into the gate settings and operations be undertaken and a set of procedures established for regular checking to ensure that the full gate opening of 3.8 metres can be obtained.

It is suggested that a study be undertaken to verify the hydraulic efficiency of the gates at the point of discontinuity.

Chapter 12

Conclusions and Recommendations

12.1. Probable Maximum Flood

The PMF is used as a standard of flood design where a very high level of safety is warranted. The PMF is not the maximum possible flood and therefore the PMF has a small probability of exceedance. The PMF should constitute a reasonable estimate. The inclusion of an excessive degree of conservatism is undesirable because the design procedures recommended by ANCOLD for the analysis of spillway capacity already contain safety factors. An unduly conservative estimate will distort the calculated probability of the PMF.

The best estimate of the outflow PMF for Mt Bold Reservoir was 9,300 m^3/sec from a 4 hour duration PMP. The inflow PMF was determined to be 10,200 m^3/sec from a 3 hour duration PMP. The inflow PMF corresponding to the outflow of 9,300 m^3/sec was 10,100 m^3/sec. The largest calculated inflow PMF of 10,200 m^3/sec was adopted as the inflow PMF because the frequency curve was considered to be an envelope of frequency curves for different durations.

The value adopted for the inflow PMF represents an increase of over 3 times the value determined by Kotwicki (1984). The limitations referred to in that report such as linear modelling, uniform rainfall and longer durations have been addressed in this study.

The best estimate of the PMF was 9,200 m^3/sec at Clarendon Weir and 7,900 m^3/sec at Old Noarlunga. The inflow and outflow PMFs calculated using the catchment to Old

Noarlunga were larger than those calculated using the catchment to Mt Bold Reservoir. This was because of a partial area effect. The estimates of the PMF for Mt Bold Reservoir calculated using the catchment to Old Noarlunga were not considered to be reasonable estimates.

The calculated outflow PMF overtops the dam wall by approximately 6.4 metres and overtops an adjoining saddle by approximately 5.5 metres.

The sensitivity of the PMF estimate to the choice of various parameters was tested. It was found that the PMF estimate was particularly sensitive to the model non-linearity and the choice of uniform or spatially varying rainfall. The use of uniform rainfall to determine the PMF was shown to underestimate the outflow PMF by approximately 10 percent for the critical duration. The use of a linear model, such as a unit hydrograph, underestimated the outflow PMF by approximately 33 percent.

Both high and low bounds of the PMF were calculated. These do not represent physical limits but rather are intended to be bounds in which reasonable PMF estimates should lie. The high and low bounds of the inflow PMF were 12,400 m^3/sec and 5,600 m^3/sec respectively. The high and low bounds of the outflow PMF were 10,800 m^3/sec and 5,200 m^3/sec respectively.

Although the upper bound of the outflow PMF is 22 percent above the adopted value, it should be noted that the peak stage height for the upper bound PMF is only 650 mm above the peak stage height for the adopted outflow PMF.

Based upon an analysis of the shape of the frequency curve and consideration of the effective transposition area used to determine the PMP, the PMF was assigned a probability of 1 in 10^6 (IEAust, 1987).

12.2. Probable Maximum Precipitation

Bulletin 51 published by the Bureau of Meteorology was used to determine PMPs for durations up to 6 hours. Estimates of longer duration PMPs that have been applied elsewhere in south-eastern Australia, were used to determine estimates of the PMP depths for longer durations. A requisition should be made to the Bureau of Meteorology for PMPs of durations longer than three hours. The estimates of the PMP are considered to be satisfactory because the critical duration for the Mt Bold outflow PMF is approximately four hours.

The analysis of the PMP considered only stationary thunderstorms. A storm moving slowly down the catchment would give extremely large peak flows.

12.3. Flood Frequency Analysis

The determination of extreme floods less than the PMF depends on the magnitude and probability of the PMF and the shape of the flood frequency curve.

The inflow flood frequency analysis was undertaken for the Mt Bold Reservoir using an extended data set. A number of different theoretical distributions were tested and the log Normal distribution was found to be the most appropriate.

Inflows were also calculated using design rainfalls and a RORB model of the catchment. The model was calibrated using sensitivity fitting. This is a simple extension of the parameter interaction curve and allows the optimum model parameters to be determined. It also examines the likely errors associated with a choice of model parameters.

The calibration of the model reinforced the importance of an adequate record of the rainfall. The quality of fit between the calculated and the recorded hydrographs improved as the number of pluviometers increased.

The inflow flood frequency curve calculated using the calibrated RORB model, the design rainfalls and the losses derived from calibration, was above that determined from the flood frequency analysis. The design rainfalls were reduced by 13 percent so that the calculated inflows corresponded to those obtained from the flood frequency analysis.

The outflow flood frequency was calculated considering the probability of both the inflow and the initial storage level. Storage probability distributions were determined for different periods of the year using synthetic data sets. A number of different storage distributions were tested and the choice of storage distribution was found to have a significant effect on the calculated outflow flood frequency. A composite distribution was chosen as the most appropriate.

A new spillway rating curve was developed for Mt Bold Reservoir which addressed the inadequacies in the existing rating. It is recommended that the maximum gate opening be reset to 3.8 metres and that the gate operation and official operating policy should be aligned. The examination of the spillway rating highlighted a discontinuity between non-gated and gated spillway flow. This discontinuity should be examined further as it has an important effect on the calculated IFF.

12.4. Imminent Failure Flood

Two levels were considered for the IFF:

1. An elevation of 248.42 metres. For a gate opening of 2.9 metres, this results in a flow of 915 m³/sec with an AEP of 1 in 750. If the gates are opened to 3.8 metres, a flow of 1,170 m³/sec results with an AEP of 1 in 1,300.

2. An elevation of 249.42 metres. If the spillway gates are opened to 2.9 metres, the flow is 1,300 m³/sec with a calculated AEP of 1 in 1,800. If the gates are opened to 3.8 metres, a flow of 1,580 m³/sec results with a calculated AEP of 1 in 2,400.

If the design life of the Mt Bold Dam is considered to be 100 years and the maximum gate openings are set to 3.8 metres, there is a 7.4 percent chance that the dam crest will be exceeded and a 4.1 percent chance that the dam crest will be exceeded by more than 1 metre, during the design life.

12.5. Further Research

This research has highlighted several inadequacies in the present design methods used in the estimation of extreme floods.

Further research should be undertaken to analyse the probability of the PMF. Greater guidance needs to be provided in what constitutes a 'reasonable' estimate of the PMF.

Flood frequency analysis is another area in which further research is required. Although much work has already been undertaken on the analysis of different distributions, procedures need to be established to determine the best theoretical distribution. This research should also examine the problem of analysing flows resulting from two distinct rainfall mechanisms.

The probability of outflows from reservoirs also needs to be studied. Further work is required on the joint probability of the inflow and the initial reservoir level.

This research has shown it may not be appropriate to use the losses obtained from the calibration of the hydrological model with the design rainfalls. The choice of design rainfalls and losses needs to be further researched.

Another area in which research is required is the effect of the number of pluviometers on the calibration of hydrological models. Poor fits were obtained between the calculated and recorded hydrographs for events for which there were only a few pluviometers operating.

The problem of spatial patterns for particular storms was not addressed in this study. Many of the events used for calibration of the model had a typical isohyetal pattern. It is felt that the inclusion of rainfall spatial patterns in general flood analysis is an area of research that should be progressed. The effect of a moving rainstorm should also be studied.

Chapter 13

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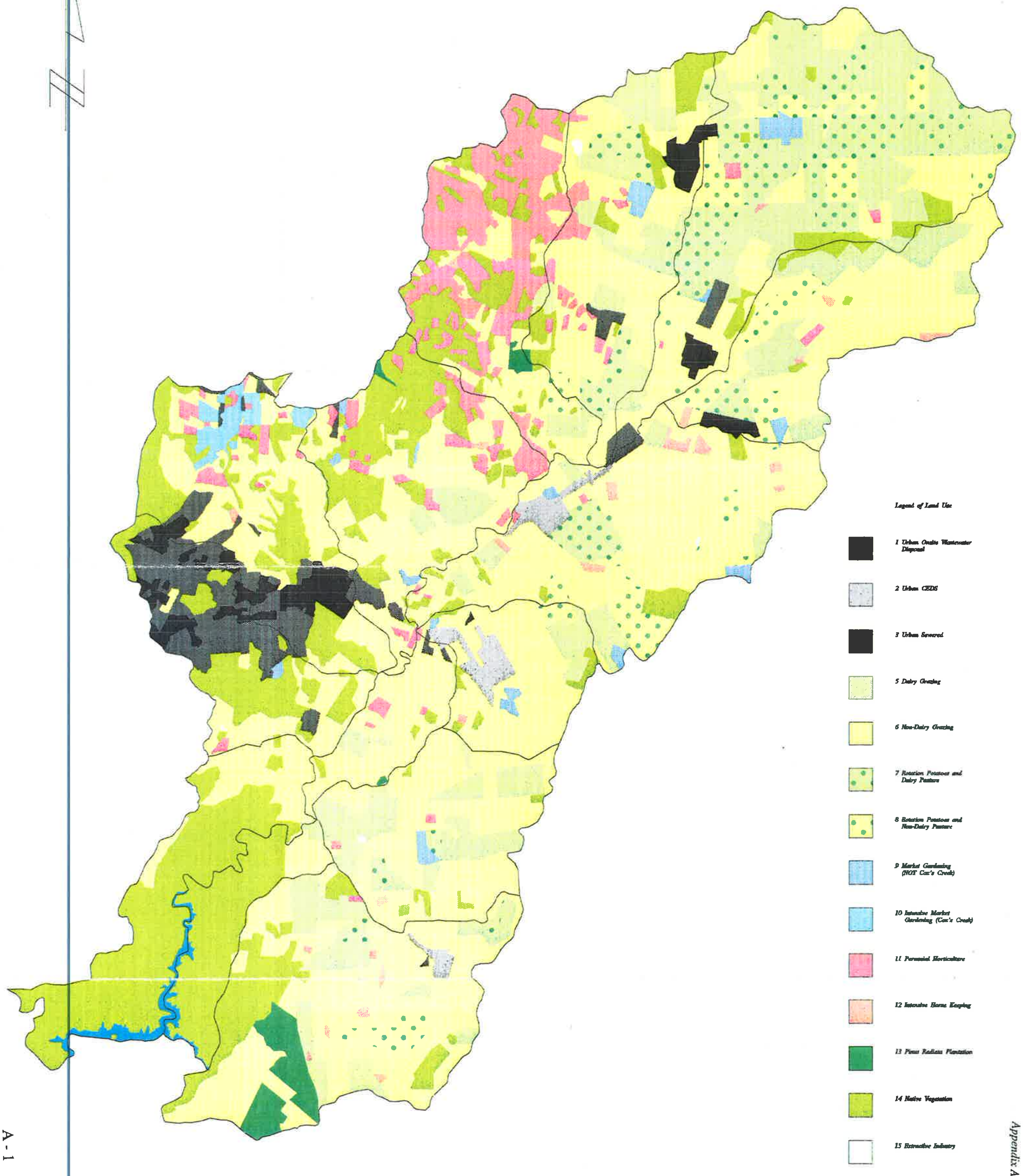
Appendix A

Land Use in the Mt Bold Catchment

C.M.S.S. Onkaparinga Catchment

LANDUSE



Appendix B
Instrumentation in the Onkaparinga Catchment

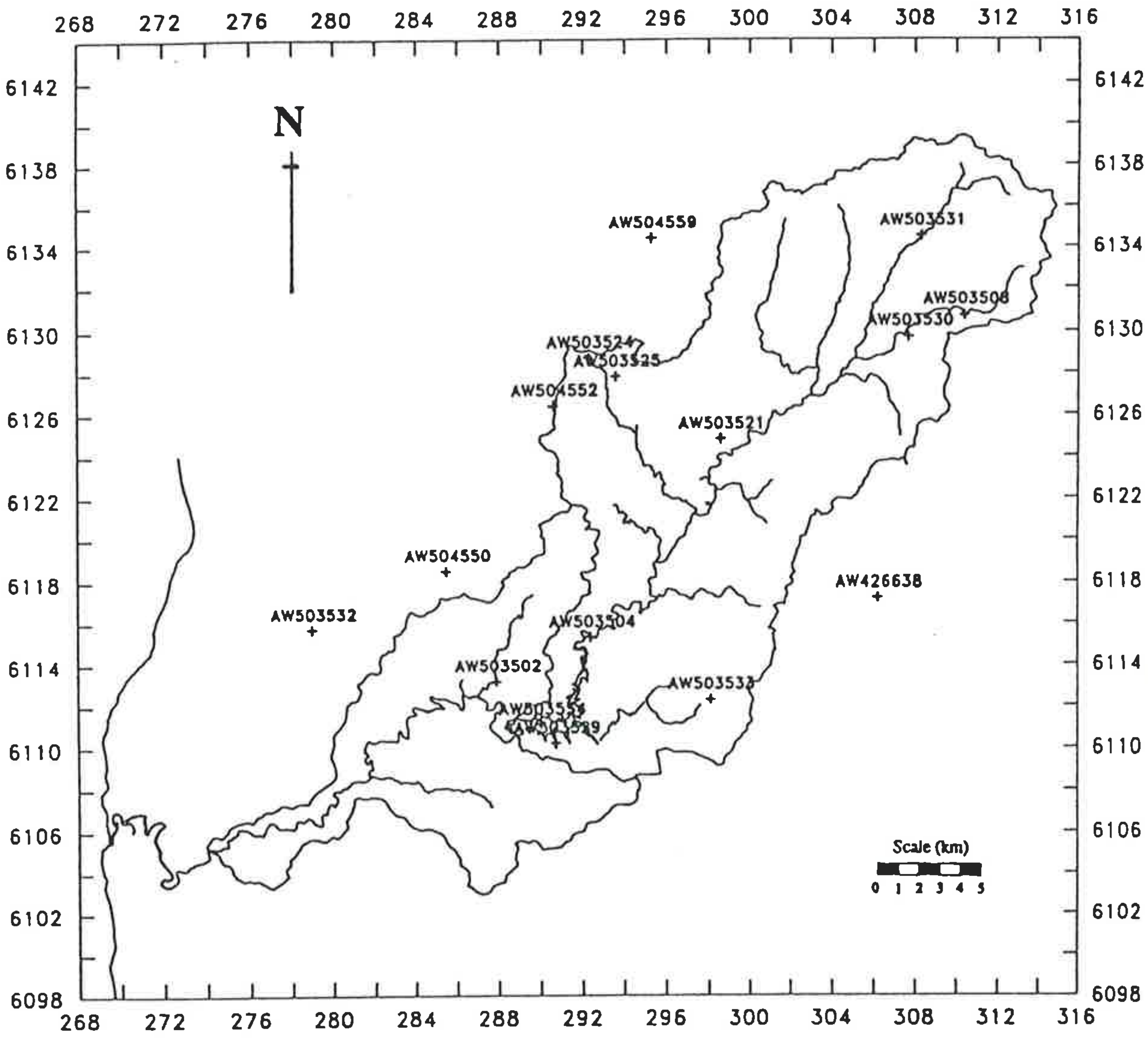


Figure B.1 Location of E&WS Pluviometers for the Onkaparinga Catchment

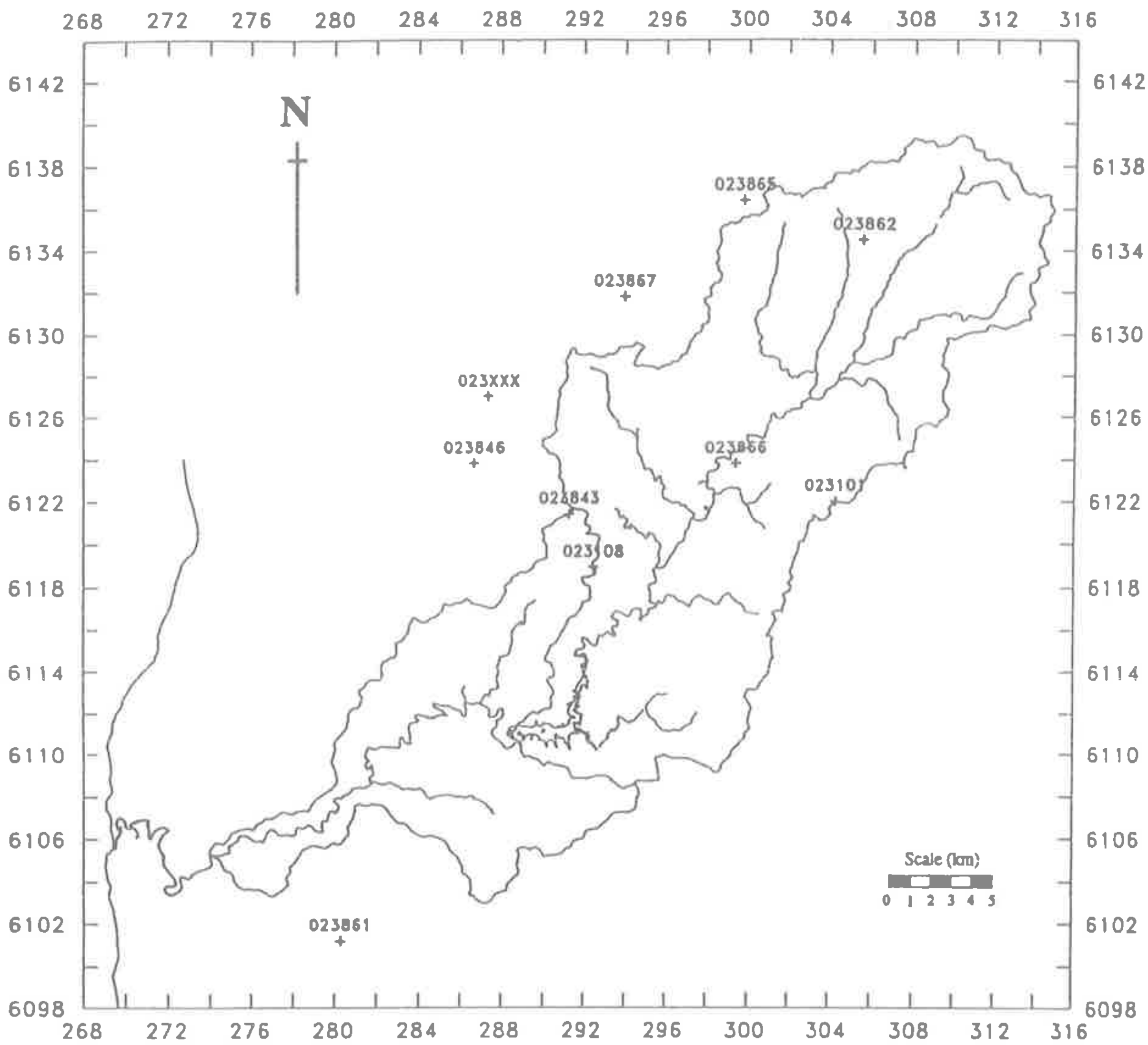


Figure B.2 Location of Bureau of Meteorology Pluviometers for the Onkaparinga Catchment

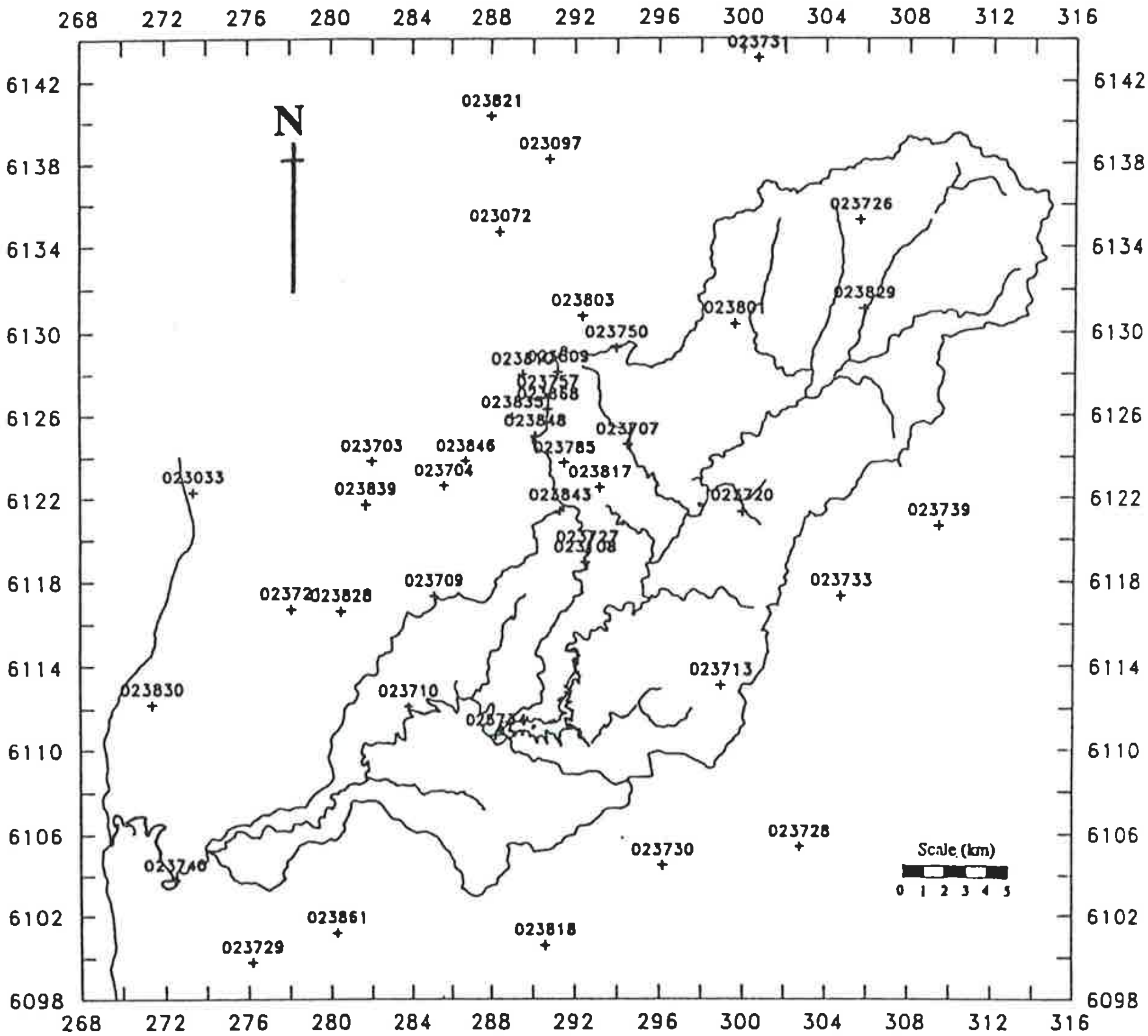


Figure B.3 Location of Bureau of Meteorology Daily Read Raingauges for the Onkaparinga Catchment

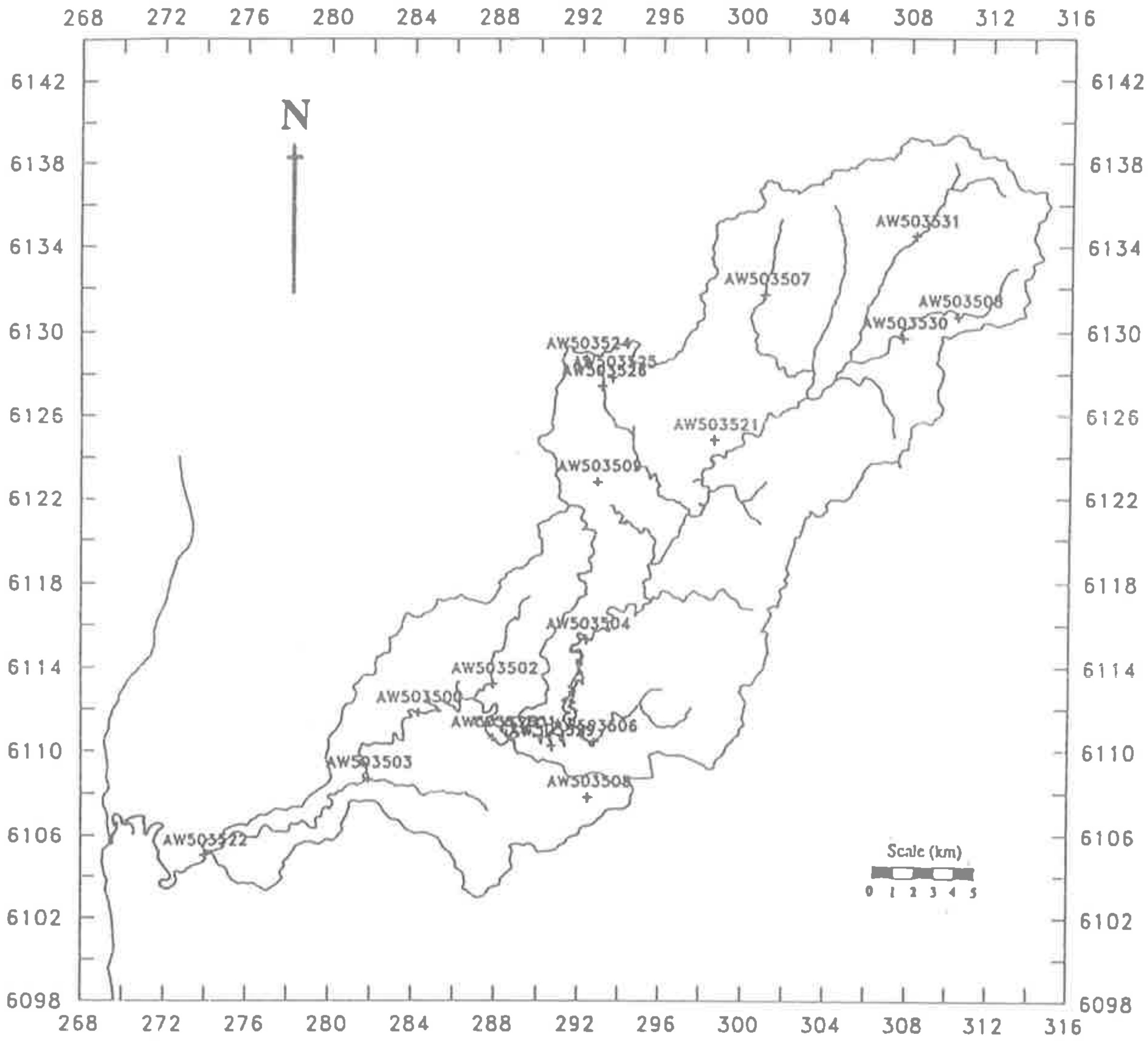


Figure B.4 Location of E&WS Gauging Stations for the Onkaparinga Catchment

Gaugings at Houlgraves Weir (AW503504)

Table B.1 indicates all of the gaugings that have occurred at Houlgraves Weir at flows greater than 50 m³/sec.

Table B.1 Gaugings at Houlgraves Weir Greater than 50 m³/sec

Reg No.	Date	Mean Ght (m)	Discharge (m³/sec)	Mean Vel. (m/sec)	Rise (mm/hr)	Fall (mm/hr)	Quality
47	25.06.81	5.213	57.700	0.750	-	23	G
48	25.06.81	5.107	52.000	0.729	-	65	G
49	26.06.81	5.409	65.500	0.827	292	-	F
57	02.07.81	6.069	115.000	1.150	-	30	G
58	02.07.81	6.030	103.000	1.056	-	46	G
59	04.07.81	5.353	64.200	0.850	-	133	G
60	04.07.81	5.182	57.500	0.801	-	135	G
63	24.07.81	6.057	118.600	1.247	-	69	G
64	24.07.81	5.816	95.300	1.056	-	109	G
65	24.07.81	5.499	73.900	0.947	-	144	G
66	14.08.81	5.946	108.450	1.186	-	73	G
67	14.08.81	5.624	92.026	1.054	-	124	G
68	14.08.81	5.424	77.591	0.996	-	54	G
77	08.09.83	5.494	77.500	0.916	-	248	F
78	08.09.83	5.131	57.400	0.768	-	179	F
81	21.08.84	5.492	76.900	0.920	-	200	G
83	24.06.87	6.701	184.439	1.374	-	149	F
84	24.06.87	6.446	157.185	1.253	-	167	G
85	24.06.87	6.100	123.494	1.144	-	185	G
86	24.06.87	5.667	89.834	0.959	-	186	G
87	30.08.92	7.662	326.542	1.654	-	639	F
88	30.08.92	7.035	228.068	1.506	-	480	F
89	30.08.92	6.592	172.022	1.288	-	348	G

Gauging Quality Codes:

G - Gauging was considered to be good

F - Gauging was considered to be fair

Gaugings at Old Noarlunga (AW503522)

Although there has been 58 gaugings recorded at Noarlunga gauging station, the majority of these have been at low flows. Table B.2 indicates all of the gaugings that have been greater than 1 m^3/sec.

The nature of the control and other factors pertinent to the site are discussed in Section 3.2.4.3.

Table B.2 Gaugings at Noarlunga that Have Exceeded 1 m^3/sec

Reg. No.	Date	Stage (m)	Discharge (m^3/sec)	Area (m^2)	Velocity (m/sec)
12	04/10/74	8.220	72.5	143	0.51
4	03/09/73	6.630	23.6	39.2	0.60
1	26/07/73	6.136	8.16	34.5	0.24
3	08/08/73	5.970	6.42	32.1	0.20
2	26/07/73	5.975	6.17	27.1	0.23
10	14/08/74	5.512	2.04	2.56	0.80
58	11/07/84	5.657	2.00	2.32	0.86
11	09/09/74	5.369	1.36	1.97	0.69

Appendix C

History of Flooding in the Onkaparinga River

Floods in the Onkaparinga River from 1896 to 1933

The following summary of peak flows in the Onkaparinga River was taken from E&WS (1933). This report summarised the recorded peak flows (in ft³/sec) in the Onkaparinga River prior to the construction of the Mt Bold Dam.

Table C.1 Floods in the Onkaparinga River from 1896 to 1933

Peak Flows (m^3/sec)	Dates
142 - 170	6/6/1901, 13/7/1904, 16/5/1909, 14/5/1917, 17/9/1925
170 - 198	14/6/1898, 16/7/1898, 30/8/1900, 12/6/1905, 24/8/1906, 19/8/1909, 6/8/1915, 21/8/1917
198 -227	28/9/1909, 13/6/1917
227 - 255	15/8/1900, 17/9/1903, 1/10/1909, 12/9/1917, 18/9/1921, 10/6/1923
255 - 283	9/9/1906, 22/9/1923
283 - 312	23/6/1915, 9/8/1920
312 - 340	19/7/1917

Major Point Rainfalls for South Australia

The major point rainfalls recorded in South Australia were collated by BC Tonkin and Associates (1985) using data from the Bureau of Meteorology. The storm at Lenswood on 30 August 1992 was included in the list.

Table C.2 Major Point Rainfalls in South Australia

Date	Month	Year	Station	Rainfall (mm)	
				24 hrs	48 hrs
13	Jan	1874	Mt. Gambier	91	157
13	May	1884	Baroota	158	166
12	Dec	1884	Kanmantoo	125	-
3	Apr	1889	Truro	146	166
17	Apr	1889	Stirling	208	241
5	Apr	1891	Baratta	174	206
31	May	1893	Wirrabers F.R.	-	212
7	Mar	1910	Wirrabers F.R.	173	-
13	Feb	1913	Nunong	175	186
12	Nov	1920	Cadelga	217	217
1	Mar	1921	Wilmington	181	214
6	Feb	1925	North Adelaide	164	164
19	Feb	1938	Cooper Pedy	165	165
17	Apr	1938	Houghton	145	145
25	Jan	1941	Encounter Bay	226	350
13	Apr	1941	Marree	167	215
18	Feb	1946	Stansbury	222	228
27	Mar	1947	Parakylia	169	179
11	Apr	1948	Torrens Vale	151	168
5	Mar	1949	Clifton Hills	213	230
2	Feb	1950	Commodore	220	239
17	Mar	1950	Wooltana	188	188
13	Feb	1955	Hawker	189	217
3	Mar	1955	Clifton Hills	215	217
30	Mar	1956	Arkaroola	190	190
4	Dec	1966	Ungarra P.O.	109	172
30	Apr	1968	Pt. Lincoln	99	165
9	Feb	1969	Stirling	146	-
10	Apr	1970	Mt. Barker P.O.	131	-
16	July	1971	St. Kitts	129	130
6	Feb	1973	Brinkworth	114	153
14	Jan	1974	Lameroo	170	174
31	Jan	1974	Innaminka	189	330
25	Oct	1975	Wirrabera F.R.	-	214

13	Dec	1975	Oakbank	165	166
9	Feb	1976	Oodnadatta	200	246
28	Nov	1977	Gordon	138	173
4	June	1978	Wirrabera F.R.	222	230
17	Jan	1979	Clifton Hills	198	293
31	Dec	1979	Pt. Pirie	125	143
26	Jan	1981	Buckleboo	191	-
2	Mar	1983	Dutton	330	-
23	Mar	1983	Caroline F.R.	-	174
14	Jan	1984	Lake Eyre	193	206
30	Aug	1992	Lenswood	104	139

Summary of Peak Flows at Houlgraves Weir

AW503504

Peak flows at Houlgraves Weir were extracted using HYEVENT. The minimum value was 75 m³/sec and the minimum separation between events was 3 days. The period of record was from 17 April 1973 to 20 December 1992.

Table C.3 Peak Flows at Houlgraves Weir

Time _ Date	Stage (m)	Discharge (m³/sec)
23:14_01/09/1973	6.846	202.784
00:27_04/10/1974	6.104	122.236
22:57_05/07/1978	6.012	114.043
03:23_06/09/1979	5.819	98.361
23:49_11/09/1979	6.034	115.994
07:28_12/10/1979	5.939	107.938
18:06_26/06/1981	7.367	276.802
21:19_03/07/1981	6.477	159.165
10:58_24/07/1981	6.089	120.905
20:50_03/08/1981	5.816	98.077
14:02_08/08/1981	6.237	134.746
10:32_14/08/1981	6.051	117.466
04:09_25/08/1983	5.689	88.546
08:19_08/09/1983	6.100	121.880
08:52_21/08/1984	5.837	99.771
07:23_24/06/1987	6.988	221.326
16:43_15/07/1987	6.407	151.816
05:27_24/05/1988	6.494	160.996
06:26_15/08/1990	5.599	82.141
18:46_15/09/1991	6.643	177.965
11:40_30/08/1992	8.210	431.813
05:09_16/09/1992	5.793	96.326
20:57_08/10/1992	6.769	193.166
21:46_19/12/1992	6.710	185.944

Total Missing Data: 7762 Minutes (5.39 Days)

Peak Flows at Clarendon Weir

AW503500

Peak flows at Clarendon Weir were extracted using HYEVENT. The minimum value was 75 m³/sec and the minimum separation between events was 3 days. The period of record was from 19 September 1937 to 5 January 1993.

Table C.4 Peak Flows at Clarendon Weir

Time _ Date	Stage (m)	Discharge (m³/sec)
18:03_30/08/1939	0.994	98.760
11:06_29/06/1942	1.143	120.286
10:45_04/07/1942	1.067	108.977
18:03_12/08/1942	1.542	191.359
07:01_01/09/1942	0.942	91.770
12:07_16/09/1942	1.167	124.011
09:04_08/08/1943	1.073	109.818
22:48_31/07/1946	0.904	86.969
06:44_16/07/1951	1.463	175.719
00:19_03/09/1953	0.853	80.702
22:44_23/06/1955	1.137	119.444
21:21_17/06/1956	0.974	96.016
15:49_28/06/1956	1.521	187.080
01:27_26/07/1956	1.025	103.098
15:28_15/08/1963	1.073	109.907
22:02_25/08/1963	1.294	144.860
12:01_12/09/1963	0.883	84.254
09:53_16/09/1964	0.838	78.857
10:51_08/10/1964	1.012	101.221
20:06_18/11/1964	1.179	125.889
18:35_09/08/1968	1.620	192.181
16:19_07/10/1968	1.029	96.755
04:38_26/10/1968	0.947	86.138
09:43_02/06/1971	3.954	81.084
18:17_10/08/1971	4.100	99.874
12:53_14/08/1971	4.036	91.307
05:05_29/08/1971	5.051	261.888
20:39_26/09/1971	4.321	132.919
00:16_02/09/1973	4.878	228.850
23:29_03/10/1974	1.215	126.378
18:43_24/10/1974	0.916	82.422
10:34_24/10/1975	1.060	102.891
05:11_12/09/1979	0.930	84.260

10:39_06/10/1979	0.861	75.109
10:30_12/10/1979	1.195	123.271
14:24_24/07/1981	1.143	115.318
21:49_03/08/1981	0.973	90.196
13:47_08/08/1981	1.149	116.267
10:48_14/08/1981	0.990	92.624
12:08_08/09/1983	0.969	89.715
20:13_12/09/1986	0.882	77.902
23:19_18/07/1987	0.924	83.535
16:36_18/09/1991	1.015	96.271
05:13_16/09/1992	0.984	91.787
00:26_09/10/1992	1.298	139.644
19:24_18/12/1992	0.945	86.324

Total Missing Data: 1062718 Minutes (2.02 Years)

Peak Flows at Old Noarlunga

AW503522

Peak flows at Old Noarlunga were extracted using HYEVENT. The minimum value was 75 m³/sec and the minimum separation between events was 3 days. The period of record was from 27 June 1973 to 15 February 1988.

Table C.4 Peak Flows at Clarendon Weir

Time _ Date	Stage (m)	Discharge (m³/sec)
02:44_02/09/1973	10.287	201.826
02:07_04/10/1974	9.047	110.095
12:26_24/10/1975	8.738	92.225
07:06_12/09/1979	8.628	86.326
12:34_12/10/1979	9.126	114.976
16:53_24/07/1981	8.998	107.145
00:47_04/08/1981	8.596	84.665
15:56_08/08/1981	9.049	110.244
13:46_14/08/1981	8.656	87.824
14:20_08/09/1983	8.650	87.487
23:08_12/09/1986	8.494	79.464

Total Missing Data: 279142 Minutes (193.85 Days)

Appendix D

1992 Floods in the Onkaparinga River

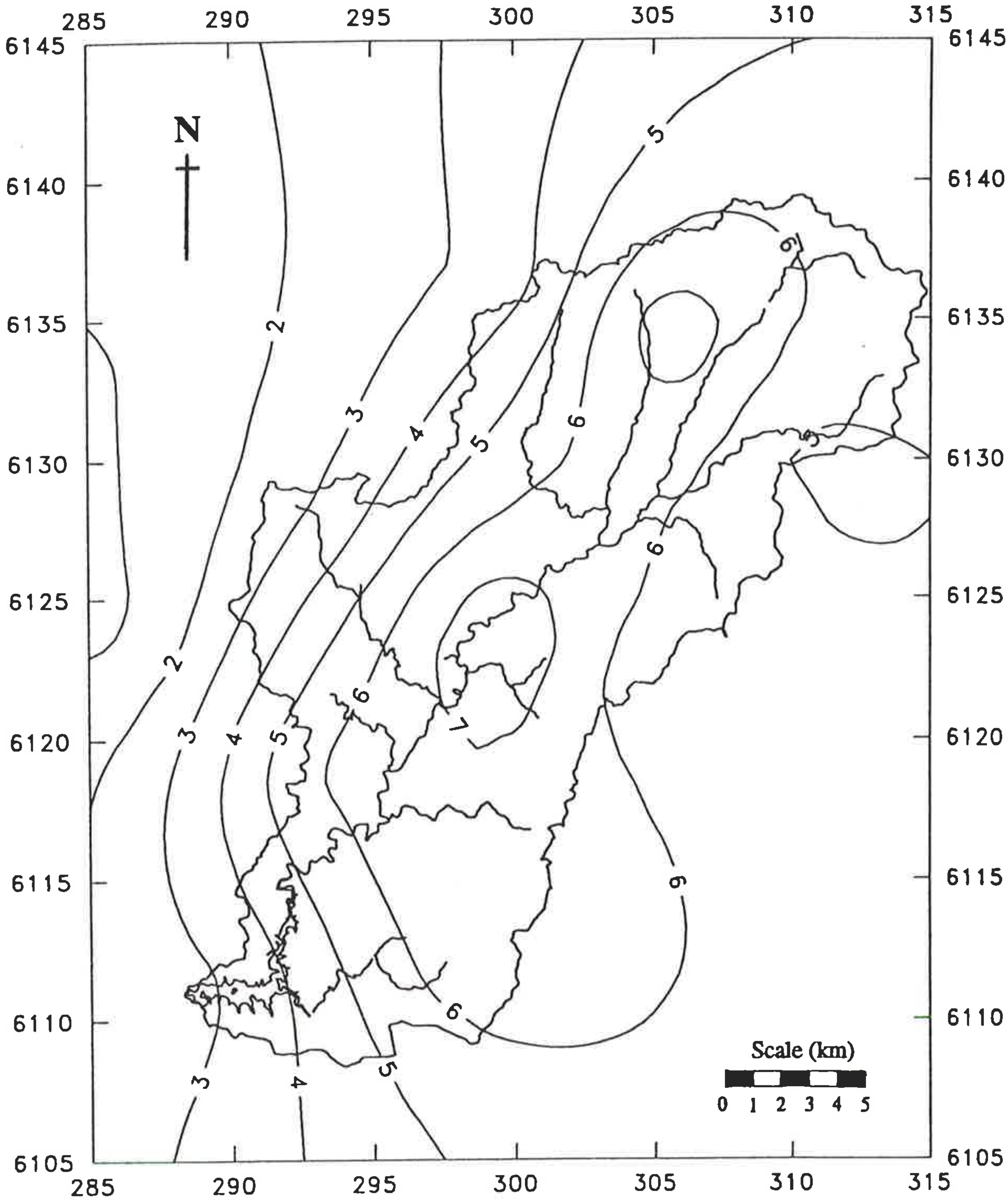


Figure D.1 2 hour Rainfall (mm) to 0200 hours 30 August 1992

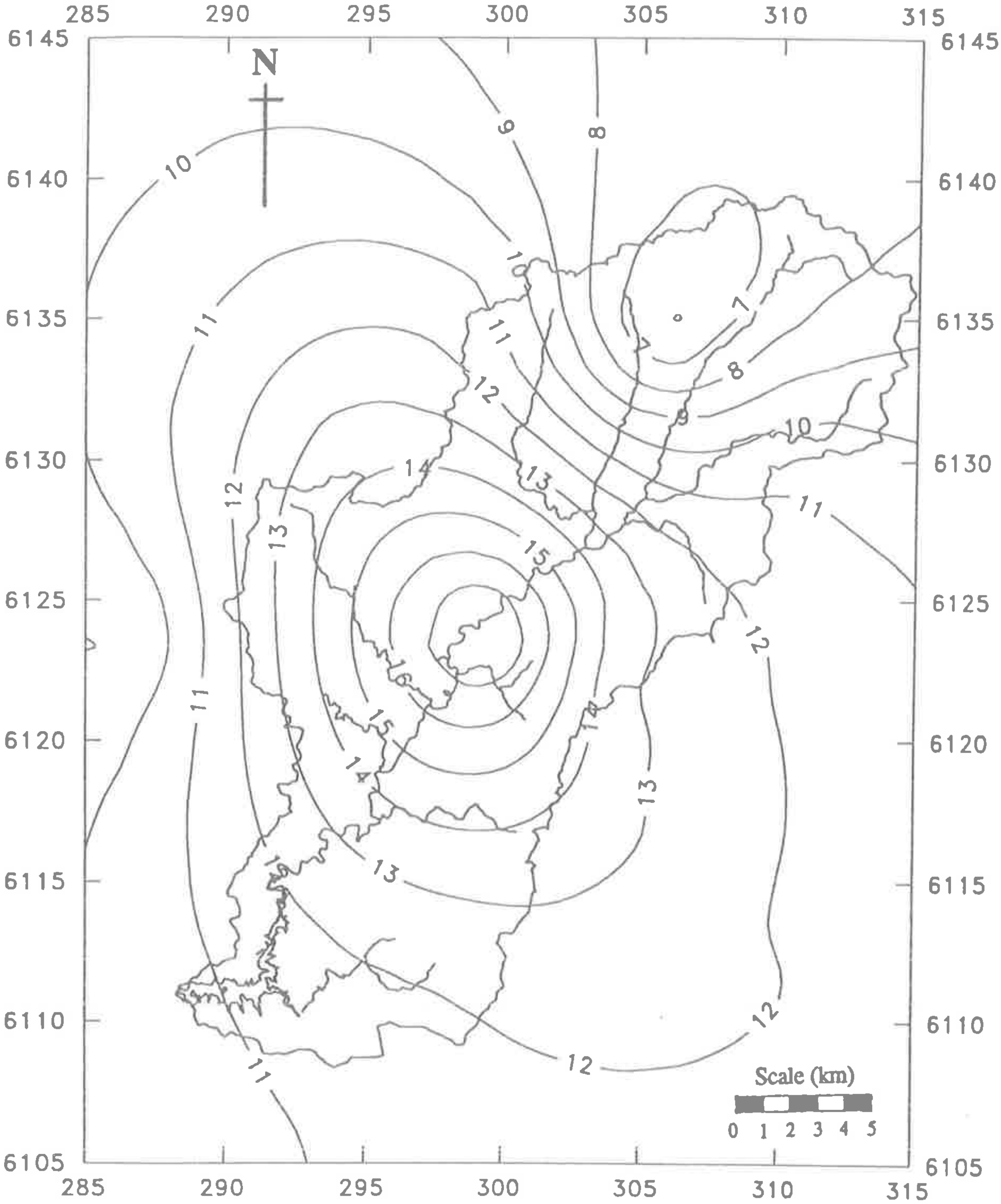


Figure D.2 2 hour Rainfall (mm) to 0400 hours 30 August 1992

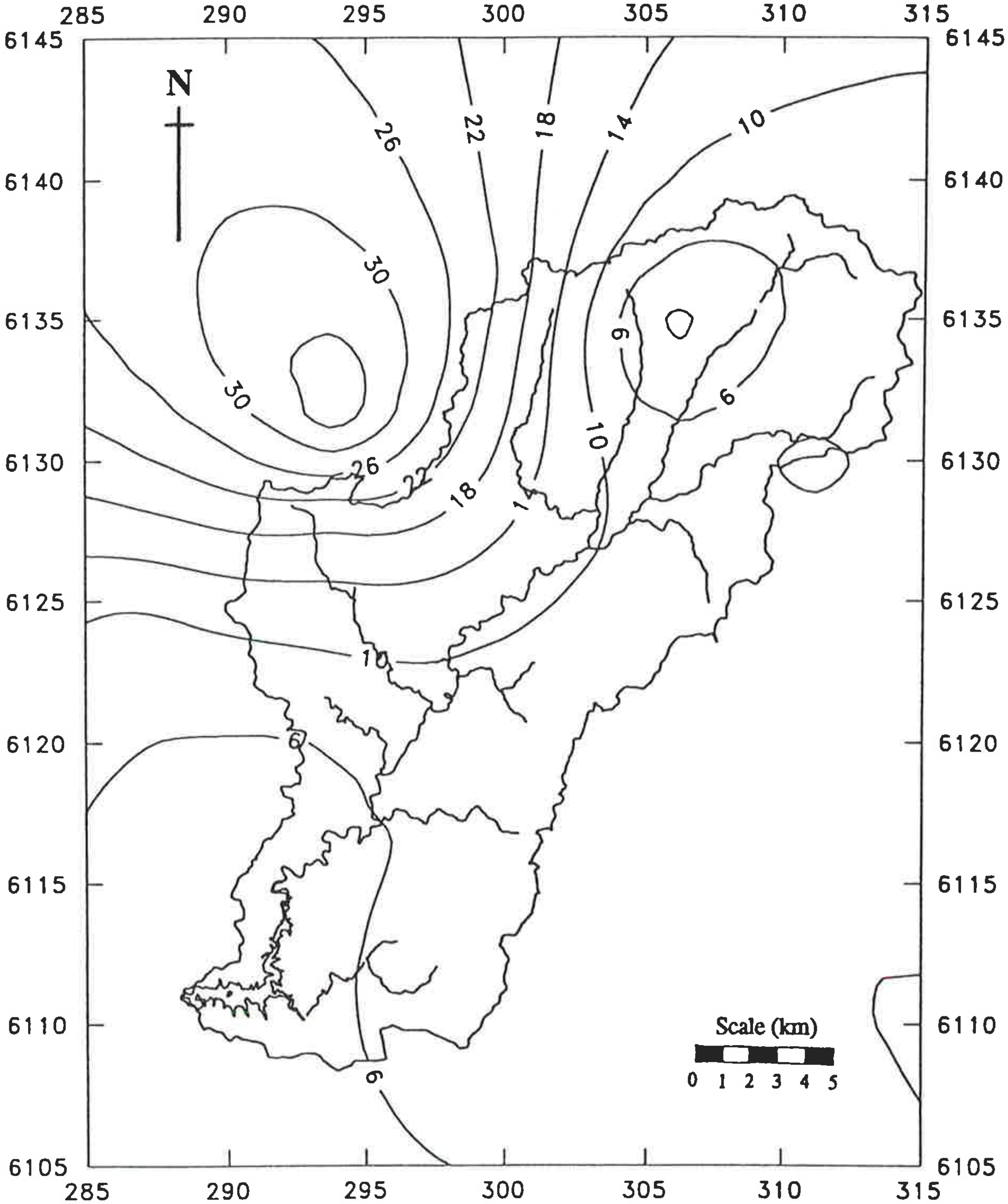


Figure D.3 2 hour Rainfall (mm) to 0600 hours 30 August 1992

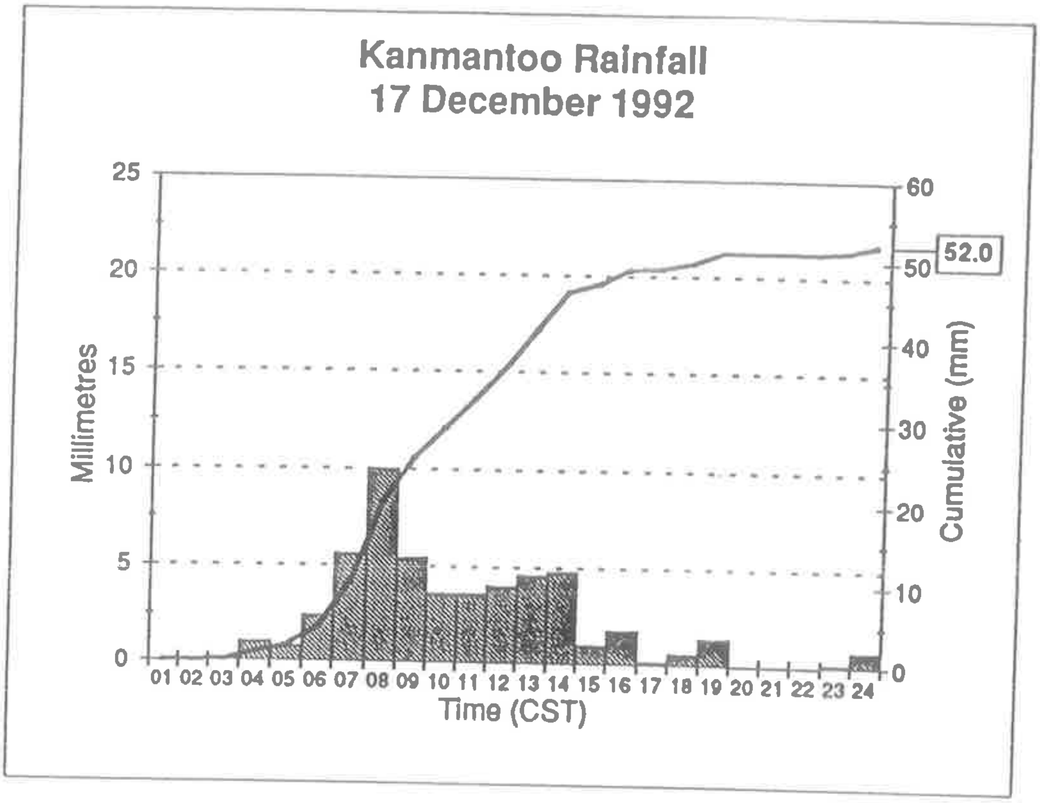


Figure D.4a Kanmantoo Pluviometer 17 December 1992

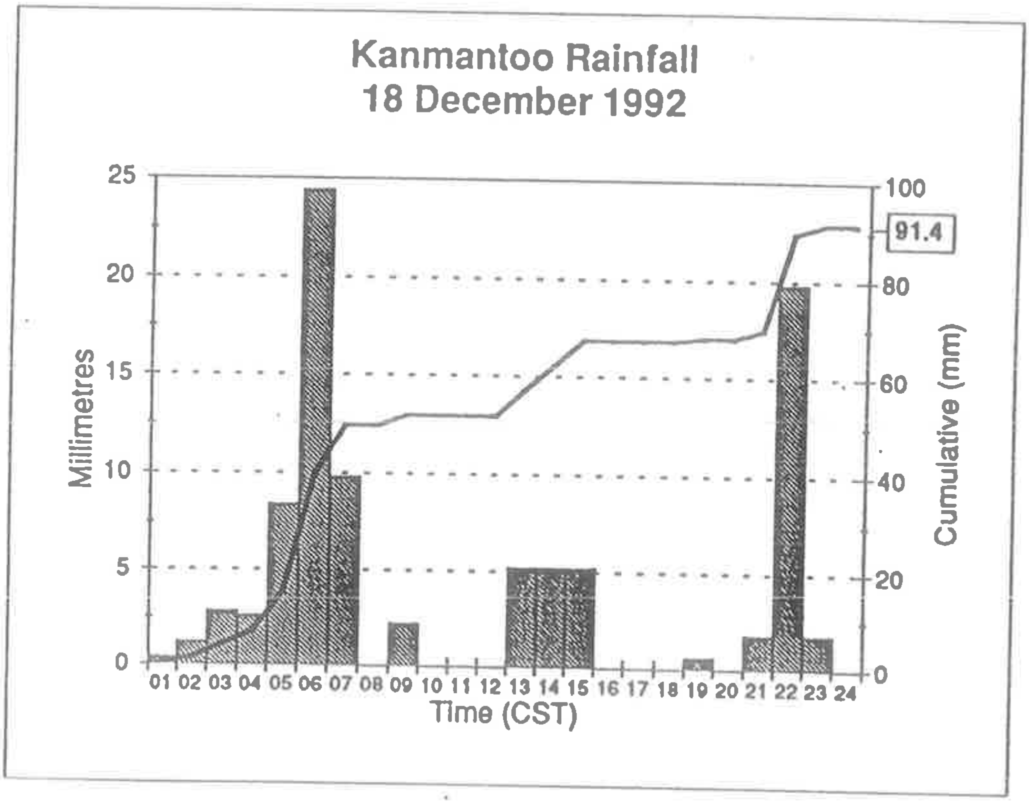


Figure D.4b Kanmantoo Pluviometer 18 December 1992

Appendix E
Mt Bold Reservoir Spillway

Mt. Bold Dam Spillway Gate Operation

The following operating procedure for the Mt Bold Gates is the current recommended operating policy of the E&WS (Parsons, 1993, pers. comm.). It differs from that found in Kotwicki (1984) and Design Services (1982).

Reservoir Level Rising

1. The gates shall be kept in the lowered position until the water level in the reservoir rises to within 300 mm of the top of the gates (GH 41.10 m).
2. Gates 4 and 5 shall then each be raised 1 increment of 100 mm and readings taken of the water level indicator at frequent intervals until it is known whether the level of the reservoir is rising or falling.
3. When the reservoir level has risen 100 mm (GH 41.20 m) gates 3 and 6 shall be raised 100 mm, and further readings taken from the water level indicator.
4. When the reservoir level has risen another 100 mm (GH 41.30 m) gates 2 and 7 shall be raised 100 mm, and further readings shall be taken from the water level indicator.
5. If the reservoir level continues to rise, gates 1 and 8 shall be raised 100 mm and the gates shall continue to be raised in this sequence in steps of 100 mm, if the reservoir level still continues to rise.
6. With large inflows it may be necessary to use steps of more than 1 increment of 100 mm but after each successive opening, readings must be taken of the water level indicator.

Reservoir Level Falling

7. If after step 3, the water level in the reservoir starts to fall, gates 4 and 5 shall each be closed.
8. If at any stage the reservoir level begins to fall, the gates shall be closed in steps of 100 mm in the reverse sequence in which they were opened until the reservoir level begins to rise again.

Calculation of the Total Outflow from Mt Bold Reservoir

Design Head	Hd	3.8 m	
GH = EL - 205.50			
		EL	GH
Spillway Crest	CE	244.004	38.504
Full Supply Level	FS	246.900	41.400
Dam Crest	DC	248.424	42.924
Max Permis. Flood	MPF	247.970	42.470
Width of a Spillway	b	7.315 m	
Horizontal Contraction Coefficients:			
Pier	Kp	0.02	
Abutments	Ka	0.1	

Gauge Height	GH	41.41 m
Head	Ho	2.906 m
	Xo	0.76474
Discharge Coeff.	Cd	0.47671

Gate	1	2	3	4	5	6	7	8
Gate Opening (mm)	3800	3800	3800	3800	3800	3800	3800	3800
At Design Head:								
Effective Width (m)	6.859	7.163	7.163	7.163	7.163	7.163	7.163	6.859
Discharge for Hd	111.4	116.3	116.3	116.3	116.3	116.3	116.3	111.4
At Given Head:								
Effective Width (m)	6.966	7.199	7.199	7.199	7.199	7.199	7.199	6.966
Z	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
Discharge for Ho	72.9	75.3	75.3	75.3	75.3	75.3	75.3	72.9

Valves

Valve	OLD1	OLD2	TEMP	NEW1	NEW2	
Opening (no. turns)			-	-	-	(The two OLD valves)
(%)	-	-		50		(TEMP & NEW valves)
Discharge for GH				4.65		(Ebsary (1987))

Total Discharge from Spillway:	597.6 cumecs
Total Discharge from Outlet Valves:	4.7 cumecs
Total Outflow from Mt Bold Res:	**602.2 cumecs**

Figure E.1 Spreadsheet which Calculates the Total Outflow from Mt Bold Reservoir

Appendix F

Design Rainfalls for the Onkaparinga Catchment

Design Rainfalls for the Onkaparinga Catchment to Mt Bold Reservoir

Table F.1 30 minute Duration (5 minute increment)

ARI (years)	Rainfall Intensity (mm/hr)	Total Rainfall (mm)	Areal Reduction Factor	Design Rainfall (mm)	Temporal Pattern (mm)					
					1	2	3	4	5	6
5	34.6	17.3	0.6	10.4	1.7	2.6	3.4	0.9	1.1	0.6
10	40.4	20.2	0.6	12.1	1.9	3.0	4.0	1.1	1.3	0.7
20	48.4	24.2	0.6	14.5	2.3	3.6	4.8	1.3	1.6	0.9
50	60.0	30.0	0.6	18.0	2.9	4.3	5.4	1.8	2.2	1.4
100	69.7	34.9	0.6	20.9	3.3	5.0	6.3	2.1	2.5	1.7
200	80.4	40.2	0.6	24.1	3.9	5.8	7.2	2.4	2.9	1.9
500	96.3	48.2	0.6	28.9	4.6	6.9	8.7	2.9	3.5	2.3

Table F.2 1 hour Duration (5 minute increment)

ARI (years)	Rainfall Intensity (mm/hr)	Total Rainfall (mm)	Areal Reduction Factor	Design Rainfall (mm)	Temporal Pattern (mm)											
					1	2	3	4	5	6	7	8	9	10	11	12
5	22.6	22.6	0.8	16.9	0.7	1.2	2.8	2.0	3.9	1.7	1.5	1.0	0.8	0.5	0.4	0.3
10	26.2	26.2	0.8	19.6	0.8	1.4	3.3	2.4	4.6	2.0	1.7	1.1	0.9	0.6	0.5	0.4
20	31.1	31.1	0.8	23.3	0.9	1.6	3.9	2.8	5.4	2.4	2.1	1.3	1.1	0.7	0.6	0.4
50	38.3	38.3	0.8	28.7	1.2	2.1	4.6	3.3	6.2	2.9	2.6	1.7	1.5	1.0	0.9	0.7
100	44.2	44.2	0.8	33.2	1.4	2.4	5.3	3.8	7.2	3.3	3.0	2.0	1.7	1.2	1.0	0.8
200	50.8	50.8	0.8	38.1	1.6	2.8	6.1	4.4	8.3	3.8	3.4	2.3	2.0	1.3	1.1	0.9
500	60.4	60.4	0.8	45.3	1.9	3.3	7.3	5.3	9.8	4.5	4.1	2.7	2.4	1.6	1.4	1.0

Table F.3 2 hour Duration (5 minute increment)

ARI (years)	Rainfall Intensity (mm/hr)	Total Rainfall (mm)	Areal Reduction Factor	Design Rainfall (mm)	Temporal Pattern (mm)																							
					1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
5	14.9	29.8	0.8	25.0	0.4	3.1	4.1	1.5	0.9	2.5	1.7	2.1	1.3	0.7	0.8	1.0	1.1	0.7	0.5	0.6	0.3	0.5	0.4	0.3	0.2	0.2	0.2	0.1
10	17.1	34.2	0.8	28.7	0.5	3.6	4.7	1.7	1.0	2.8	2.0	2.4	1.5	0.8	0.9	1.1	1.3	0.7	0.5	0.7	0.4	0.6	0.4	0.3	0.3	0.2	0.2	0.1
20	20.1	40.3	0.8	33.8	0.6	4.2	5.5	2.0	1.2	3.3	2.3	2.8	1.7	0.9	1.1	1.4	1.5	0.9	0.6	0.8	0.4	0.7	0.5	0.4	0.3	0.3	0.2	0.1
50	24.5	48.9	0.8	41.1	0.7	5.1	6.6	2.3	1.4	4.0	2.8	3.4	2.0	1.1	1.3	1.6	1.8	1.1	0.8	0.9	0.6	0.9	0.7	0.5	0.5	0.4	0.3	0.2
100	28.1	56.1	0.8	47.1	0.8	5.8	7.6	2.7	1.6	4.6	3.2	3.9	2.3	1.3	1.5	1.8	2.1	1.2	0.9	1.1	0.7	1.0	0.8	0.6	0.5	0.5	0.3	0.2
200	32.0	64.0	0.8	53.7	1.0	6.6	8.6	3.1	1.8	5.3	3.7	4.5	2.6	1.5	1.7	2.1	2.4	1.4	1.1	1.2	0.8	1.1	0.9	0.7	0.6	0.5	0.4	0.3
500	37.7	75.4	0.8	63.3	1.1	7.8	10.2	3.6	2.2	6.2	4.3	5.3	3.1	1.7	2.0	2.5	2.8	1.6	1.3	1.5	0.9	1.3	1.1	0.8	0.7	0.6	0.4	0.3

Table F.4 3 hour Duration (15 minute increment)

ARI (years)	Rainfall Intensity (mm/hr)	Total Rainfall (mm)	Areal Reduction Factor	Design Rainfall (mm)	Temporal Pattern (mm)											
					1	2	3	4	5	6	7	8	9	10	11	12
5	11.6	34.9	0.9	30.4	1.9	8.3	5.5	3.8	2.9	2.3	1.5	1.3	1.0	0.9	0.6	0.4
10	13.3	39.8	0.9	34.6	2.1	9.4	6.3	4.4	3.3	2.6	1.7	1.5	1.2	1.0	0.7	0.5
20	15.5	46.5	0.9	40.5	2.5	11.0	7.4	5.1	3.8	3.0	2.0	1.7	1.4	1.1	0.9	0.6
50	18.7	56.2	0.9	48.9	3.0	13.1	8.8	6.2	4.6	3.7	2.4	2.1	1.7	1.4	1.1	0.7
100	21.4	64.1	0.9	55.8	3.4	15.0	10.0	7.0	5.3	4.2	2.8	2.4	2.0	1.6	1.2	0.8
200	24.2	72.7	0.9	63.3	3.9	17.0	11.4	8.0	6.0	4.7	3.2	2.7	2.2	1.8	1.4	0.9
500	28.4	85.2	0.9	74.1	4.5	19.9	13.3	9.3	7.0	5.6	3.7	3.2	2.6	2.1	1.6	1.1

Table F.5 6 hour Duration (30 minute increment)

ARI (years)	Rainfall Intensity (mm/hr)	Total Rainfall (mm)	Areal Reduction Factor	Design Rainfall (mm)	Temporal Pattern (mm)											
					1	2	3	4	5	6	7	8	9	10	11	12
5	7.6	45.6	0.9	41.0	1.7	3.2	11.2	7.1	5.1	4.0	2.6	2.1	1.4	1.1	0.8	0.5
10	8.6	51.3	0.9	46.2	1.9	3.6	12.6	8.0	5.8	4.5	3.0	2.4	1.6	1.2	0.9	0.6
20	9.9	59.5	0.9	53.5	2.2	4.2	14.6	9.3	6.7	5.2	3.4	2.8	1.9	1.4	1.1	0.7
50	11.8	70.9	0.9	63.9	2.7	5.0	17.2	11.0	8.0	6.3	4.1	3.3	2.3	1.8	1.3	0.9
100	13.4	80.3	0.9	72.3	3.1	5.6	19.4	12.4	9.0	7.1	4.6	3.8	2.6	2.0	1.5	1.0
200	15.1	90.3	0.9	81.3	3.5	6.3	21.9	14.0	10.2	8.0	5.2	4.2	2.9	2.3	1.7	1.1
500	17.5	104.8	0.9	94.3	4.1	7.4	25.4	16.2	11.8	9.2	6.0	4.9	3.4	2.6	2.0	1.3

Table F.6 12 hour Duration (30 minute increment)

ARI (years)	Rainfall Intensity (mm/hr)	Total Rainfall (mm)	Areal Reduction Factor	Design Rainfall (mm)	Temporal Pattern (mm)																							
					1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
5	5.0	59.7	0.9	55.5	4.1	13.0	1.5	8.2	5.6	3.3	2.4	2.6	2.1	1.6	1.4	0.9	1.2	1.8	1.1	0.8	0.7	0.7	0.5	0.6	0.4	0.3	0.4	0.3
10	5.5	66.4	0.9	61.8	4.6	14.5	1.7	9.1	6.2	3.6	2.7	2.8	2.3	1.8	1.5	1.1	1.4	2.0	1.2	0.9	0.8	0.7	0.6	0.6	0.4	0.4	0.4	0.3
20	6.4	76.3	0.9	70.9	5.2	16.6	1.9	10.5	7.2	4.2	3.1	3.3	2.7	2.1	1.8	1.2	1.6	2.3	1.3	1.1	0.9	0.9	0.6	0.7	0.5	0.4	0.5	0.4
50	7.5	89.9	0.9	83.6	6.1	19.4	2.2	12.3	8.4	4.8	3.7	3.8	3.2	2.3	2.0	1.4	1.8	2.8	1.6	1.3	1.2	1.1	0.8	0.9	0.7	0.6	0.8	0.5
100	8.4	100.9	0.9	93.8	6.8	21.8	2.4	13.8	9.4	5.4	4.1	4.3	3.6	2.6	2.3	1.6	2.0	3.1	1.8	1.5	1.3	1.2	0.9	1.0	0.8	0.7	0.8	0.6
200	9.4	112.6	0.9	104.7	7.6	24.3	2.7	15.4	10.5	6.1	4.6	4.8	4.0	2.9	2.5	1.8	2.2	3.5	2.0	1.7	1.5	1.4	1.0	1.2	0.8	0.7	0.9	0.6
500	10.8	129.3	0.9	120.3	8.8	27.9	3.1	17.7	12.0	7.0	5.3	5.5	4.6	3.4	2.9	2.0	2.5	4.0	2.3	1.9	1.7	1.6	1.2	1.3	1.0	0.8	1.1	0.7

Table F.7 24 hour Duration (1 hour increment)

ARI (years)	Rainfall Intensity (mm/hr)	Total Rainfall (mm)	Areal Reduction Factor	Design Rainfall (mm)	Temporal Pattern (mm) 1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
5	3.1	75.5	0.9	71.0	8.0	17.1	6.2	10.8	3.8	4.8	3.1	2.7	2.6	2.1	1.5	1.3	1.8	1.1	0.6	0.9	0.8	0.5	0.4	0.3	0.3	0.1	0.1	0.2
10	3.5	84.1	0.9	79.1	8.9	19.1	7.0	12.0	4.3	5.4	3.4	3.0	2.8	2.4	1.7	1.4	2.0	1.2	0.6	1.0	0.9	0.6	0.4	0.3	0.3	0.1	0.2	0.2
20	4.0	96.5	0.9	90.7	10.3	21.9	8.0	13.8	4.9	6.2	3.9	3.4	3.3	2.7	1.9	1.6	2.3	1.4	0.7	1.2	1.0	0.6	0.5	0.4	0.4	0.1	0.2	0.3
50	4.7	113.8	0.9	106.9	12.0	25.6	9.3	16.0	5.7	7.2	4.6	4.1	3.8	3.2	2.2	1.9	2.7	1.7	1.0	1.5	1.3	0.9	0.6	0.5	0.5	0.0	0.2	0.4
100	5.3	127.7	0.9	120.0	13.4	28.7	10.4	18.0	6.4	8.0	5.2	4.6	4.3	3.6	2.5	2.2	3.0	1.9	1.1	1.7	1.4	1.0	0.7	0.6	0.6	0.0	0.2	0.5
200	5.9	142.5	0.9	133.9	15.0	32.0	11.7	20.1	7.1	9.0	5.8	5.1	4.8	4.0	2.8	2.4	3.3	2.1	1.2	1.9	1.6	1.1	0.8	0.7	0.7	0.0	0.3	0.5
500	6.8	163.6	0.9	153.8	17.2	36.8	13.4	23.1	8.2	10.3	6.6	5.8	5.5	4.6	3.2	2.8	3.8	2.5	1.4	2.2	1.8	1.2	0.9	0.8	0.8	0.0	0.3	0.6

Table F.8 48 hour Duration (2 hour increment)

ARI (years)	Rainfall Intensity (mm/hr)	Total Rainfall (mm)	Areal Reduction Factor	Design Rainfall (mm)	Temporal Pattern (mm) 1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
5	1.9	93.1	1.0	88.5	14.2	24.2	9.9	6.7	5.2	4.6	2.7	4.2	3.4	2.3	1.9	1.1	1.6	1.3	0.7	0.9	0.8	0.5	0.5	0.4	0.4	0.3	0.4	0.4
10	2.2	103.7	1.0	98.5	15.8	26.9	11.0	7.5	5.8	5.1	3.1	4.7	3.7	2.6	2.1	1.2	1.8	1.5	0.8	1.0	0.9	0.6	0.6	0.5	0.4	0.3	0.4	0.4
20	2.5	119.0	1.0	113.0	18.1	30.9	12.7	8.6	6.7	5.9	3.5	5.4	4.3	2.9	2.4	1.4	2.0	1.7	0.9	1.1	1.0	0.7	0.7	0.6	0.5	0.3	0.5	0.5
50	2.9	140.2	1.0	133.2	21.1	36.0	14.8	10.0	7.7	6.8	4.1	6.4	5.1	3.5	2.8	1.6	2.4	2.0	1.2	1.5	1.3	0.9	0.9	0.8	0.5	0.4	0.7	0.8
100	3.3	157.4	1.0	149.5	23.6	40.4	16.6	11.2	8.7	7.6	4.6	7.2	5.7	3.9	3.1	1.8	2.7	2.2	1.3	1.6	1.5	1.0	1.0	0.9	0.6	0.4	0.7	0.9
200	3.7	175.7	1.0	166.9	26.4	45.1	18.5	12.5	9.7	8.5	5.2	8.0	6.3	4.3	3.5	2.0	3.0	2.5	1.5	1.8	1.7	1.2	1.2	1.0	0.7	0.5	0.8	1.0
500	4.2	201.7	1.0	191.6	30.3	51.7	21.3	14.4	11.1	9.8	5.9	9.2	7.3	5.0	4.0	2.3	3.4	2.9	1.7	2.1	1.9	1.3	1.3	1.1	0.8	0.6	1.0	1.1

Table F.9 72 hour Duration (4 hour increment)

ARI (years)	Rainfall Intensity (mm/hr)	Total Rainfall (mm)	Areal Reduction Factor	Design Rainfall (mm)	Temporal Pattern (mm)																	
					1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
5	1.4	102.3	1.0	98.2	18.1	33.3	12.0	8.0	6.0	3.8	5.0	2.9	1.7	2.3	0.8	1.3	1.0	0.5	0.6	0.2	0.6	0.3
10	1.6	113.9	1.0	109.3	20.1	37.1	13.3	8.9	6.7	4.3	5.6	3.3	1.9	2.5	0.9	1.4	1.1	0.5	0.7	0.2	0.7	0.3
20	1.8	130.6	1.0	125.4	23.1	42.5	15.3	10.2	7.7	4.9	6.4	3.8	2.1	2.9	1.0	1.6	1.3	0.6	0.8	0.3	0.8	0.4
50	2.1	154.0	1.0	147.8	26.9	49.7	17.9	11.8	8.9	5.8	7.4	4.4	2.5	3.4	1.3	2.1	1.6	1.0	1.0	0.4	1.0	0.6
100	2.4	172.8	1.0	165.9	30.2	55.7	20.1	13.3	10.0	6.5	8.3	5.0	2.8	3.8	1.5	2.3	1.8	1.2	1.2	0.5	1.2	0.7
200	2.7	192.8	1.0	185.1	33.7	62.2	22.4	14.8	11.1	7.2	9.3	5.6	3.1	4.3	1.7	2.6	2.0	1.3	1.3	0.6	1.3	0.7
500	3.1	221.4	1.0	212.5	38.7	71.4	25.7	17.0	12.8	8.3	10.6	6.4	3.6	4.9	1.9	3.0	2.3	1.5	1.5	0.6	1.5	0.9

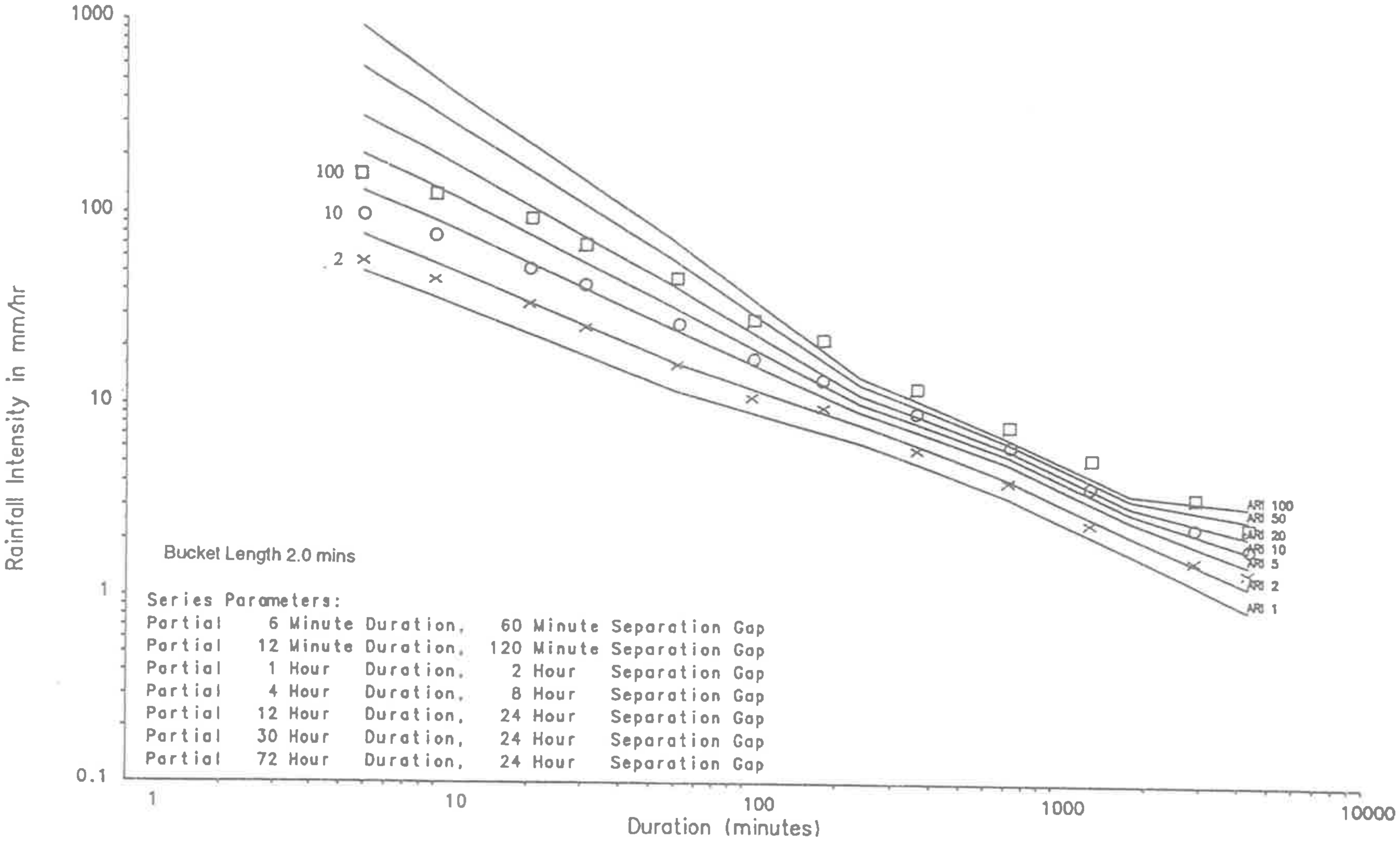


Figure F.1 Comparison of Generalised IFD Curves and the Pluviometer Record at Inverbrackie Creek, AW503508 (01/01/1985 - 01/01/1993)

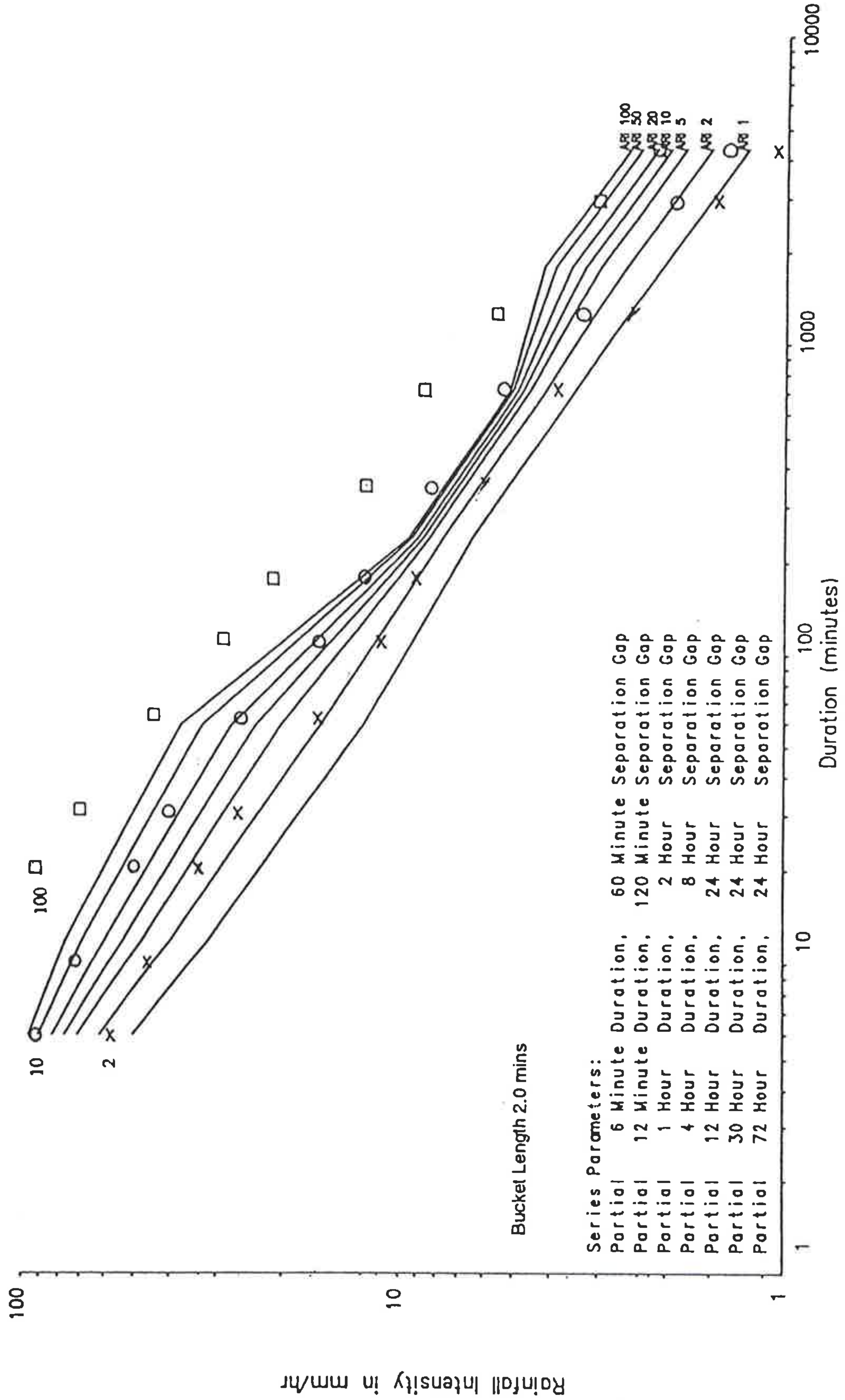


Figure F.2 Comparison of Generalised IFD Curves and the Pluviometer Record at Sutton Creek, AW503525 (01/01/1984 - 01/01/1993)

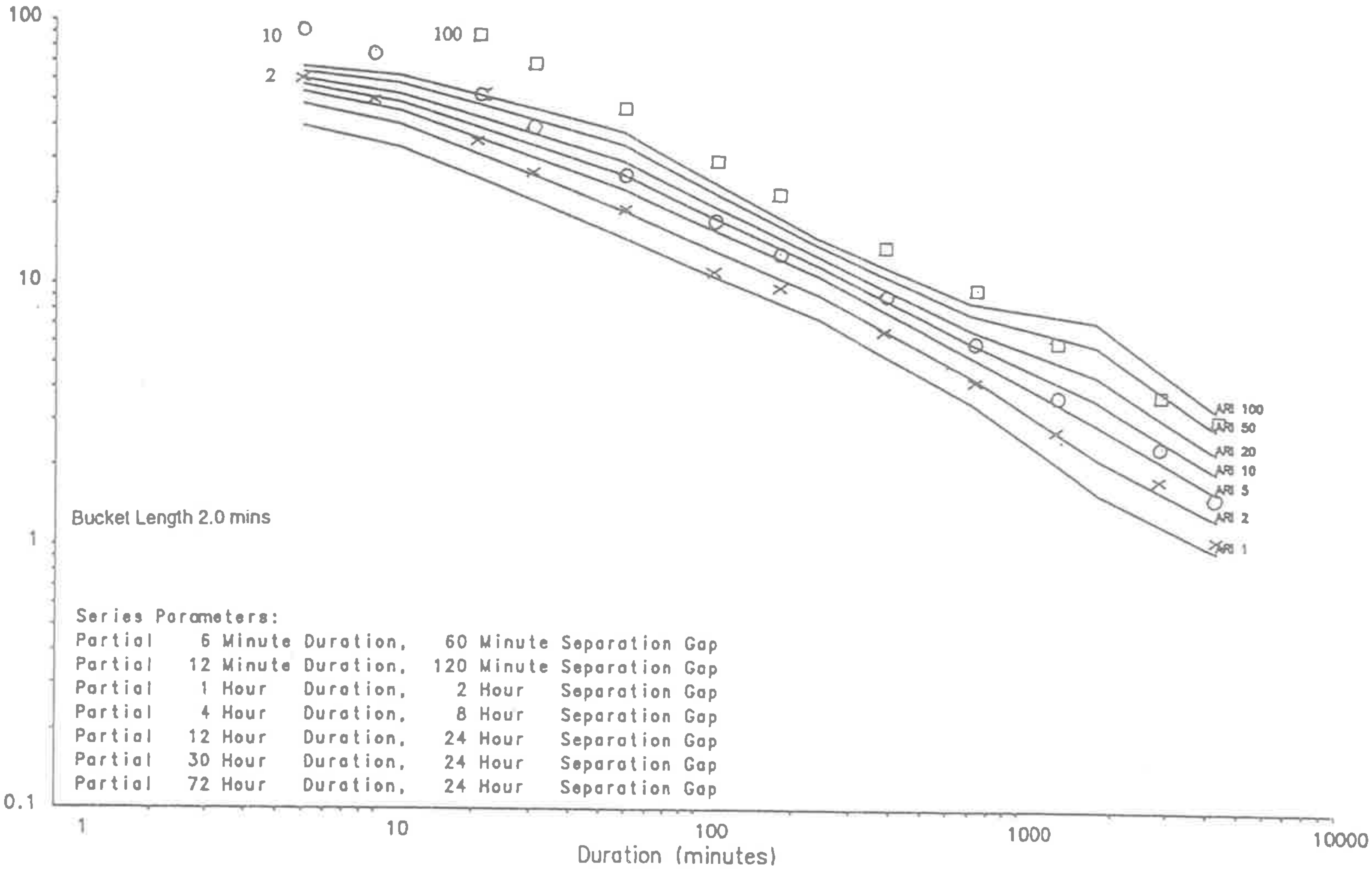


Figure F.3 Comparison of Generalised IFD Curves and the Pluviometer Record at Stirling P.O., 023785 (01/01/1965 - 01/01/1980)

Appendix G
Modelling of Streamflow in the Onkaparinga Catchment

Assumed Mt Bold Reservoir Gate Operations

In order to calculate a rating for Mt Bold Reservoir the gate operations shown in Table G.1 were assumed. These operations are based on those in Kotwicki (1984), Design Services (1982) and discussions with the reservoir operators as described in Section 4.2.1.2. It was assumed that no pre-releasing occurred.

The gate operation procedures in Kotwicki (1984) and Design Services (1982) were based on increments of gate openings of 75 mm. The current operating procedure has been modified and is now based upon gate openings of 100 mm (Appendix E). The current procedure only specifies gate operations up to a gauge height of 41.3 metres. If the reservoir continues to rise above this level, the instruction is to continue to raise the gates in the same sequence.

The rating resulting from the official operating procedure is compared to the adopted rating in Figure G.1. The adopted rating is very similar to that produced by the official operating policy and therefore the adopted rating should not have produced any significant error in the calculated outflow hydrographs.

It was assumed that the maximum gate openings have been adjusted to 3.8 metres as recommended in Section 4.4.

Table G.1 Assumed Mt Bold Reservoir Gate Operations

Elevation (m)	G.H. (m)	Gate Openings (mm)								Discharge (m^3/sec)
		1	2	3	4	5	6	7	8	
246.500	41.000	0	0	0	0	0	0	0	0	0
246.595	41.095	0	0	0	150	150	0	0	0	9
246.745	41.245	0	0	0	225	225	0	0	0	15
246.820	41.320	0	0	0	300	300	0	0	0	20
246.900	41.400	0	0	0	750	750	0	0	0	50
247.000	41.500	0	0	0	1500	1500	0	0	0	99
247.100	41.600	0	0	750	2000	2000	750	0	0	182
247.200	41.700	450	1000	1000	2000	2000	1000	1000	450	304
247.300	41.800	1500	2000	2000	2000	2000	2000	2000	1500	509
247.400	41.900	3800	3800	3800	3800	3800	3800	3800	3800	768
247.600	42.100	3800	3800	3800	3800	3800	3800	3800	3800	842
247.800	42.300	3800	3800	3800	3800	3800	3800	3800	3800	919
248.000	42.500	3800	3800	3800	3800	3800	3800	3800	3800	998
248.200	42.700	3800	3800	3800	3800	3800	3800	3800	3800	1080
248.400	42.900	3800	3800	3800	3800	3800	3800	3800	3800	1164

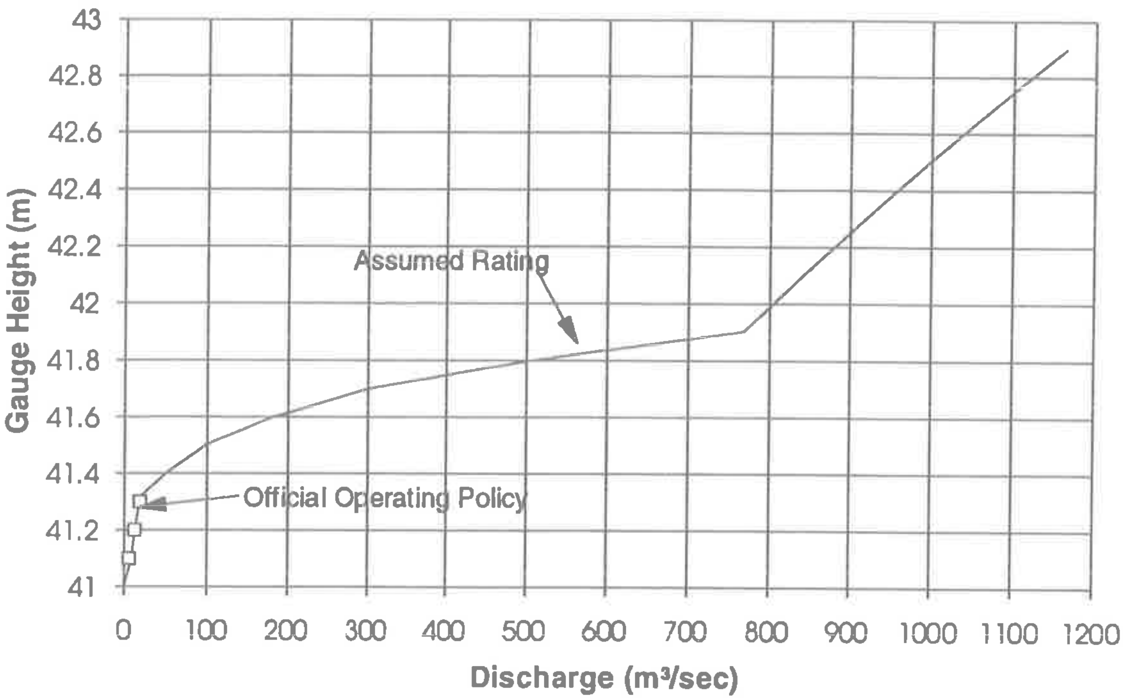


Figure G.1 Comparison of Assumed Rating with the Rating Produced by the Official Operating Policy

RORB Data File for the Onkaparinga River to Old Noarlunga

```
ONKAPARINGA RIVER TO OLD NOARLUNGA
0,              MIXTURE OF REACHES
1,1,8.02,-99,      STOR 1 - SUB AREA A
3
1,1,6.51,-99,      STOR 2 - SUB AREA B
4
5,1,1.30,-99,      STOR 3
3
1,1,3.51,-99,      STOR 4 - SUB AREA C
4
5,1,1.89,-99,      STOR 5
3
1,1,5.73,-99,      STOR 6 - SUB AREA D
3
1,1,7.30,-99,      STOR 7 - SUB AREA E
4
5,1,1.50,-99,      STOR 8
4
5,1,3.35,-99,      STOR 9
2,1,6.30,-99,      STOR 10 - SUB AREA F
3
1,1,2.52,-99,      STOR 11 - SUB AREA G
4
5,1,1.32,-99,      STOR 12
5,1,1.08,-99,      STOR 13
3
1,1,6.14,-99,      STOR 14 - SUB AREA H
4
5,1,3.36,-99,      STOR 15
3
1,1,5.21,-99,      STOR 16 - SUB AREA I
4
5,1,2.36,-99,      STOR 17
3
1,1,2.09,-99,      STOR 18 - SUB AREA J
4
5,1,6.99,-99,      STOR 19 - ROUTE TO HOULGRAVES
5,1,0.93,-99,      STOR 20
5,4,3.93,-99,      STOR 21
2,4,1.64,-99,      STOR 22 - SUB AREA K
3
1,1,6.31,-99,      STOR 23 - SUB AREA L
5,4,1.90,-99,      STOR 24
4
8,4,-99
7
Design inflow to Mt Bold Res
6
Mt Bold Res
C    Stor-Discharge Table   1
C    Month - August, September, October, November
C    Exceedance Probability  25 %
```

```
C      Initial Drawdown       1.09E+06 m^3
1,-1.09E+06
30,    no of pairs of values in storage - discharge table
4.498E+07,0
4.499E+07,9
4.545E+07,15
4.568E+07,20
4.592E+07,50
4.623E+07,99
4.654E+07,182
4.685E+07,304
4.716E+07,509
4.747E+07,768
4.810E+07,842
4.874E+07,919
4.938E+07,998
5.003E+07,1080
5.068E+07,1164
5.074E+07,1172
5.100E+07,1212
5.266E+07,1349
5.436E+07,1617
5.608E+07,1956
5.785E+07,2365
5.856E+07,2550
6.148E+07,3459
6.384E+07,4312
6.525E+07,4915
6.916E+07,6613
7.323E+07,8634
7.744E+07,10898
8.181E+07,13373
8.632E+07,16039,-99
C gates operated according to Kotwicki
C
C      Elevation Storage Relation Flag
C      H-S Table                          1
C      no of pairs of values in H-S table      49
1,49
0,0.000E+00
4,8.900E+04
5,2.090E+05
6,3.520E+05
7,5.090E+05
8,6.840E+05
9,8.890E+05
10,1.129E+06
11,1.409E+06
12,1.734E+06
13,2.106E+06
14,2.528E+06
15,3.003E+06
16,3.534E+06
17,4.123E+06
18,4.774E+06
19,5.489E+06
20,6.274E+06
21,7.136E+06
```

```
22,8.079E+06
23,9.106E+06
24,1.022E+07
25,1.142E+07
26,1.271E+07
27,1.410E+07
28,1.558E+07
29,1.716E+07
30,1.883E+07
31,2.061E+07
32,2.250E+07
33,2.450E+07
34,2.661E+07
35,2.884E+07
36,3.119E+07
37,3.365E+07
38,3.623E+07
39,3.894E+07
40,4.176E+07
41,4.469E+07
41.4,4.590E+07
42,4.779E+07
43,5.100E+07
44,5.436E+07
45,5.785E+07
46,6.147E+07
47,6.525E+07
48,6.916E+07
49,7.323E+07
50,7.744E+07,-99
7
Outflow from Mt Bold Res
C
C                    Lower Onkaparinga River
5,1,5.02,-99,        stor 25
3
1,1,7.53,-99,        stor 26 - Sub Area M
4
5,1,0.74,-99,        stor 27
3
1,1,0.91,-99,        stor 28 - Sub Area N
4
5,1,2.50,-99,        stor 29
5,4,2.27,-99,        Clarendon Weir
6
Clarendon Weir
C      Weir formula only       3
C      Q=Kw*Ls*(H)^1.5
C      Initial Drawdown        0 m^3
C      no of Spillways        1
C      Spillway Elevation      10 m
C      Length of Spillway      61 m
C      Spillway coefficient   1.54
3,0,1
10,61,1.54,-99
C      Storage - Elevation Information
C      S=a(H-Ho)^b            2
C      a                      6,800
```

```
C      b                   1.8556
C      Ho                  -8
2,6800,1.8556,-8,-99
C            Pipeflow from Clarendon Weir
C            9,0,0,0,0,-99
7
Flow at Clarendon
5,1,8.02,-99,          stor 30
3
1,1,7.51,-99,          stor 31 - Sub Area O
4
5,1,1.45,-99,          stor 32
2,1,12.32,-99,         stor 33 - Sub Area P
7
Flow at Old Noarlunga
0
C SUB AREAS
51.5,27.6,19.4,30.6,29.1,46.5,22.0,34.8,23.0,37.7,22.9,39.8,26.5,31.2,48.0,36.0,-99
0,-99, NO IMPERVIOUS AREAS
24 hour 10 ARI
Design
C time incr, no of time incr, no of bursts, no of pluvios, flag(=0)
1.00,80,1,1,0,-99
C no of time incr from beginning to end of burst
0,24
ARR Vol II Temporal Pattern
7.7,16.4,6.0,10.3,3.7,4.6,2.9,2.6,2.4,2.0,1.4,1.2,1.7,1.0,0.5,0.9,0.7,0.5,0.3,0.3,0.3,0.1,
0.1,0.2,-99
```

Using Surfer to Construct Isohyets

The main advantages of using a package such as Surfer to generate isohyets are:

1. Ease of presentation of results as the output can be directed to a laser printer.
2. Many catchments have already been digitised.
3. Speed of drawing isohyets. Once the catchment boundaries and rivers have been digitised and the location of all the raingauges has been determined, isohyets can be generated almost instantaneously.

Surfer requires a input ASCII file of the locations and depths of rainfall. A grid file is then produced from which the contours are determined.

There are three different griding methods available:

1. Inverse Distance. The Inverse Distance method uses a weighted averaging technique to interpolate grid nodes.
2. Kriging. The Kriging method uses geostatistical techniques to calculate the autocorrelation between data points and produces a minimum variance unbiased estimate.
3. Minimum Curvature. This method first examines all data and sets the nearest grid node to that data value. The values at the other grid nodes are then computed so as to give a grided surface of minimum curvature through the set grid nodes.

The choice of griding method can have a dramatic effect on the isohyets which are produced. The three methods were examined for a number of different cases and it was found that the Kriging method consistently produced the most satisfactory results.

Once the grided file was produced using the Kriging method, the isohyets were drawn using TOPO. The locations of the centroid for each subarea, the catchment boundaries and rivers can be shown.

Flow at Houlgraves Weir for 1987 and 1988

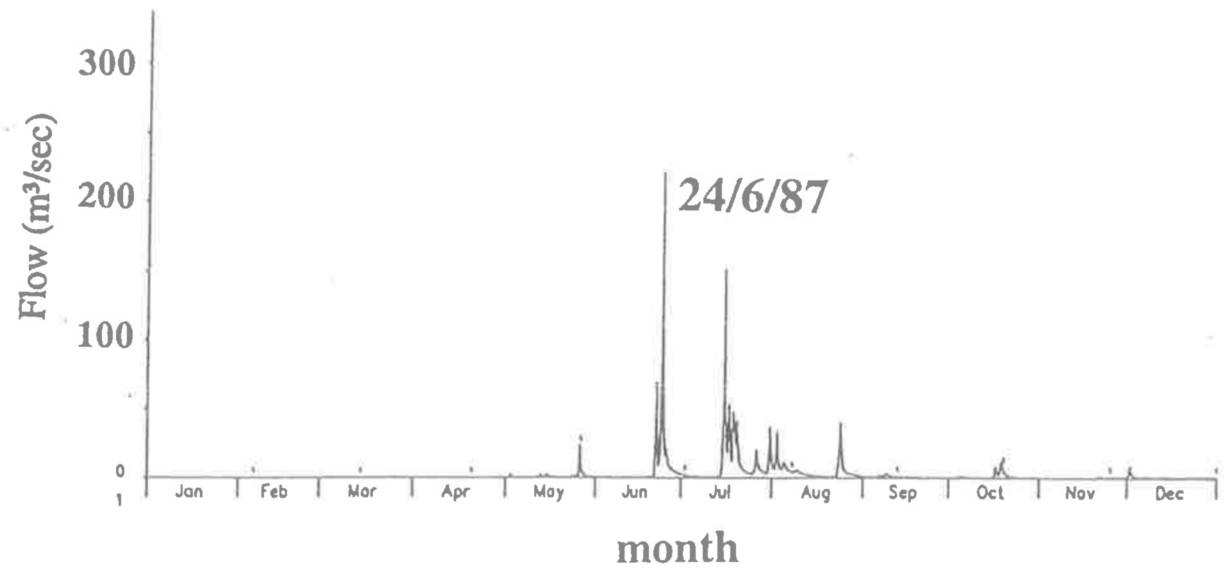


Figure G.1 Flow at Houlgraves Weir 1987

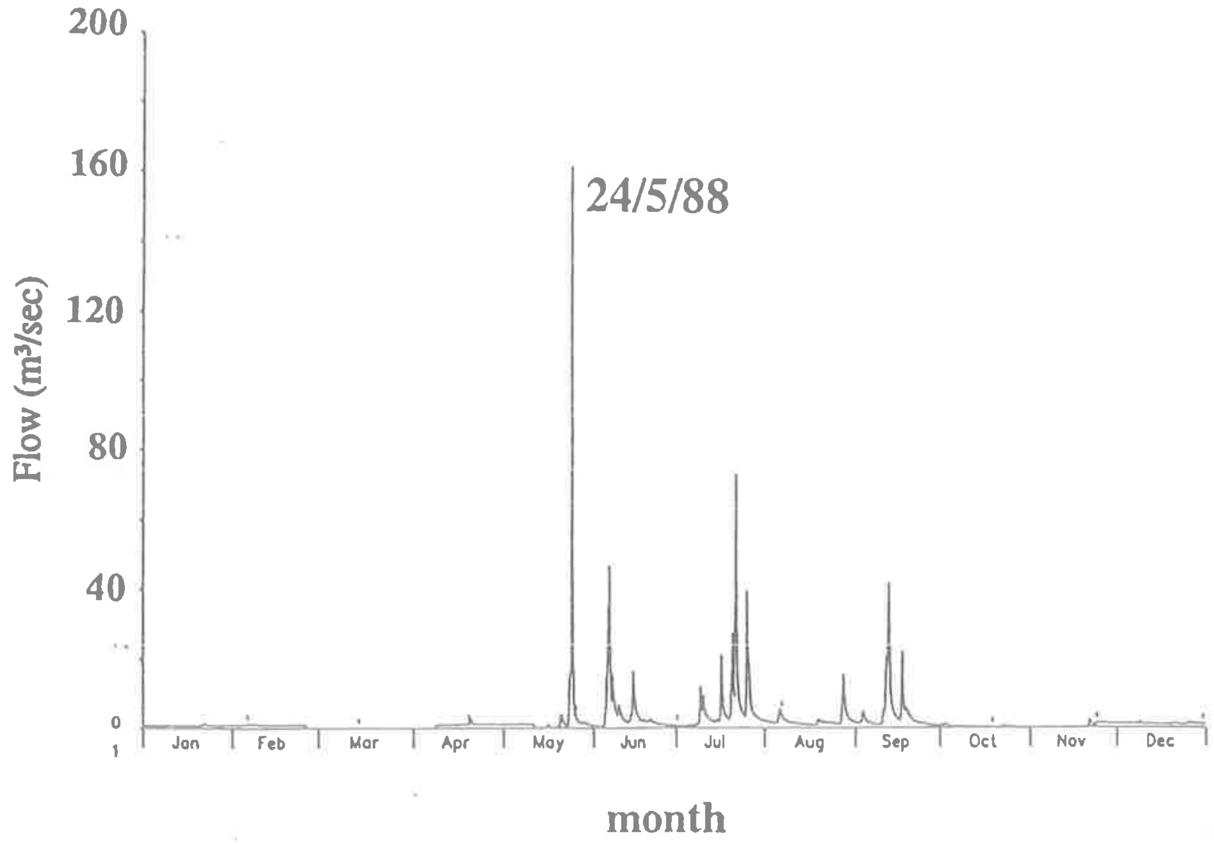


Figure G.2 Flow at Houlgraves Weir 1988

Comparison of Calculated and Recorded Inflows to Mt Bold Reservoir for the 3 Largest Events

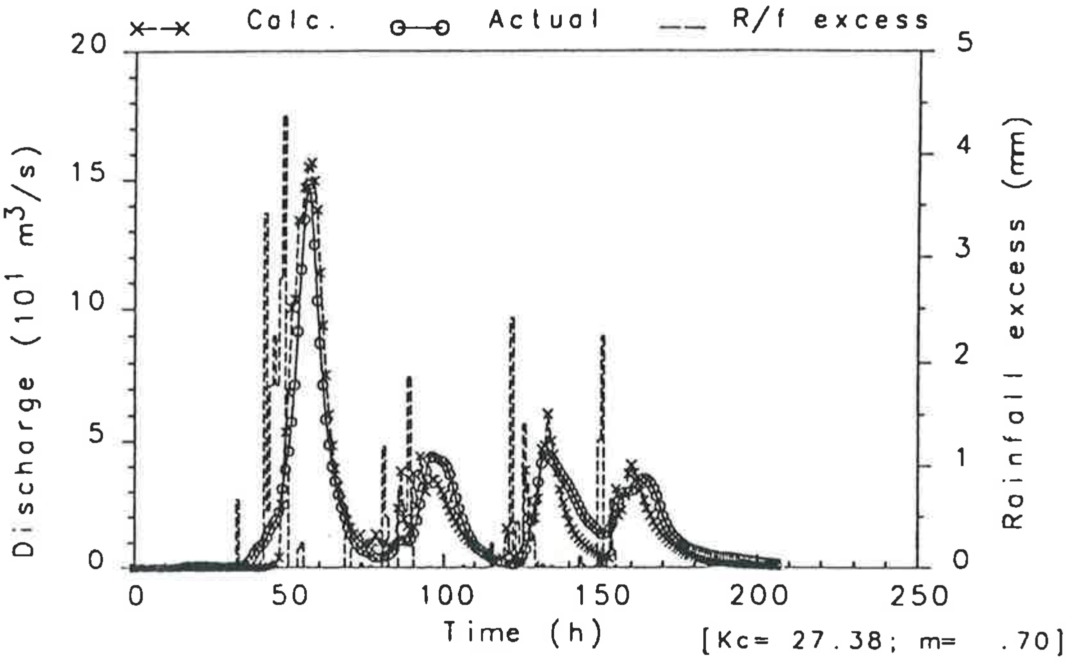


Figure G.3 Comparison of Calculated and Recorded Mt Bold Inflow Hydrographs for 15 July 1987 (using adopted model parameters)

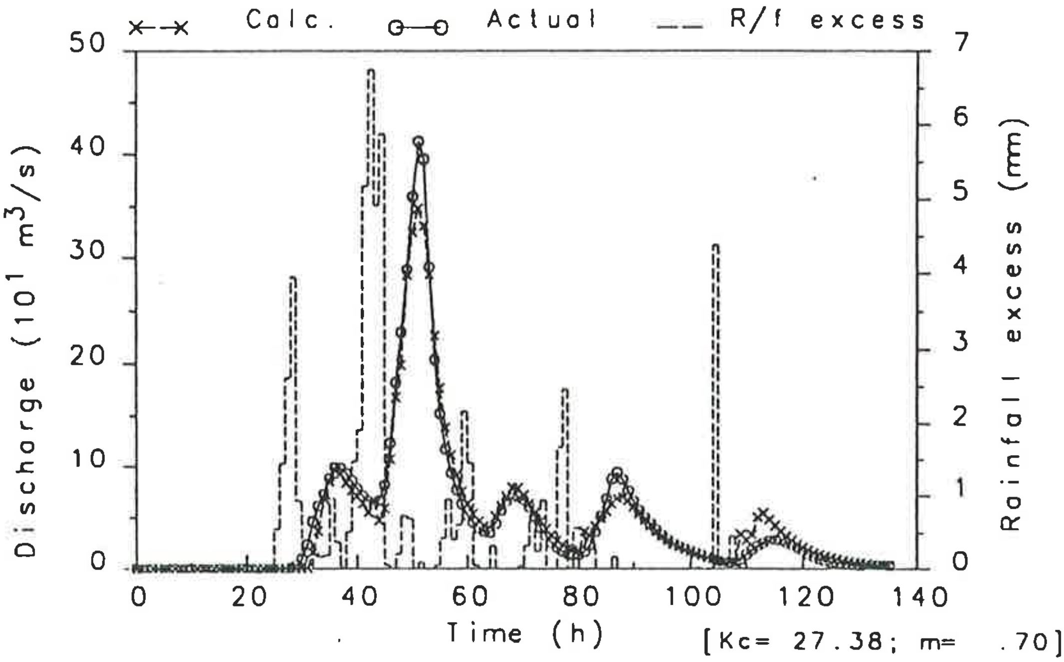


Figure G.4 Comparison of Calculated and Recorded Mt Bold Inflow Hydrographs for 30 August 1992 (using adopted model parameters)

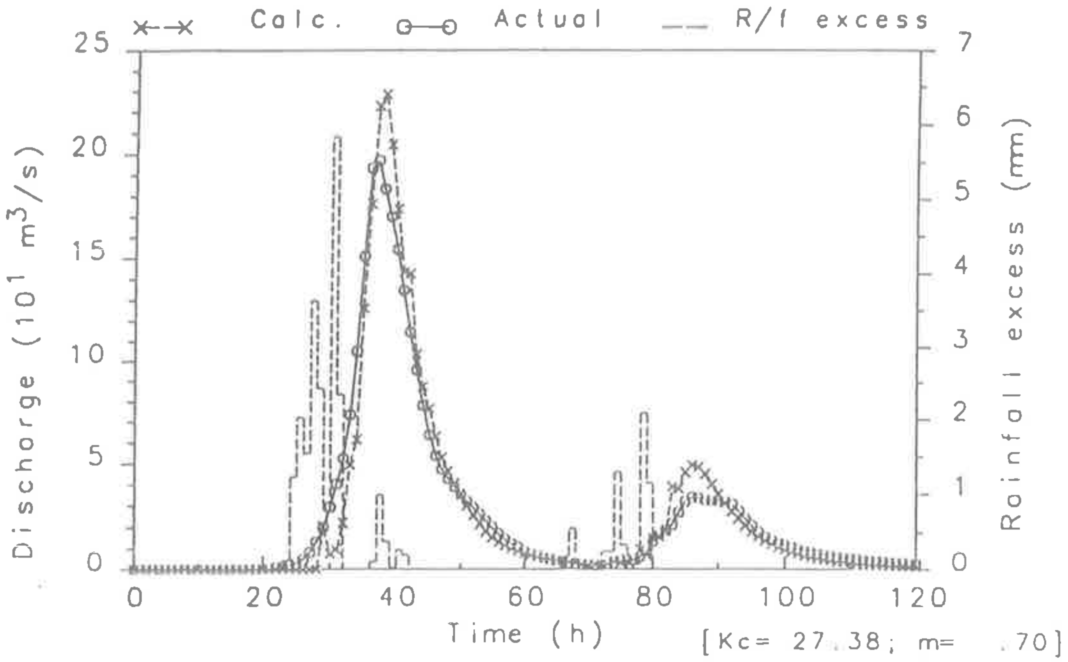


Figure G.5 Comparison of Calculated and Recorded Mt Bold Inflow Hydrographs for 8 October 1992 (using adopted model parameters)

Appendix H

WSO87 Output for Inflows to Mt Bold Reservoir

WSO87 Output for Inflows to Mt Bold Reservoir

Data and Statistics of Full Record

Inflows to Mt Bold Reservoir (1897-35, 74-92)
Flow (m^3/sec)

```
INPUT DATA :-
                NUMBER OF EVENTS =     58
                MAGNITUDE OF EVENTS  :

           103.00        171.00         37.00        224.00        131.00
            18.00        228.00        131.00        166.00        244.00
            71.00        111.00        234.00         99.00         40.00
           115.00         37.00          9.00        287.00         81.00
           306.00         84.00         74.00        276.00        219.00
            92.00        254.00         84.00        131.00         74.00
            40.00         77.00         40.00         96.00        171.00
           107.00        342.00         64.00        342.00        138.00
            72.00         43.00         18.00        129.00        131.00
            54.00        313.00         15.00        138.00        113.00
            62.00         74.00        250.00        182.00         58.00
            93.00        201.00        488.00

                     STATISTICS OF NORMAL DATA

                                MEAN =   137.6
                  STANDARD DEVIATION =   100.2
             COEFFICIENT OF SKEWNESS =     1.195
        STANDARD ERROR COEFF OF SKEW =      .314

                     STATISTICS OF LOGS OF DATA

                    LOGARITHMIC MEAN =     2.013
   LOGARITHMIC STANDARD DEVIATION =         .363
     COEFF OF SKEWNESS OF LOGARITHMS =     -.670
                      GEOMETRIC MEAN =   103.0
        STANDARD ERROR COEFF OF SKEW =      .314

                  TEST OF OUTLIERS

             VALUE OF LOW OUTLIER =        4.734
             VALUE OF HIGH OUTLIER =       710.547

             NO OUTLIERS ARE FOUND IN THIS DATA SET

          ORDERED VALUES WITH CORRESPONDING RANK AND PROBABILITY

          NOTE: P = 100*(M- .40)/(N+ .20), HIGHEST VALUE HAS RANK M = 1.
```

RANK	MAGNITUDE	P	RANK	MAGNITUDE	P	RANK	MAGNITUDE	P
1	488.00	1.0	2	342.00	2.7	3	342.00	4.5
4	313.00	6.2	5	306.00	7.9	6	287.00	9.6
7	276.00	11.3	8	254.00	13.1	9	250.00	14.8
10	244.00	16.5	11	234.00	18.2	12	228.00	19.9
13	224.00	21.6	14	219.00	23.4	15	201.00	25.1
16	182.00	26.8	17	171.00	28.5	18	171.00	30.2
19	166.00	32.0	20	138.00	33.7	21	138.00	35.4
22	131.00	37.1	23	131.00	38.8	24	131.00	40.5
25	131.00	42.3	26	129.00	44.0	27	115.00	45.7
28	113.00	47.4	29	111.00	49.1	30	107.00	50.9
31	103.00	52.6	32	99.00	54.3	33	96.00	56.0
34	93.00	57.7	35	92.00	59.5	36	84.00	61.2
37	84.00	62.9	38	81.00	64.6	39	77.00	66.3
40	74.00	68.0	41	74.00	69.8	42	74.00	71.5
43	72.00	73.2	44	71.00	74.9	45	64.00	76.6
46	62.00	78.4	47	58.00	80.1	48	54.00	81.8
49	43.00	83.5	50	40.00	85.2	51	40.00	86.9

52	40.00	88.7	53	37.00	90.4	54	37.00	92.1
55	18.00	93.8	56	18.00	95.5	57	15.00	97.3
58	9.00	99.0						

PEARSON TYPE III DISTRIBUTION FITTED TO FULL RECORD

RECURRENCE INTERVAL(YRS.),	EXCEEDANCE PROBABILITY(%)	MAGNITUDE
1.01	99.00	-7.9
1.05	95.00	12.9
1.11	90.00	28.7
1.25	80.00	53.0
2.00	50.00	118.1
5.00	20.00	211.0
10.00	10.00	271.9
25.00	4.00	346.7
50.00	2.00	400.6
100.00	1.00	452.9
200.00	.50	504.1
1000.00	.10	619.5
10000.00	.01	779.1

LOG-PEARSON TYPE III DISTRIBUTION FITTED TO FULL RECORD

RECURRENCE INTERVAL(YRS.)	EXCEEDENCE PROBABILITY(%)	MAGNITUDE
1.01	99.00	9.9
1.05	95.00	22.6
1.11	90.00	33.8
1.25	80.00	53.0
2.00	50.00	113.0
5.00	20.00	210.9
10.00	10.00	278.3
25.00	4.00	361.3
50.00	2.00	420.1
100.00	1.00	475.6
200.00	.50	528.0
1000.00	.10	638.0
10000.00	.01	769.3

NORMAL DISTRIBUTION FITTED TO FULL RECORD

RECURRENCE INTERVAL(YRS.)	EXCEEDANCE PROBABILITY(%)	MAGNITUDE
1.01	99.00	-95.5
1.05	95.00	-27.2
1.11	90.00	9.1
1.25	80.00	53.2
2.00	50.00	137.6
5.00	20.00	222.0
10.00	10.00	266.1
25.00	4.00	313.1
50.00	2.00	343.5
100.00	1.00	370.7
200.00	.50	395.8
1000.00	.10	447.3
10000.00	.01	510.3

LOG-NORMAL DISTRIBUTION FITTED TO FULL RECORD

RECURRENCE INTERVAL(YRS.)	EXCEEDANCE PROBABILITY(%)	MAGNITUDE
1.01	99.00	14.7
1.05	95.00	26.0
1.11	90.00	35.2
1.25	80.00	50.9
2.00	50.00	103.0
5.00	20.00	208.3

10.00	10.00	301.1
25.00	4.00	445.8
50.00	2.00	574.4
100.00	1.00	721.3
200.00	.50	889.1
1000.00	.10	1367.0
10000.00	.01	2314.1

GUMBEL DISTRIBUTION FITTED TO FULL RECORD

RECURRENCE INTERVAL (YRS.)	EXCEEDENCE PROBABILITY(%)	MAGNITUDE POTTER	GUMBEL
1.01	99.00	-41.	-27.
1.05	95.00	-4.	7.
1.11	90.00	18.	27.
1.25	80.00	48.	55.
2.00	50.00	120.	121.
5.00	20.00	216.	210.
10.00	10.00	279.	268.
25.00	4.00	360.	342.
50.00	2.00	419.	397.
100.00	1.00	478.	452.
200.00	.50	537.	506.
1000.00	.10	674.	632.
10000.00	.01	869.	812.

LOG-GUMBEL DISTRIBUTION FITTED TO FULL RECORD

RECURRENCE INTERVAL (YRS.)	EXCEEDENCE PROBABILITY(%)	MAGNITUDE POTTER	GUMBEL
1.01	99.00	23.	26.
1.05	95.00	32.	35.
1.11	90.00	38.	41.
1.25	80.00	49.	52.
2.00	50.00	89.	90.
5.00	20.00	198.	188.
10.00	10.00	336.	307.
25.00	4.00	658.	570.
50.00	2.00	1081.	901.
100.00	1.00	1772.	1421.
200.00	.50	2899.	2238.
1000.00	.10	9062.	6405.
10000.00	.01	46204.	28786.

POWER TRANSFORMED NORMAL DISTRIBUTION FITTED TO FULL RECORD

RECURRENCE INTERVAL(YRS.)	EXCEEDANCE PROBABILITY(%)	MAGNITUDE
1.01	99.00	8.2
1.05	95.00	22.3
1.11	90.00	34.4
1.25	80.00	54.5
2.00	50.00	113.5
5.00	20.00	207.2
10.00	10.00	273.1
25.00	4.00	358.0
50.00	2.00	421.9
100.00	1.00	485.6
200.00	.50	549.9
1000.00	.10	700.3
10000.00	.01	920.9

FISHER-TIPPETT TYPE III DISTRIBUTION FITTED TO FULL RECORD

RECURRENCE INTERVAL(YRS.)	EXCEEDANCE PROBABILITY(%)	MAGNITUDE

1.01	99.00	4.6
1.05	95.00	16.6
1.11	90.00	29.3
1.25	80.00	51.2
2.00	50.00	116.1
5.00	20.00	212.2
10.00	10.00	274.8
25.00	4.00	349.3
50.00	2.00	401.5
100.00	1.00	451.2
200.00	.50	498.3
1000.00	.10	602.9
10000.00	.01	740.3

RESULTS OF GOODNESS OF FIT TEST

DISTRIBUTION TYPE / TEST NO.	1	2	3	4
PEARSON	4.4859	3.0595	6.3103	6.0897
LOG-PEARSON	4.1413	3.0395	6.3103	6.0897
NORMAL	10.1138	5.2298	8.0345	7.8138
LOG-NORMAL	10.1396	8.2399	.7931	.5724
GUMBEL	4.9710	3.0944	5.9655	5.9345
POTTER	5.1314	2.5780	6.3103	6.0897
LOG-GUMBEL	25.0810	21.9417	2.5172	2.2966
LOG-POTTER	31.3391	28.0343	4.5862	4.3655
P.T.NORMAL	3.8527	2.9886	5.9655	5.9345
FISHER-TIPPETT	4.0532	2.9111	5.9655	5.7448

TEST NO.	TEST TYPE	TEST EVALUATION
1	DIFFERENCE TEST	SMALLEST = BEST
2	MODIFIED DIFFERENCE TEST (BOTTOM HALF OF DATA)	SMALLEST = BEST
3	CHI-SQUARED TEST	SMALLEST = BEST
4	MODIFIED CHI-SQUARED TEST (BOTTOM 80% OF DISTRIBUTION)	SMALLEST = BEST

Appendix I

End of Month Storage Probability Distributions for Mt Bold Reservoir

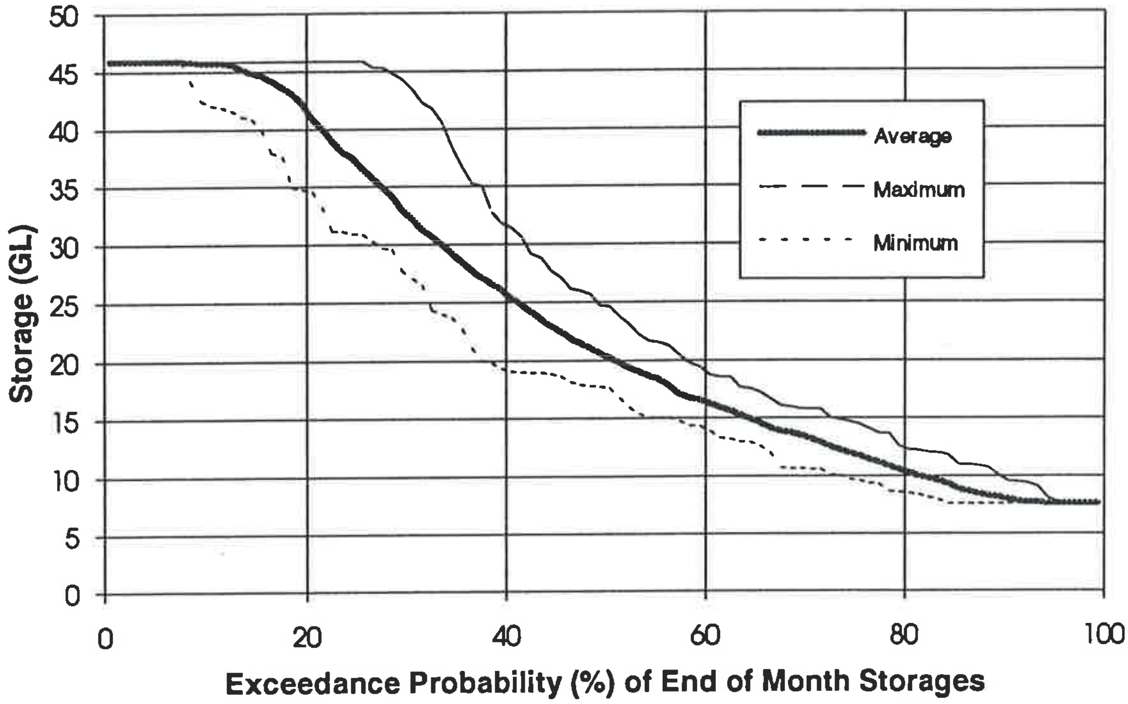


Figure I.1 End of July Storage Exceedance Probability (based upon 2000 years of synthetic data)

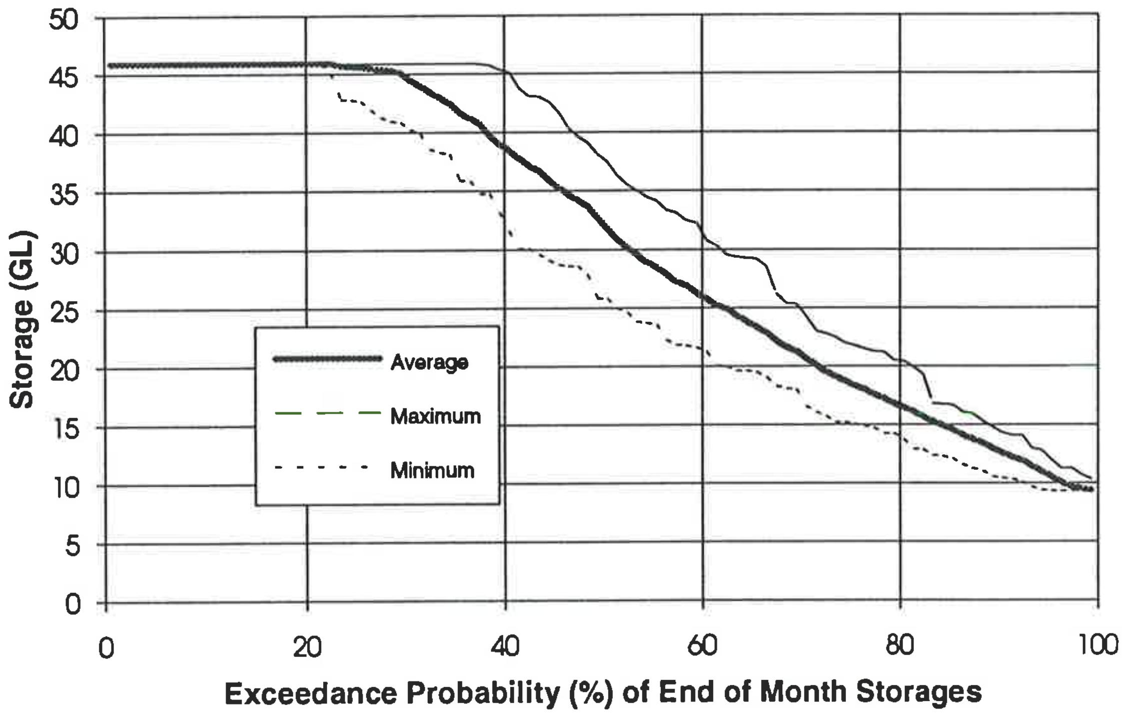


Figure I.2 End of August Storage Exceedance Probability (based upon 2000 years of synthetic data)

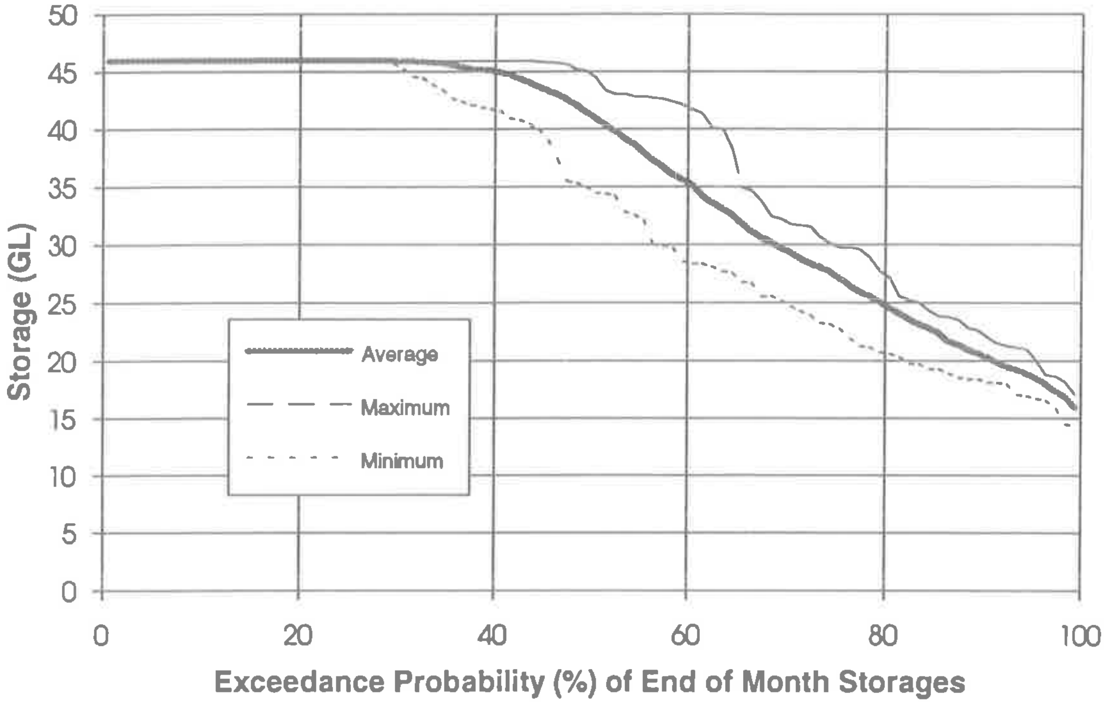


Figure I.3 End of September Storage Exceedance Probability (based upon 2000 years of synthetic data)

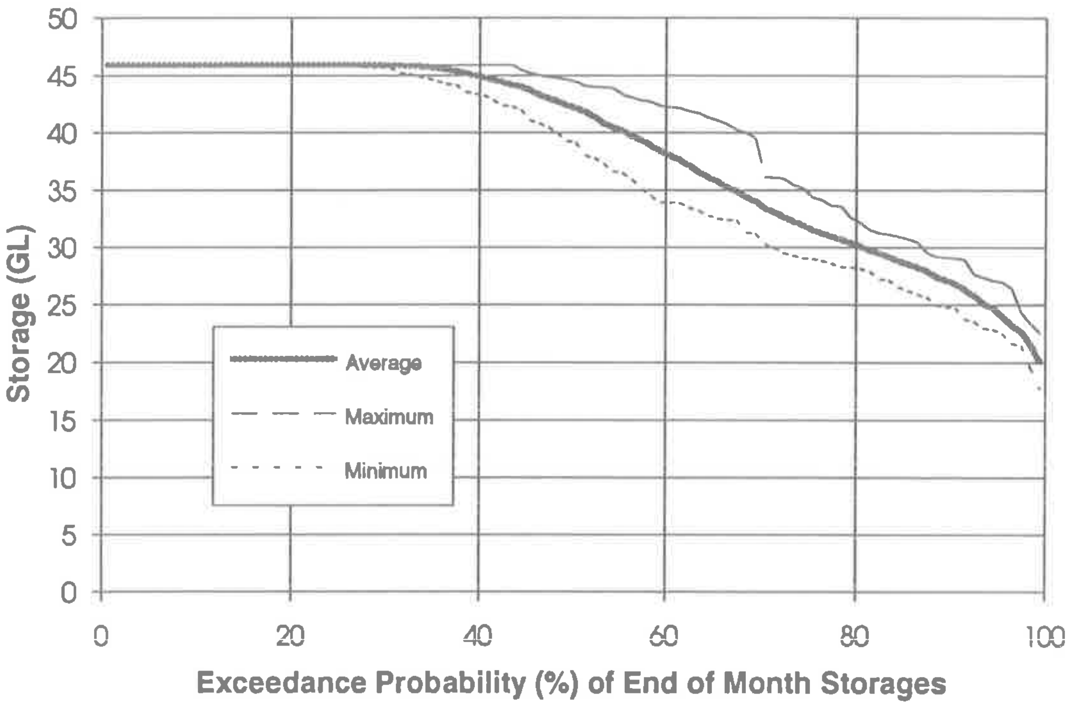


Figure I.4 End of October Storage Exceedance Probability (based upon 2000 years of synthetic data)

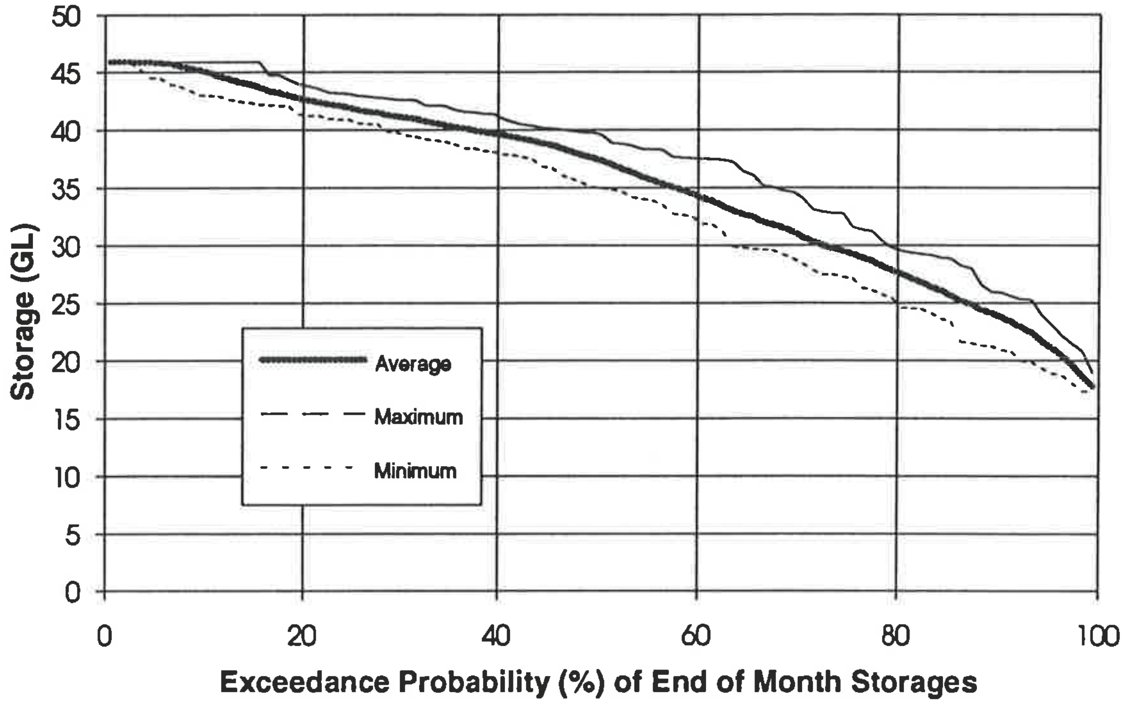


Figure I.5 End of November Storage Exceedance Probability (based upon 2000 years of synthetic data)

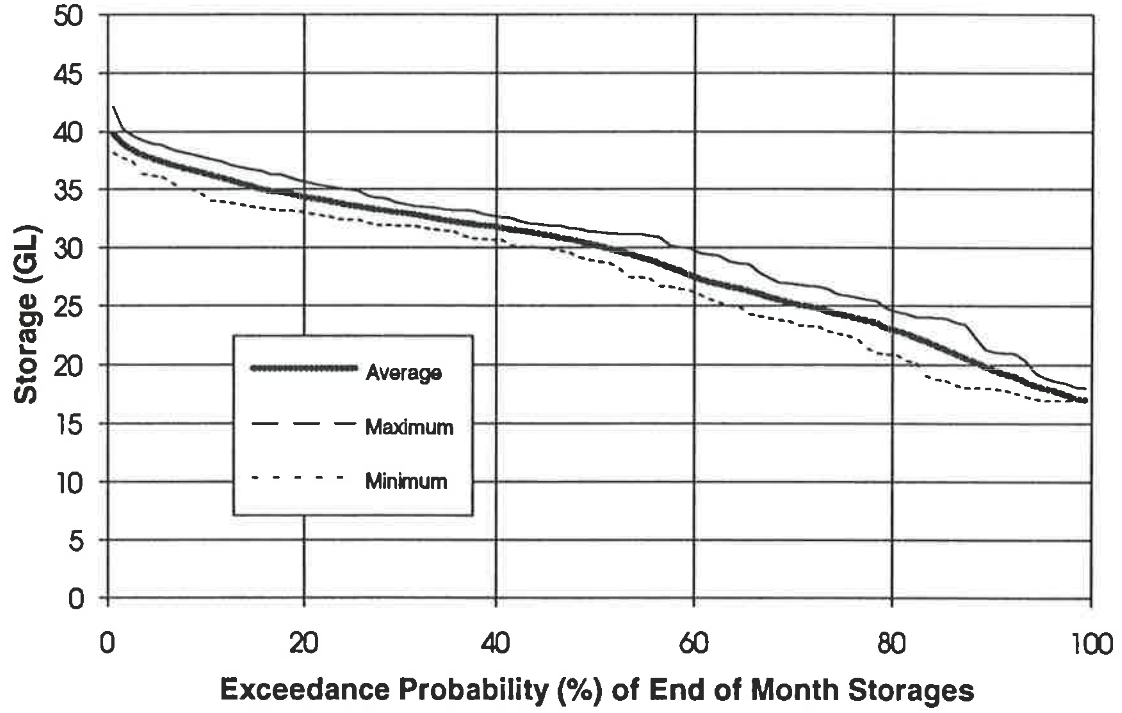


Figure I.6 End of December Storage Exceedance Probability (based upon 2000 years of synthetic data)

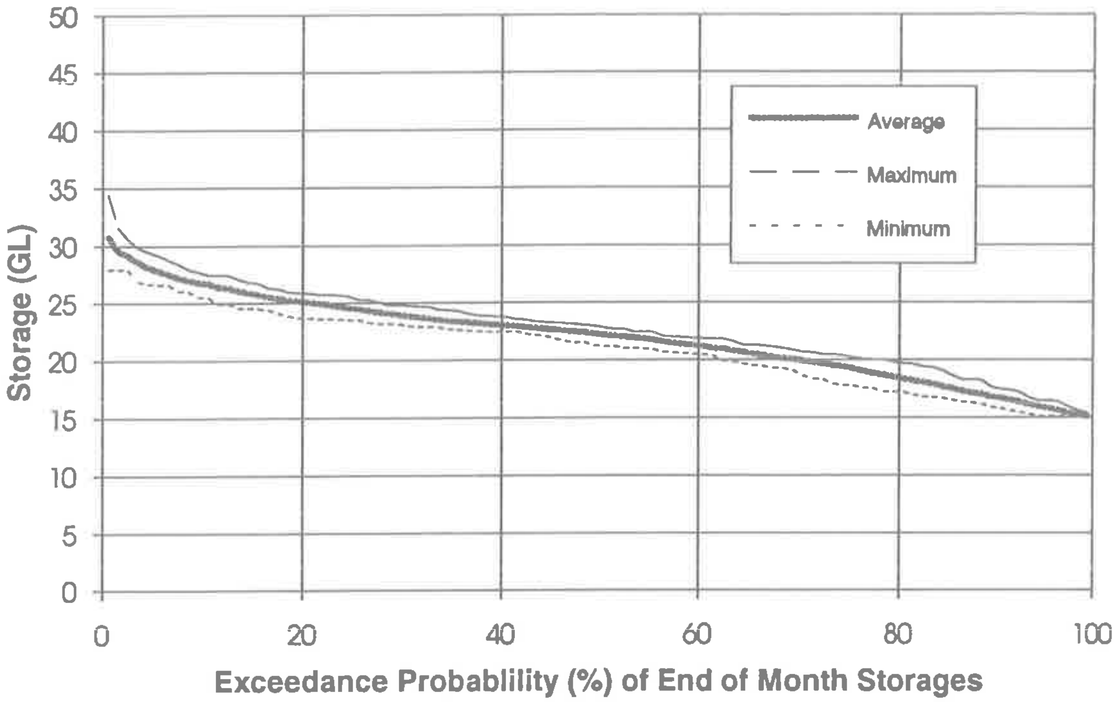


Figure I.7 End of January Storage Exceedance Probability (based upon 2000 years of synthetic data)

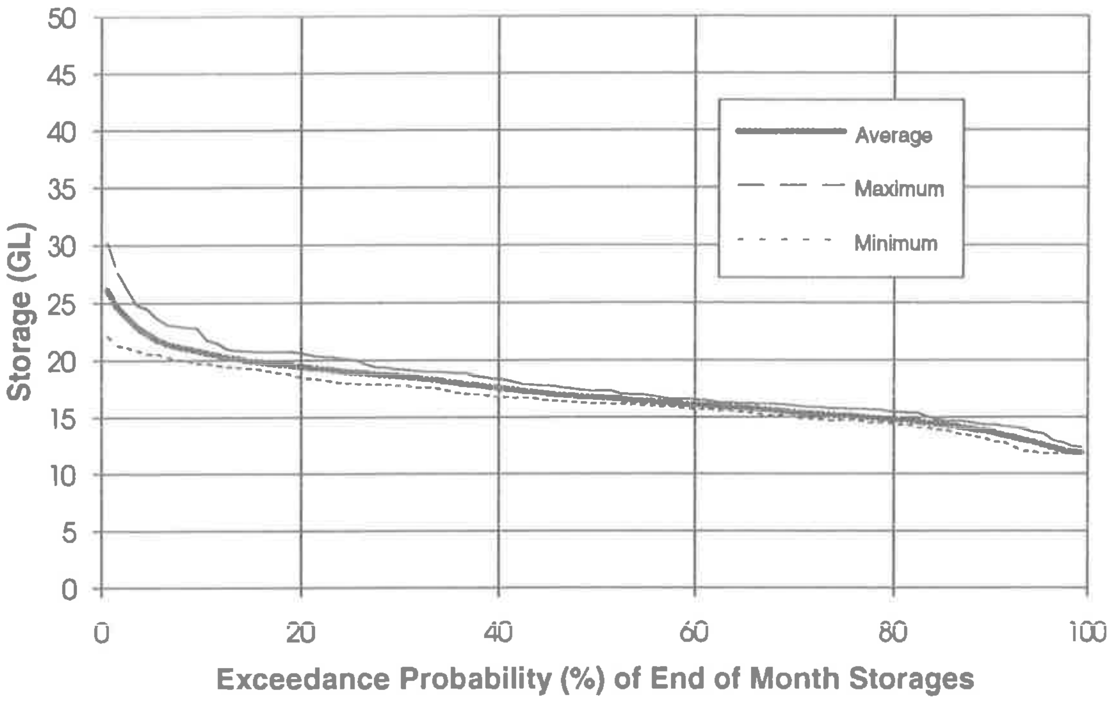


Figure I.8 End of February Storage Exceedance Probability (based upon 2000 years of synthetic data)

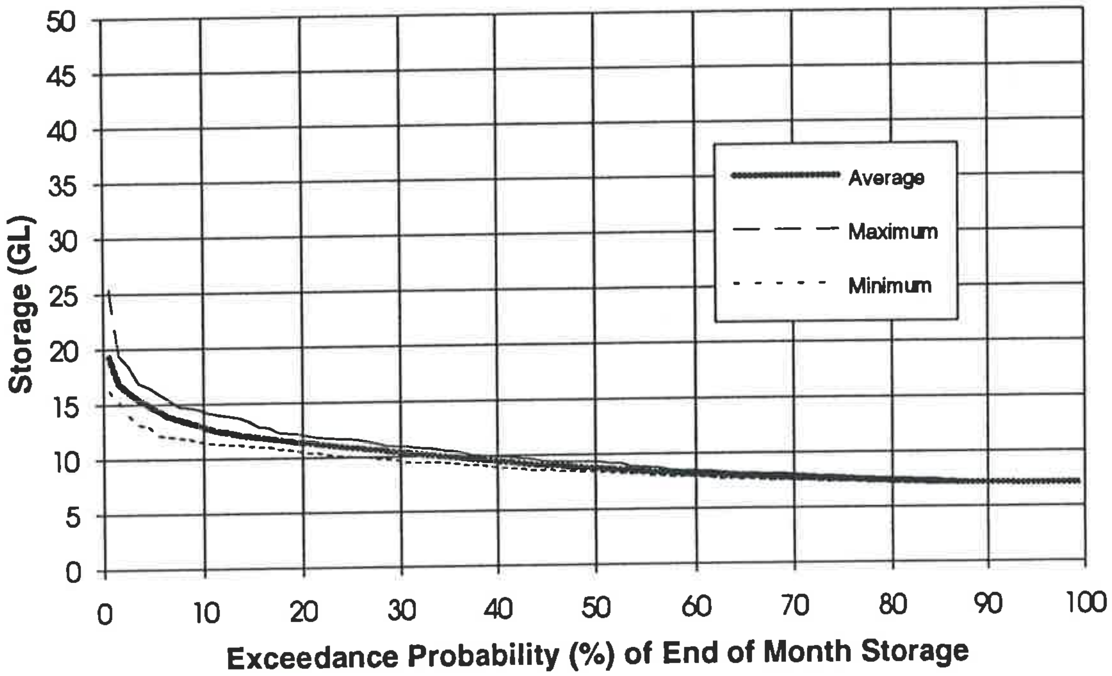


Figure I.9 End of March Storage Exceedance Probability (based upon 2000 years of synthetic data)

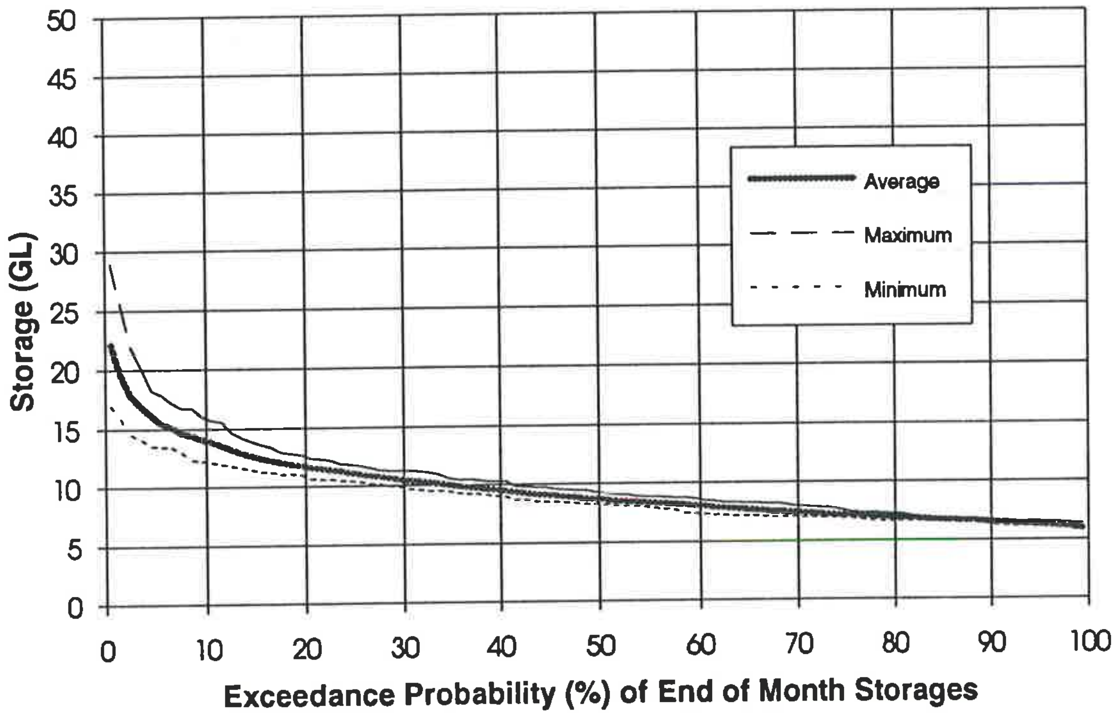


Figure I.10 End of April Storage Exceedance Probability (based upon 2000 years of synthetic data)

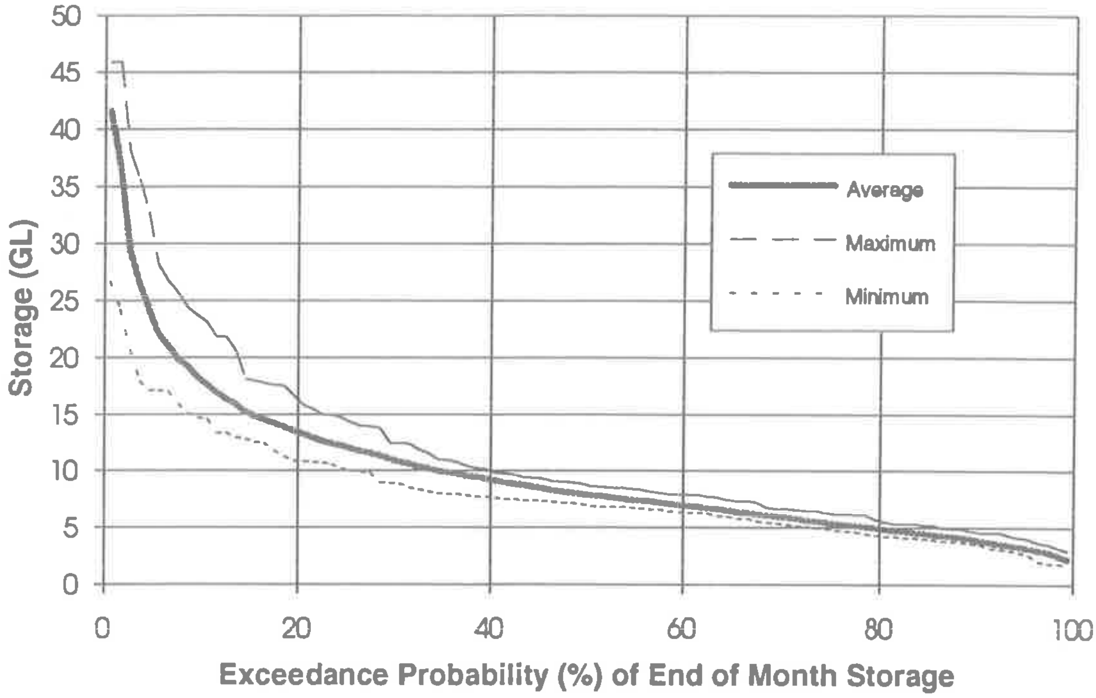


Figure I.11 End of May Storage Exceedance Probability (based upon 2000 years of synthetic data)

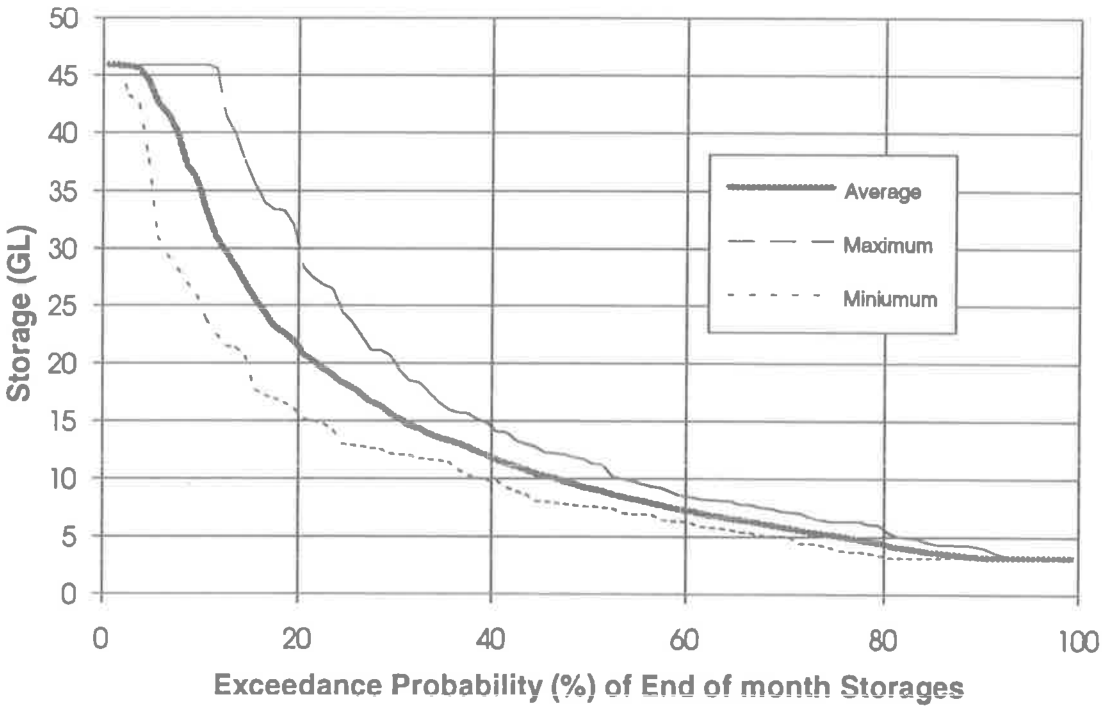


Figure I.12 End of June Storage Exceedance Probability (based upon 2000 years of synthetic data)

Appendix J
Probability of Flows Downstream of Mt Bold Reservoir

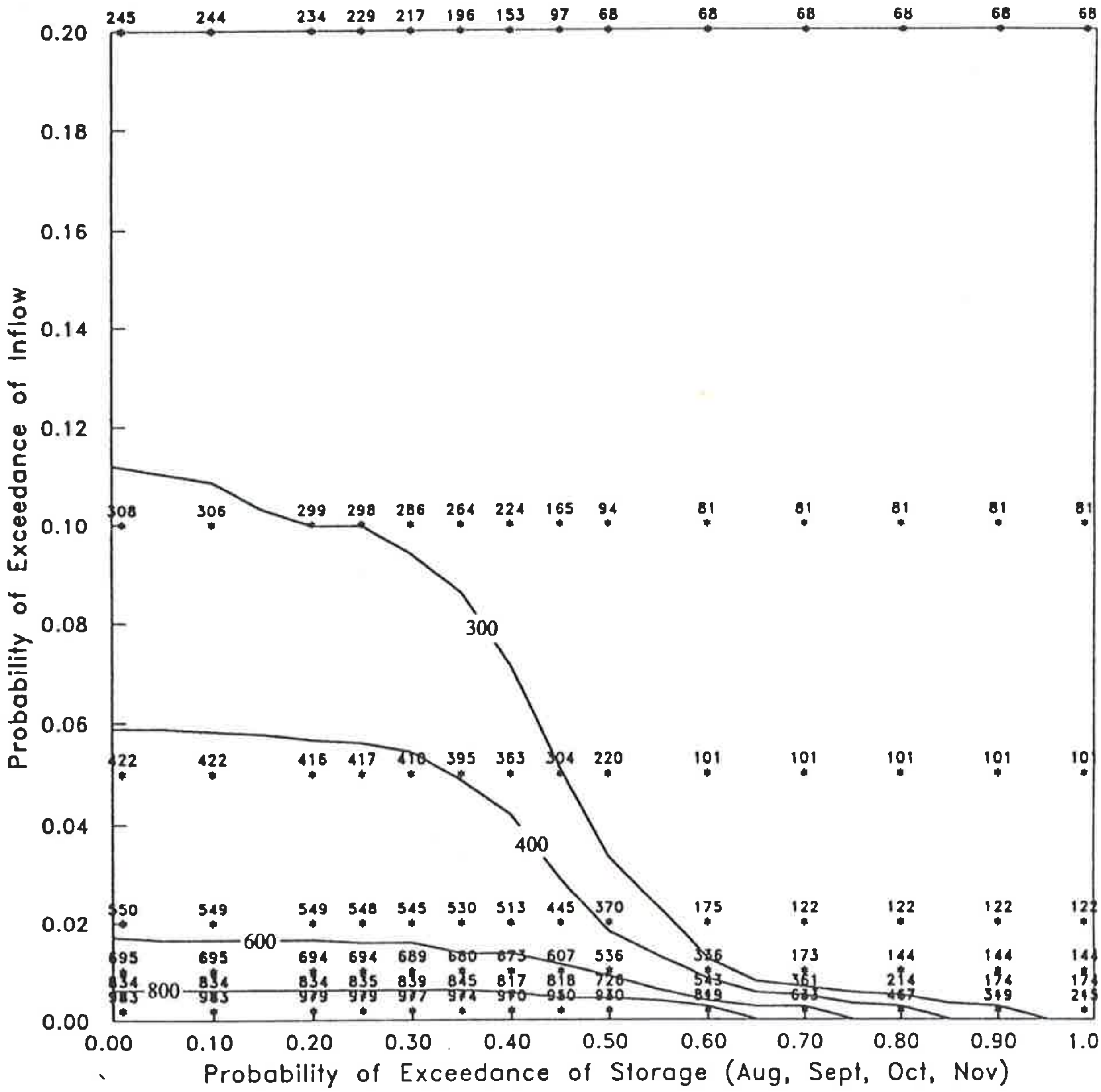


Figure J.1 Joint Probability of Outflows from Mt Bold Reservoir for the Average of the End of Month Storage Exceedance Probabilities for August, September, October and November

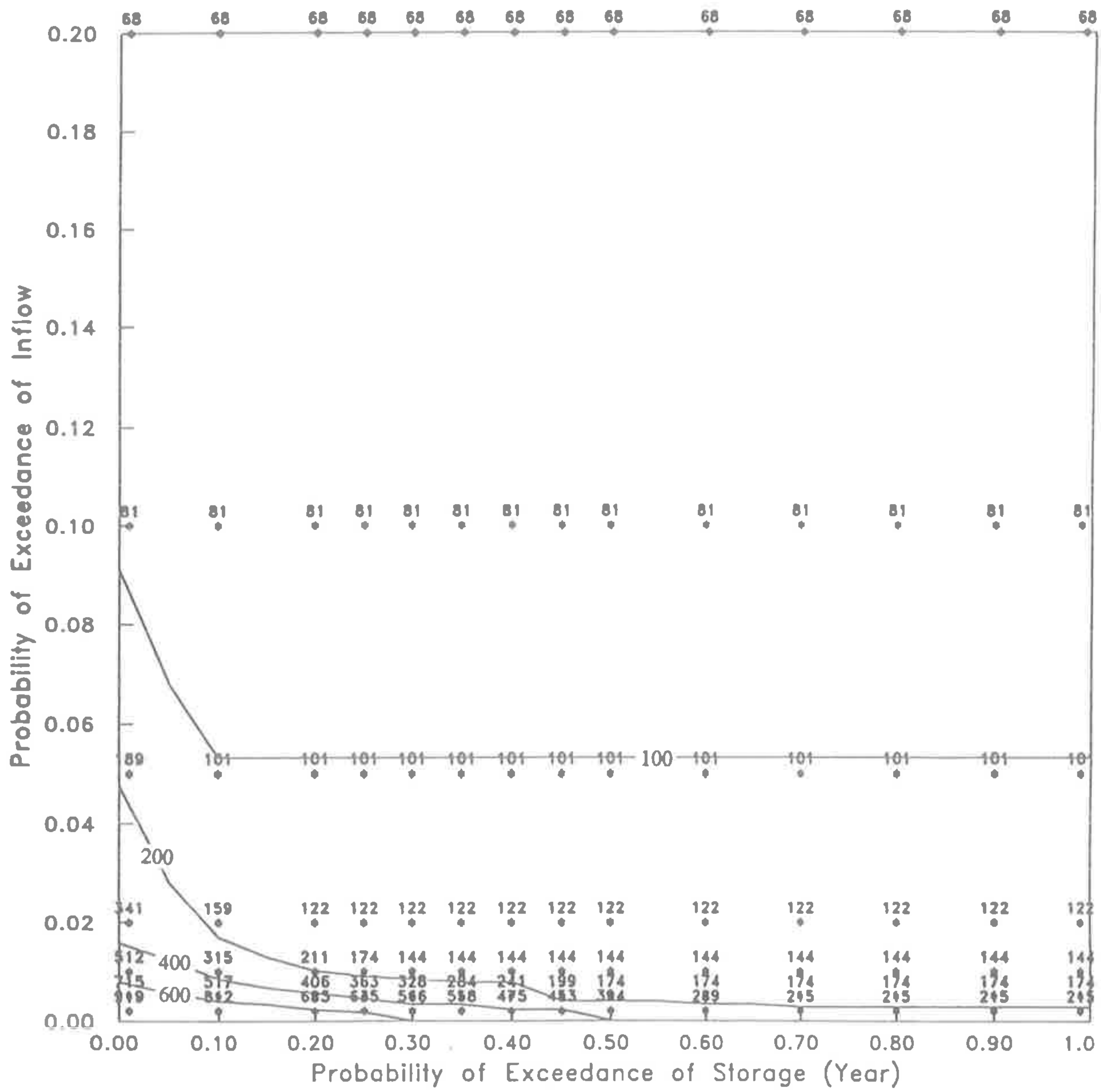


Figure J.2 Joint Probability of Flows at Clarendon Weir for the Average of the End of Month Storage Exceedance Probabilities over the Year

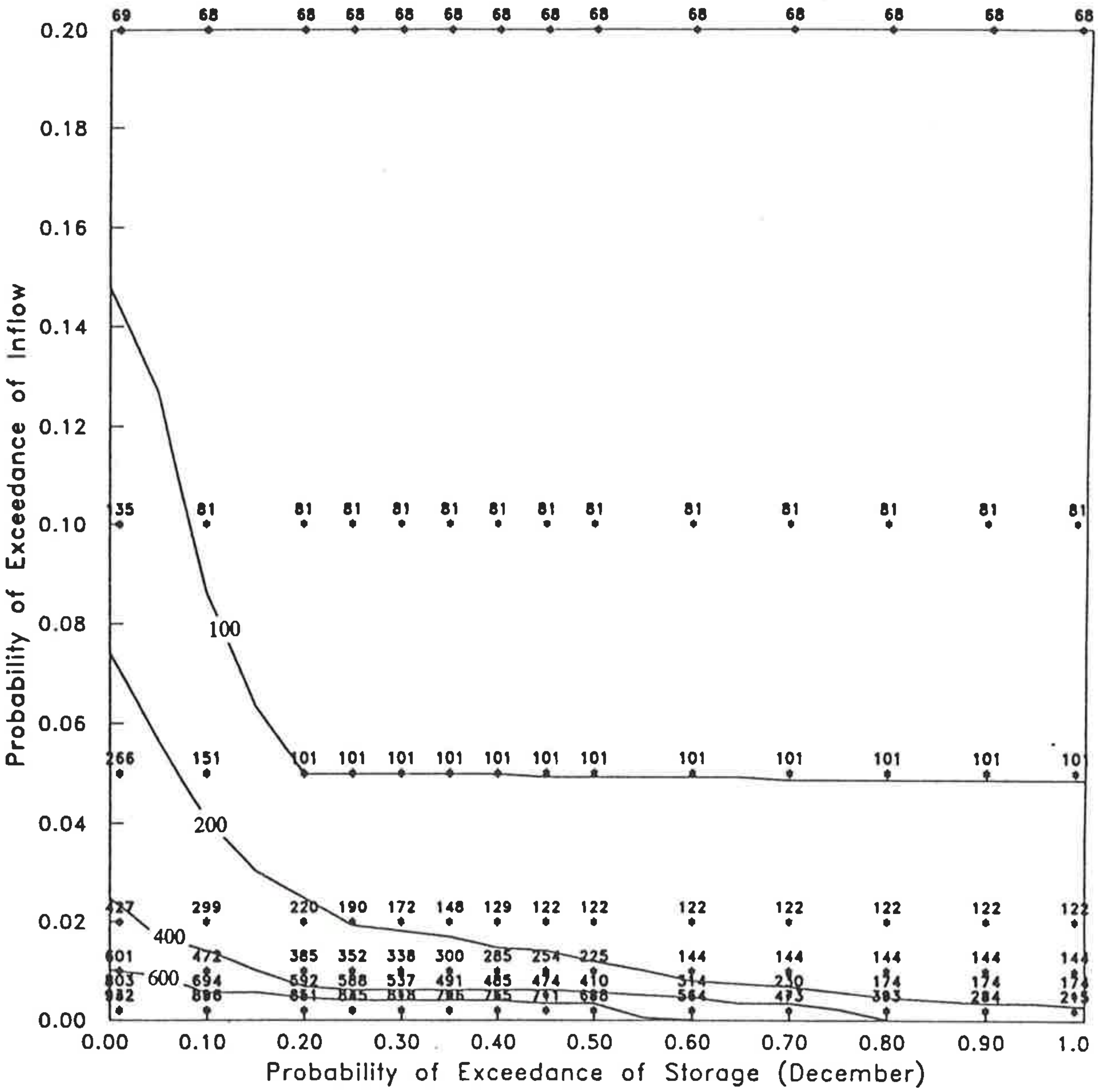


Figure J.3 Joint Probability of Flows at Clarendon Weir for the End of Month Storage Exceedance Probability for December

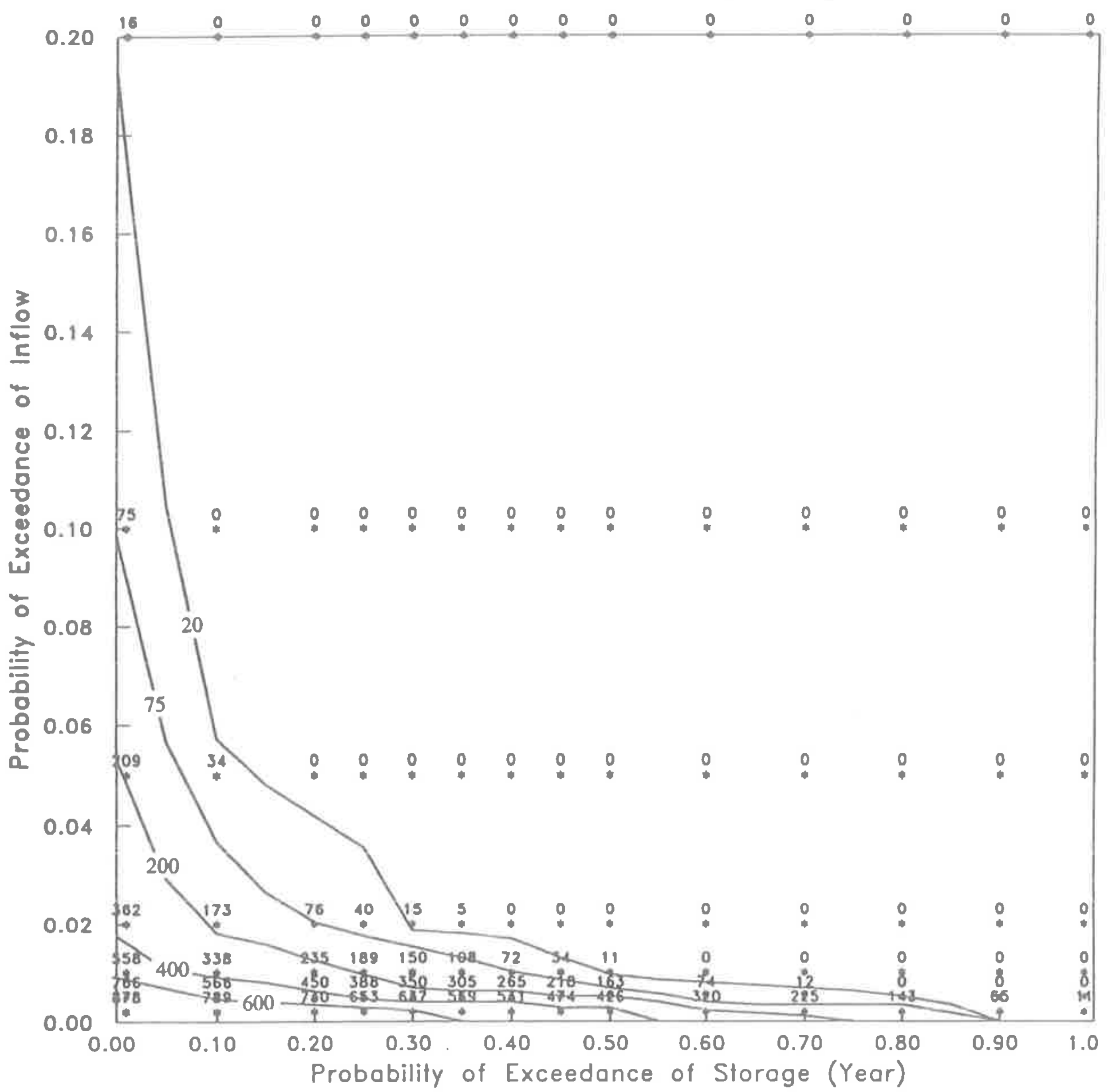


Figure J.4 Joint Probability of Outflows from Mt Bold Reservoir for the Average of the End of Month Storage Exceedance Probabilities over the Year

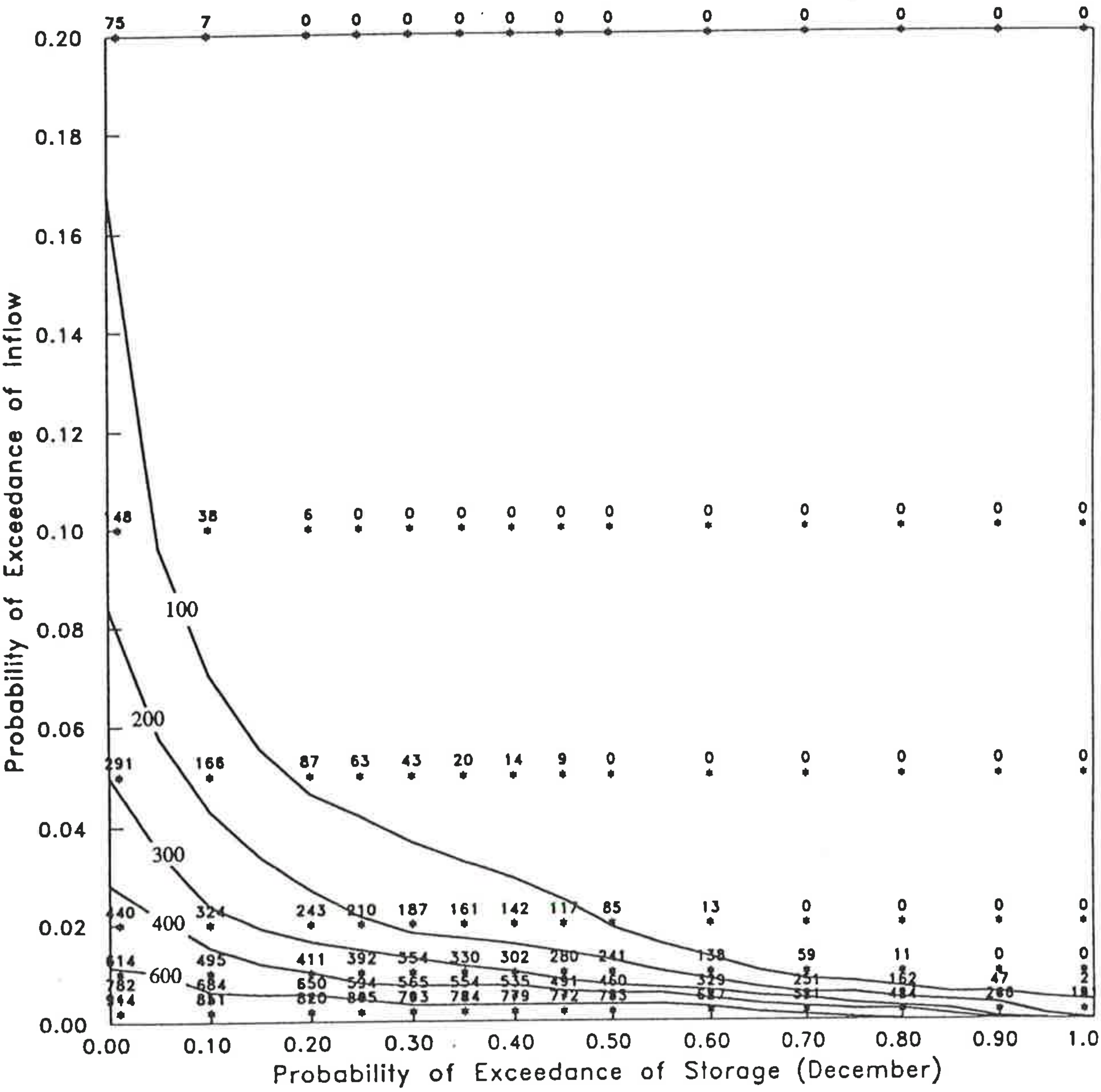


Figure J.5 Joint Probability of Outflows from Mt Bold Reservoir for the End of Month Storage Exceedance Probability for December

Appendix K
Probable Maximum Precipitation

C.M.S.S. Onkaparinga Catchment

SLOPE

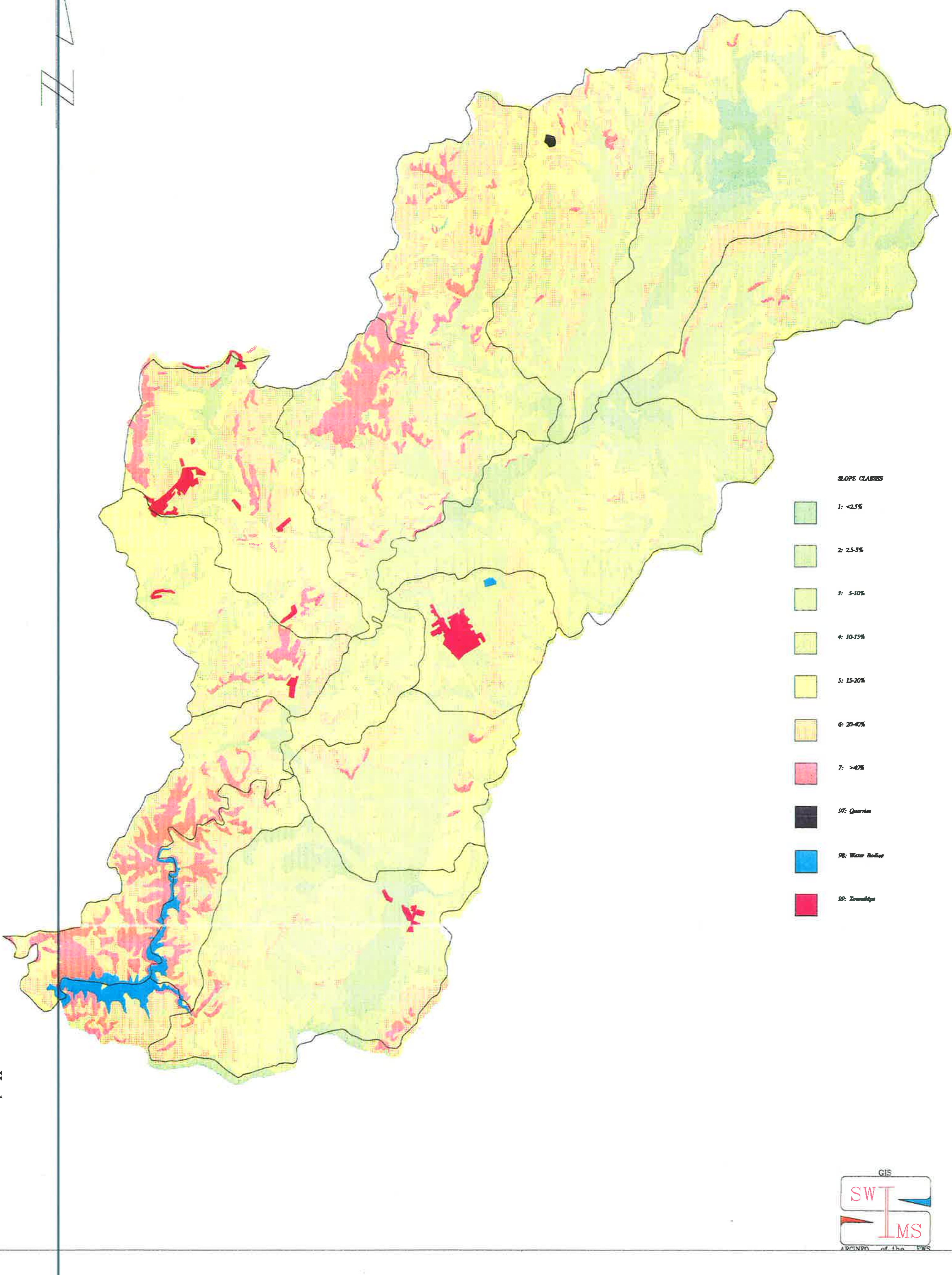


Table K.1 Preliminary PMP Depths (mm) Location 1 - Centre of Catchment

Sub catchment	Duration (hours)					
	1	2	3	4	5	6
A	89	146	189	218	239	261
B	104	167	217	254	276	298
C	178	275	343	400	429	460
D	133	205	257	309	331	360
E	158	248	308	357	386	416
F	267	394	474	539	576	609
G	376	529	617	672	710	745
H	306	443	526	587	625	658
I	306	432	514	587	625	658
J	267	394	503	539	576	609
K	104	167	217	254	276	298
L	119	178	229	266	288	311

Table K.2 Preliminary PMP Depths (mm) Location 3 - Reverse Centre of Catchment

Sub - catchment	Duration (hours)					
	1	2	3	4	5	6
A	16	29	37	65	78	99
B	36	63	81	103	124	145
C	141	225	285	330	353	383
D	73	121	161	194	216	238
E	130	208	273	310	334	357
F	266	404	490	550	589	621
G	396	548	651	718	759	793
H	375	520	620	679	719	754
I	339	485	583	647	687	721
J	297	439	540	601	621	655
K	182	289	366	420	451	483
L	182	289	366	420	458	489

Table K.3 Preliminary PMP Depths (mm) Location 4 - Towards Reservoir

Sub-Catchment	Duration (hours)					
	1	2	3	4	5	6
A	44	72	102	126	147	168
B	72	120	160	193	213	236
C	144	229	293	332	360	391
D	83	132	172	206	227	249
E	94	156	204	239	260	283
F	199	307	383	439	467	498
G	294	433	523	585	620	653
H	216	337	415	472	507	539
I	294	433	523	585	620	653
J	421	571	670	738	773	808
K	199	307	383	432	467	498
L	310	451	542	605	640	673

Table K.4 Preliminary PMP Depths (mm) Location 5 - High in Catchment

Sub - Catchment	Duration (hours)					
	1	2	3	4	5	6
A	218	347	422	479	515	548
B	218	347	422	479	515	548
C	364	464	545	607	644	671
D	281	429	508	569	606	639
E	296	447	526	588	618	652
F	364	464	606	671	709	743
G	229	364	434	492	528	561
H	166	270	330	377	399	430
I	104	176	220	256	277	300
J	83	141	184	224	245	267
K	5	18	31	45	58	78
L	5	18	31	45	58	78

Table K.5 PMP Depth (mm) Catchment to Old Noarlunga, Location 3 - Reverse Centre of Catchment

Sub - Catchment	Duration (hours)					
	1	2	3	4	5	6
A	17	32	40	70	84	105
B	40	71	88	111	133	154
C	155	252	310	355	377	405
D	80	136	175	209	230	251
E	143	233	296	334	356	377
F	292	453	532	592	628	657
G	435	614	707	773	810	838
H	412	582	673	731	768	796
I	372	543	633	696	733	761
J	326	491	586	648	663	691
K	200	323	397	453	482	510
L	200	323	397	453	489	517
M	206	330	404	460	496	531
N	103	181	229	265	286	307
O	80	136	175	209	230	251
P	29	52	74	91	105	133

Table K.6 PMP Depth (mm) Catchment to Old Noarlunga, Location 4 - Towards Reservoir

Sub - Catchment	Duration (hours)					
	1	2	3	4	5	6
A	50	83	115	140	162	183
B	81	139	179	214	235	257
C	162	264	330	370	396	425
D	93	153	193	229	250	271
E	106	181	229	266	286	308
F	224	354	430	488	514	543
G	330	500	587	651	683	711
H	243	389	466	525	558	587
I	330	500	587	651	683	711
J	474	660	752	821	852	880
K	224	354	430	481	514	543
L	349	521	609	673	705	733
M	224	354	430	488	521	557
N	87	146	186	222	242	264
O	50	83	115	140	162	183
P	12	28	29	44	59	81