Flood hydrology
for southern Africa

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SANCOLD

W.J.R. Alexander
Department of Civil Engineering, University of Pretoria

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URGENT MESSAGE TO FIRST TIME READERS OF THIS HANDBOOK

If this is the first time that you have opened this handbook, and particularly if you are referring to it because you have a problem that you have to solve in a hurry, then I urge you to read to the bottom of this page and then go straight to Chapter 14 and complete the first three case studies using the microcomputer programs, or the hand calculation methods if no microcomputer is available.

These three case studies require the application of the principal flood magnitude determination methods detailed in this handbook. When you have completed the analysis you will see the wide disparity in the answers that the different methods produce, and you will have to decide which of the answers to accept for the solution of the problems described.

The first and most important lesson to be learnt from this handbook is that there is no single calculation method that is better than all other methods under all the wide variety of flood magnitude determination problems that will be encountered in practice. Consequently you will have to apply your own experience and knowledge to your particular problem. The rest of this handbook will assist you in developing or expanding your knowledge; the case studies will help you develop your experience; and the computer programs will provide a tool for rapidly exploring a wide range of solutions - something that is seldom possible with the more laborious, time consuming and consequently more costly hand calculation methods.

THE AUTHOR

FOREWORD BY THE CHAIRMAN OF SANCOLD

The aim with this publication was to develop a handbook for hydrological calculations with specific reference to South African conditions. All aspects of hydrology are of vital importance for the planning, design, construction and operation of dams, which is the field of the International Commission on Large Dams, of which SANCOLD is an active member. More often than not, our lack of suitable data, knowledge and experience in hydrology, is the most important constraint in arriving at optimum solutions for water resource development. Mistakes in estimating hydrological characteristics are invariably costly for owners of dams, and can in extreme cases, lead to court action against the owner or the engineer. The importance of flood estimates can be illustrated by considering the effect of the September 1987 floods in Natal and the February/March 1988 floods in the Orange Free State where some 400 dams over 5 m high were damaged or destroyed. This book will not only serve dam engineering but should also be of great value to other users.

Hydrological practice in South Africa has been greatly furthered by the HYDRO Courses which were started by Will Alexander and have been presented during the past 15 years. These course notes have served as the embryo out of which this book has developed. Whilst the Hydrological Research Unit of the University of the Witwatersrand and others have contributed greatly to the development of hydrology in South Africa, there was a need for a comprehensive work which would collate available knowledge. Often, dams are not large enough or important enough to support a detailed individual study and the availability of readily accessible hydrological reference material will therefore help in this regard.

Implementation of Dam Safety Legislation has highlighted the need for a more consistent approach to flood analyses. This is often outside the speciality field of civil engineers. Will Alexander fortunately agreed to undertake the mammoth task of developing a comprehensive handbook. The handbook will supplement the SANCOLD guidelines for the safety evaluation of dams in relation to floods.

Acknowledgment is also due to SANCOLD Committee on Floods for Dams for its contribution in soliciting comments from expert reviewers and for its endeavours to accommodate the wide ranging and often conflicting viewpoints of different experts.

Whilst this handbook represents an important milestone in South African hydrological development, certain deficiencies in our knowledge were brought to the fore and research should be stimulated to fill these gaps. It is accepted that this handbook will have to be updated in future as technology advances but for the time being I am convinced that it will serve our profession well and that it will stand as a monument to Will Alexander's efforts.

T P C van Robbroeck
CHAIRMAN : SANCOLD

FLOOD HYDROLOGY
FOR SOUTHERN AFRICA

Contents

PREFACE

Challenges and responsibilities

In 1822 Dr. John Philip of the London Missionary Society established a mission station at Hankey on the banks of the Gamtoos River west of Port Elizabeth. In 1839 William Philip - John Philip's son - started the construction of a tunnel to divert water onto irrigated lands on the flood plain of the river. The tunnel was completed in 1844, but William Philip was drowned a few months later when attempting to rescue his young nephew in the flooded Klein River, a tributary of the Gamtoos.

In 1847 the tunnel collapsed during a flood which claimed 13 lives at Hankey. Further severe floods and loss of life on the flood plains of the Gamtoos River occurred in 1867, 1905, 1916, 1932, 1944, 1961 and 1970.

From 1960 to 1964 I was in charge of the construction of the water distribution project to serve the whole of the Gamtoos valley. One afternoon a minor flood occurred and I instructed the construction team to evacuate the area where they were working and to return across the river via a low concrete causeway before they were cut off by the flood. Tragically, one of the heavy vehicles used for the crossing swerved off the causeway, and six of the workmen were drowned in the river in front of my eyes.

In 1975 I was the leader of a team responsible for the routing of floods through the Vaal - Orange River system. This included the operation of Vaal Dam which has a fully gate controlled spillway and is located just upstream of Vereeniging. There was no radio telemetry network at that time and the daily procedure consisted of relaying rainfall and river flow observations taken at 08h00 telephonically from remote sites through to our operations room. By 10h00 we normally would have received all of this information plus the weather forecast and it took another two hours to process the data and determine the optimum flood control procedures for the next 24 hours. The flood had already submerged low lying areas in Vereeniging and it was clear that further areas would have to be evacuated. The officer in charge of civil defence phoned me early in the morning and insisted that an immediate decision would have to be made on the increase in the flood release from the dam if the area was to be evacuated in time. If we underestimated the required discharge and were forced to open the gates at a later stage the higher flood would cause confusion during evacuation. If we overestimated the discharge unnecessary disruption would be caused and the evacuation process of the more vulnerable areas would be jeopardized. We would have liked to have waited until we had gathered all the information necessary for an accurate estimate of the situation but this was not possible. Fortunately we made the correct decision.

Similarly, designers of structures vulnerable to flooding have to make decisions that may affect loss of life, damage to property or disruption of communication systems. These decisions have to be made within the constraints of inadequate data, diverse calculation results and insufficient time (i.e. cost to the client) and budgetary limitations (funds available for the construction of the project), and consequently require a high degree of experience based judgment.

Sound and desirable practice

Bearing in mind the competence and responsibilities expected of engineers and hydrologists involved in flood related design and operation procedures, and the potential consequences to life and property arising from inexperienced or incompetent action, some of my close colleagues have expressed concern that the procedures presented in this handbook and the computer programs in particular, may lead to abuse by inexperienced users who may assume that the methods and computer programs are error free; and can be applied directly without checking, interpretation, or further intervention by the user.

The producers of the recently revised *Australian Rainfall and Runoff - a Guide to Flood Estimation* faced the same problem and accepted that much design is carried out by engineers and others whose main expertise is in other fields, although this was not a desirable practice. Consequently their revised edition provides much firmer guidance and specific design data while at the same time encouraging users to critically evaluate the procedures published in the document and adopt new procedures where appropriate.

I agree with the Australian approach, but this handbook goes even further in that it is orientated towards microcomputer applications, and computer programs are available on disks. The programs are designed to be easy to use with emphasis on graphical and tabulated outputs that facilitate the interpretation of the results. The programs are well documented and users can modify them to meet their own requirements. Users of the computer programs should be aware that the programs are no more than numerical implimentations of calculation procedures and are subject to the same limitations as any other calculation method. Although the programs were developed over a number of years and applied to a wide range of problems, exhaustive testing of the programs is impossible. Users of the programs are expected to make final evaluations as to the usefulness and correctness of the programs.

I firmly believe that just as engineers in the field cannot avoid the responsibility of having to make decisions which may affect the lives of their staff; and those responsible for dam operation and flood routing cannot avoid the responsibility or the consequences of their actions on the community at large; and designers cannot avoid responsibility for the safety of their structures; so too authors of handbooks such as this cannot place the responsibility on the user to determine whether or not the methods detailed represent sound and desirable practice. Methods which do not meet these criteria have no place in the handbook. The responsibility of the user is to determine whether or not the methods are applicable to the problem at hand, and where necessary to develop the methods further, or apply other methods not detailed in the handbook, or to seek expert advice. Above all, it is the user's responsibility to interpret the results of the analysis and if necessary to modify them in the light of his own experience and judgment.

The analytical procedures detailed in Chapter 9 of this handbook represent sound and desirable engineering practice in southern Africa subject to the proviso that the results are interpreted with due care. This does not imply that other methods are necessarily less suitable, but if they are used then an additional onus is placed on the user to satisfy himself and possibly others of the validity of his approach.

Nor is it implied that the recommended methods cannot or should not be improved or new methods introduced. On the contrary, much research remains to be done, and the recommendations in this handbook will need to be reviewed and updated from time to time.

Sources

The sources referred to during the preparation of this handbook were the publications of the South African Department of Water Affairs, the South African Weather Bureau, the University of the Witwatersrand Hydrological Research Unit, the UK National Environmental Research Council's Flood Studies Report; the USA Interagency Advisory Committee on Water Data's Guidelines for Determining Flood Flow Frequency; the Australian Institution of Engineers' Guide to Flood Estimation; publications of Environment Canada; discussions with experienced hydrologists and engineers in South Africa and overseas; and numerous research publications. The principal publications are referenced at the end of the chapters where they are quoted.

Because personal experience and judgment are required in many aspects of flood risk estimation it is inevitable that there will be differences in opinion on the merits of the different procedures used in flood risk analysis among practitioners as well as between practitioners and researchers. In this handbook the greatest weight has been given to the publications of the national agencies listed in the previous paragraph followed by publications based on systematic analyses of large recorded data sets. Less weight has been given to theoretical approaches based on synthetically generated data sets, and process models based on small catchment observations. There are many interesting research papers which hold promise for future applications but until they have been tested in practice and the results published their conclusions must be treated with reserve, and consequently they are given little weight in this handbook.

Acknowledgments

This handbook is based on a series of lecture notes produced for courses on flood hydrology that were presented jointly by the Department of Water Affairs and the University of Pretoria under the auspices of the South African Institution for Civil Engineers during the period 1976 to 1988. These courses have been attended by more than 1 200 participants from several southern African countries to date. The lecture notes were prepared when I was in the service of the Department and I am indebted to my former colleagues Zoltan Kovåcs, Peter Adamson, and Amelius Muller for their comment and assistance at that time. Since then the notes have been completely updated, revised and re-edited.

The permission of the Department of Water Affairs, the Weather Bureau and the University of the Witwatersrand to include drawings and other material from their publications is gratefully acknowledged.

I must record my appreciation to the South African National Committee for Large Dams (SANCOLD) who initiated the production of this handbook, and the Department of Civil Engineering of the University of Pretoria who provided all the facilities. SANCOLD provided a grant to cover the initial costs, while the additional costs were met from income derived from the short courses.

This handbook has been reviewed by a panel of experienced engineers who provided critical comment which is gratefully acknowledged. Errors that were identified have been corrected but users should be aware of the possibility of further undetected errors being present in the document, including possible errors in the equations, graphs or computer programs. I express my appreciation to the following SANCOLD committee members and reviewers who reviewed or commented on the whole or parts of the handbook during its preparation. They are P.T.Adamson, W.S.Croucamp, Z.Kovåcs, W.C.S.Legge, D.C.Midgley, G.C.S.Pegram, H.N.F.Pels, A.Rooseboom and M.Shand.

I wish to express my gratitude to Mrs. Nellie le Roux who did most of the typing and secretarial work, and my three student assistants Rietta Wolmarans, Hugo Lotriet and Francois van Aswegen who helped in many ways. Finally, I thank my wife for her patient support during the many months when I was desk bound.

W.J.R. Alexander
University of Pretoria
January, 1990

Chapter 1

INTRODUCTION

Contents

1.1 HISTORICAL REVIEW

1.1.1 Southern Africa

In 1919 the Department of Water Affairs (then Department of Irrigation) issued the first paper *Maximum flood curves* in its Professional Paper series. In his foreword A.D. Lewis the Director of Irrigation wrote :-

> *Too much importance must not be attached to the formula. No formula is likely to be discovered which will apply to all drainage areas. The maximum flood depends on too many uncertain circumstances, such as intensity of rainfall, size and shape of catchment and channel, and permeability of ground surface.*

Now, some seventy years later Lewis' statement is still valid. Despite the collection of a vast amount of hydrological data in South Africa as well as elsewhere in the world, there is still no universally applicable method for flood risk determination.

Subsequent issues in the Professional Paper series saw the transition from the pure empirical method proposed in Professional Paper No. 1 to the statistically based method proposed by Roberts in Professional Paper No. 20 published in 1963 and his Technical Report 33 of 1965 *The empirical determination of flood peak probabilities.* In 1968 Herbst developed this method further in Technical Report 46 *Flood estimation for ungauged catchments.*

The Hydrological Research Unit of the University of the Witwatersrand at the request of the South African Institution for Civil Engineers embarked on an extensive research programme which resulted in the publication of Report 4/69 *Design flood determination in South Africa* in 1969. An updated and metricated version was published in 1972 (Midgley, 1972) which was further updated in 1979. Although this report covered most methods then available, the emphasis was on deterministic methods, more particularly the unit hydrograph method.

The report was based on an extensive analysis of the available point rainfall, large area storms and peak discharge data and is a landmark study in southern African flood hydrology. Since then the data base has almost doubled, and some of the procedures can be improved in the light of new information.

This report (HRU 1/72) will be referred to frequently in this handbook. At the time of its preparation the authors cautioned that a number of the procedures were of necessity tentative because of serious gaps in the available information. The data base been considerably expanded since then and alternative flood magnitude estimation methods have become available. Where some of the procedures in HRU 1/72 have been criticized in this handbook the intention is solely to alert readers to the shortcomings, suggest alternative procedures where these are available, and to emphasize the point that *all* methods have their particular strengths and weaknesses.

In 1973 Herbst *et al* (1973) presented their paper *Flood estimation by determination of regional parameters from limited data* at an international conference in Madrid on the design of water resources projects with inadequate data.

In 1978 Adamson (1978) of the Department of Water Affairs reviewed the statistical methods used for flood frequency analysis.

Also in 1979 Hiemstra and Francis of the University of Natal produced their report on the runhydrograph method for flood prediction, while their colleagues Schulze and Arnold in the Department of Agricultural Engineering of the same university, produced a report on the application of the US Soil Conservation Service method to flood prediction in small catchments. The final version was published in 1987 (Schmidt and Schulze, 1987).

In 1980, Z.P. Kovâcs of the Department of Water Affairs developed the Regional Maximum Flood method for estimating maximum expected peak flows. This was updated in 1989.

1.1.2 Overseas historical developments

The rational method developed by Mulvaney in Ireland in 1851 and the unit hydrograph method developed in the USA in 1932 by Sherman remain the two principal deterministic methods based on rainfall-runoff relationships. Variants of both of these methods are still in use in many countries including South Africa.

Direct statistical analysis methods have been available since early in this century. The flood plains of the major rivers in the USA are extensive and are prime development areas. There was a need to balance flood plain development with the risk of inundation and associated insurance cover. Arising from this and other considerations the United States Water Resources Council (1972) perceived an acute need for an approach to estimates of flood damage potential that would provide consistent and accurate estimates. The results of the investigation indicated that only the log normal distribution and the log Pearson Type 3 distribution with regional skew were free of substantial bias. The log Pearson distribution was recommended for use by all Federal agencies in the USA. The latest revision of the guidelines was published by the US Interagency Advisory Committee on Water Data in 1982.

Another development in the USA resulted from several major dam failures which caused loss of life and severe damage. This gave rise to the probable maximum flood concept for dam spillway design. Although this approach was severely criticized, the need for a close to zero risk flood peak value is still required for the design of dams and other hydraulic structures whose failure may cause loss of life. The method is based on the estimated probable maximum precipitation.

In 1975 the National Environmental Research Council of the UK published the Flood Studies Report which is the most comprehensive investigation to date. The objective was to develop a direct statistical analysis based method that could be applied at any site in the

UK, whether or not records were available. The general extreme value family of distributions was used together with a statistical correlation model for determining the mean annual flood peak.

In 1977 the Australian Institution of Engineers published its report on Flood Analysis and Design. Unlike the USA and UK approaches, the Australian approach was to make a deliberate attempt to eliminate the code of practice concept on the basis that this would hinder the application of sound, creative and efficient engineering concepts to hydrological analysis and design. The publication was a handbook of information and hydrological methods. The revised edition was published in 1987 (Pilgrim, 1987) after a complete review of the subject. A policy change was made in the revised edition which provided more specific guidance to designers.

A more recent development is the publication of a catalogue of world maximum floods and the development of an empirical method to categorize them (Rodier and Roche 1984). Earlier versions of the method led to the development of the Regional Maximum Flood method in South Africa.

None of the publications mentioned above purported to be the final word on the subject of flood risk determination, and all foresaw the need for further research and development.

1.1.3 Dam safety

Following the recommendations of the International Committee on Large Dams (ICOLD), investigations related to the broader issue of dam safety gathered momentum in the 1980's, and several countries including South Africa have published guidelines relating to the hydrological aspects, or are in the process of doing so. Emphasis was on risk evaluation and hydrological design standards.

1.1.4 Approach used in this handbook

South Africa has a very wide range of climatological and consequently of hydrological conditions. There have not been any comprehensive studies on the scale of the UK Flood Studies Report that are necessary for an impartial assessment of the various methods of flood frequency analyses. There is also not the need to produce a single recommended method of analysis as was the case for flood plain development in the USA.

Because of the high degree of experienced judgment required in the application of many of the methods, appropriate details of the theoretical background have been included in this handbook so that users may develop a particular method further should this be necessary.

The one aspect where this handbook differs from the other publications is the emphasis on microcomputer applications. Although main frame computers were used for the development of most of the methods published after the mid-1960's, the results were produced in the form of tables and graphs as most of the potential users did not have access to these computers. The introduction of readily available, powerful microcomputers has changed the situation. Users of this handbook are provided with information that will enable them to undertake the analyses with pocket calculators; or make use of the set of computer programs provided with this handbook; or write their own computer programs.

1.2 STRUCTURE OF THIS HANDBOOK

The handbook has been structured to facilitate its use as a reference for the casual user who may wish to solve a simple problem by using one of the computer programs, through to the user faced with a complex problem that requires further development of one or more of the methods described here, and as a reference source and point of departure for researchers.

The handbook is divided into four parts. The theoretical development is described in Part 1, the practical implementation in Part 2, flood routing, control of flood plain development and dam safety in Part 3, and computer programs and case studies in Part 4.

The casual user with a simple problem will only need to read through Chapter 13 in conjunction with the instructions on the computer screen when running the programs.

Those requiring more detailed information on the implementation of a particular method can study the description in the relevant chapter in Part 2, while users who have an interest in the theoretical background will find it in Part 1.

For any specific method - for example the application of the log normal distribution - the theoretical development will be found in Part 1, Chapter 2; the application in Part 2, Chapter 5; information on the application of the various methods in Part 4, Chapter 13; and case studies in Part 4, Chapter 14.

Statistical analysis methods require a basic knowledge of hydrological statistics with emphasis on the most often used probability distributions. As this information is not readily available in a form which can be applied directly to hydrological analyses, and also because of the dominant role of statistical methods in flood frequency analyses, this is discussed in some detail in Chapter 2.

A reasonable knowledge of the depth-area-duration-frequency relationships of storm rainfall is essential for the application of deterministic methods. These relationships are detailed in Chapter 3.

Catchment characteristics are required for the deterministic methods, as well as the regionalized statistical analysis methods. These are dealt with in Chapter 4.

The various methods for flood frequency estimation and the basis for determining the design flood are detailed in Chapters 5, 6, 7, 8 and 9.

Many of the major dams in South Africa are equipped with flood control gates. Flood routing procedures required for the operation of gates to meet the dual criteria of ensuring the safety of the structure, as well as the minimization of downstream flood damage are discussed in Chapter 10.

There are many instances where floods caused loss of life and severe damage to property within river flood plains. Criteria for controlling flood plain development are detailed in Chapter 11.

Regulations covering design requirements to ensure the safety of dams in South Africa were promulgated in 1986. Information related to dam safety criteria is given in Chapter 12 together with comment on the spillway design flood.

Details of the recommended calculation methods and guidance on the use of the accompanying computer programs is given in Chapter 13.

Case studies are grouped together in Chapter 14 as many of the examples include the application of more than one method or application.

The essence of the handbook is to be found in Chapter 9 on the design flood, Chapter 13 which contains the calculation procedures and computer programs and Chapter 14 on case studies. The remaining chapters are devoted to reference material and more specific applications.

Examples are grouped together in Chapter 14 rather than spread through the rest of the handbook. This facilitates comparison of the different methods and emphasizes the main theme of this handbook which is that there is no single optimum method and that all applicable methods should be used and the results compared and evaluated on the basis of experienced judgment.

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Chapter 2

HYDROLOGICAL STATISTICS

Contents

Note:

The figures and tables in this chapter should not be used for calculation purposes. Those that are recommended for inclusion in calculation methods are repeated in the appendix to Chapter 13.

2.1 INTRODUCTION

"Lies, damned lies and statistics"
George Bernard Shaw

Figures cannot lie, but liars can figure.
C. Northcoate Parkinson

Public agencies are very keen on amassing statistics - they collect them, add them, raise them to the nth power, take the cube root and prepare wonderful diagrams. But what you must never forget is that every one of those figures comes in the first instance from the village watchman, who just puts down what he damn pleases.
Sir Josiah Stamp as quoted in Wonnacott and Wonnacott (1972).

Statistics is concerned with scientific methods for collecting, organizing, summarizing, presenting and analyzing data, as well as drawing valid conclusions and making reasonable decisions on the basis of such analysis.
Spiegel (1961)

The Department of Water Affairs which is the public agency in South Africa responsible for the acquisition, processing and dissemination of hydrological data, has an impressive hydrological data base. Some 20 000 station years of data for river flow are available from some 800 stations with an average of more than 30 years of record per station.

The basic issue in flood hydrology is the estimation of the probable magnitude of future events based on historical observations. As it is extremely unlikely that the historical records will be repeated in future, the statistical properties of the past records have to be examined and from these estimates of the likelihood of events of given severity occurring during the economic lifetime of the project have to be made.

The most powerful tool available to hydrologists for this purpose is that of statistical analysis. The reason for the earlier mistrust of statistics was the failure on the part of the analysts to appreciate the weakness as well as the power of the basic statistical methodology.

Another reason for continuing mistrust is the differences in nomenclature and approach which are inevitable when methods and procedures are applied in several different disciplines. This is particularly so in the field of statistics where no two authors appear to use the same nomenclature, and even the form of the equations used for the same function may be different.

These notes will provide those who may have no knowledge of statistical methods with the information that they will require to carry out routine hydrological analyses.

The notation used in this chapter is internally consistent and conforms with that used in the UK *Flood Studies Report* (NERC 1975) except where stated otherwise.

2.2 PROBLEM DEFINITION

2.2.1 Exceedance Probability

The designer has to select the design flood from a whole series of possible values, each of which is associated with a probability of being *equalled or exceeded* during the effective life of the structure. The design flood is therefore a threshold value in a whole spectrum of values. The probability that a flood peak of exactly this value will occur is infinitesimally small, but its position in the spectrum of values is readily calculated.

The relationship between the severity of the event and the probability of the event being equalled or exceeded during the design lifetime is given by :

$$r = 1 - \left[1 - \frac{1}{T} \right]^{L} \qquad (2.1)$$

where r is the risk of an event having a return period T years occurring at least once during the design life L.

This relationship is developed further in Chapter 9 on the design flood.

2.3 DATA ACQUISITION

2.3.1 Systematic errors

The quotation from Josiah Stamp at the beginning of this chapter should not be ignored. The statistical analyses and the conclusions that are drawn from them can only be as accurate as the data on which they are based. Data published by the public agencies will have gone through a checking procedure but may still contain errors. The agency producing the data should be contacted in case of doubt.

If the problem is such that more sophisticated analyses are required then the first step should be a re-evaluation of the data. For example, the accuracy with which flood peaks can be measured decreases with increase in flow, and systematic errors may be introduced which may result in the calculated values being consistently higher or lower than the true values.

2.3.2 Assumptions made in statistical analyses

The fundamental assumptions in the analyses described in this chapter are that :

(a) Each observation is independent of previous and subsequent observations. This is reasonable for annual flood maxima.

(b) The data are free of measurement errors. This should be tested in all cases where the design of major structures is involved, particularly the possibility of systematic errors occurring where peak flows are well in excess of the capacity of the measuring structure.

(c) That the data are identically distributed ie that they can safely be assumed to come from a single parent population, which in turn implies a single type of rainfall producing meteorological phenomenon. This is clearly not the case in areas subject to infrequent tropical cyclones for example. Where this possibility exists, special treatment is required. Analysts should always be aware of the strong possibility that many of the apparent anomalies in statistical analyses arise from the mixture of different meteorological phenomena and different states of antecedent conditions that determine the magnitude of the runoff events.

2.4 PROPERTIES OF DATA SETS

2.4.1 Introduction

The basic methodology is illustrated in the following example :

A flow gauging station was built in the Vaal River at Standerton in 1905, and flow measurements were available from then until 1975 when the Grootdraai Dam was built just upstream of the gauging station. An estimate is required of the relationship between the magnitude of flood peaks and their annual exceedance probability (AEP).

Table 2.1 shows the maximum flood peak for each year of record. These have also been ranked from largest to smallest. For design purposes an estimate of the magnitude of floods that may occur over a range of exceedance probabilities is required. The historical record has to be used to determine these values.

Figure 2.1 shows the method used in the determination. Figure 2.1(a) is the histogram of the annual maximum floods in the sequence in which they occurred. Figure 2.1(b) shows the proportional number of occasions (years) when the annual maximum was within the ranges delineated by the horizontal lines in the figure. This diagram represents frequency of occurrence of the peaks within these intervals. Figure 2.1(c) is derived from Figure 2.1(b) and is the cumulative frequency ie the frequency with which the values delineated by the horizontal lines are equalled or exceeded, and is the cumulative sum of the values in Figure 2.1(b).

Figure 2.1(d), 2.1(e) and 2.1(f) are replicates of the figures above them except that base 10 logarithms of the annual flood peaks are used instead of the untransformed values.

TABLE 2.1 Vaal River at Standerton : Flow record and statistical properties.

Flow record		Ranked flow record		
Year	m^3/sec	#	m^3/sec	Year
5/ 6	264	1	2286	9/10
6/ 7	1260	2	2059	22/23
7/ 8	558	3	1939	73/74
8/ 9	1417	4	1823	55/56
9/10	1144	5	1546	44/45
10/11	2286	6	1460	38/39
11/12	240	7	1417	17/18
12/13	113	8	1417	8/ 9
13/14	117	9	1260	6/ 7
14/15	396	10	1171	21/22
15/16	486	11	1150	71/72
16/17	321	12	1144	9/10
17/18	1417	13	1020	56/57
18/19	452	14	984	66/67
19/20	90	15	937	55/56
20/21	767	16	864	33/34
21/22	1171	17	767	20/21
22/23	2059	18	761	29/30
23/24	120	19	750	36/37
24/25	700	20	738	35/36
25/26	100	21	706	40/41
26/27	194	22	700	24/25
27/28	154	23	681	64/65
28/29	368	24	680	52/53
29/30	761	25	613	60/61
30/31	122	26	600	57/58
31/32	139	27	597	39/40
32/33	55	28	558	7/ 8
33/34	864	29	547	58/59
34/35	229	30	523	49/50
35/36	738	31	486	15/16
36/37	750	32	475	37/38
37/38	475	33	452	18/19
38/39	1460	34	421	61/62
39/40	597	35	416	45/46
40/41	706	36	396	14/15
41/42	200	37	368	28/29
42/43	291	38	335	51/52
43/44	1546	39	331	69/70
44/45	211	40	331	47/48
45/46	416	41	321	16/17
46/47	238	42	291	42/43
47/48	331	43	285	63/64
48/49	120	44	271	62/63
49/50	523	45	265	59/60
50/51	197	46	264	5/ 6
51/52	335	47	240	11/12
52/53	680	48	238	46/47
53/54	154	49	229	34/35
54/55	1823	50	211	44/45
55/56	937	51	200	41/42
56/57	1020	52	197	50/51
57/58	600	53	194	26/27
58/59	547	54	169	67/68
59/60	265	55	154	53/54
60/61	613	56	154	27/28
61/62	421	57	144	68/69
62/63	271	58	141	73/74
63/64	285	59	139	31/32
64/65	681	60	137	70/71
65/66	38	61	122	30/31
66/67	984	62	120	48/49
67/68	169	63	120	23/24
68/69	144	64	117	13/14
69/70	331	65	113	12/13
70/71	137	66	100	25/26
71/72	1150	67	90	19/20
72/73	23	68	55	32/33
73/74	141	69	38	65/66
74/75	1939	70	23	72/73

Statistical properties : natural data (m^3/sec)

Median annual peak	: 416
Mean annual peak	: 584
Standard deviation S	: 529
Coeff. of variation	: 0.91
Skewness coeff.	: 1.41

Statistical properties : log transformed data

Mean annual peak	: 2.581
Standard deviation	: 0.432
Coeff. of variation	: 0.17
Skewness coeff.	: -0.35

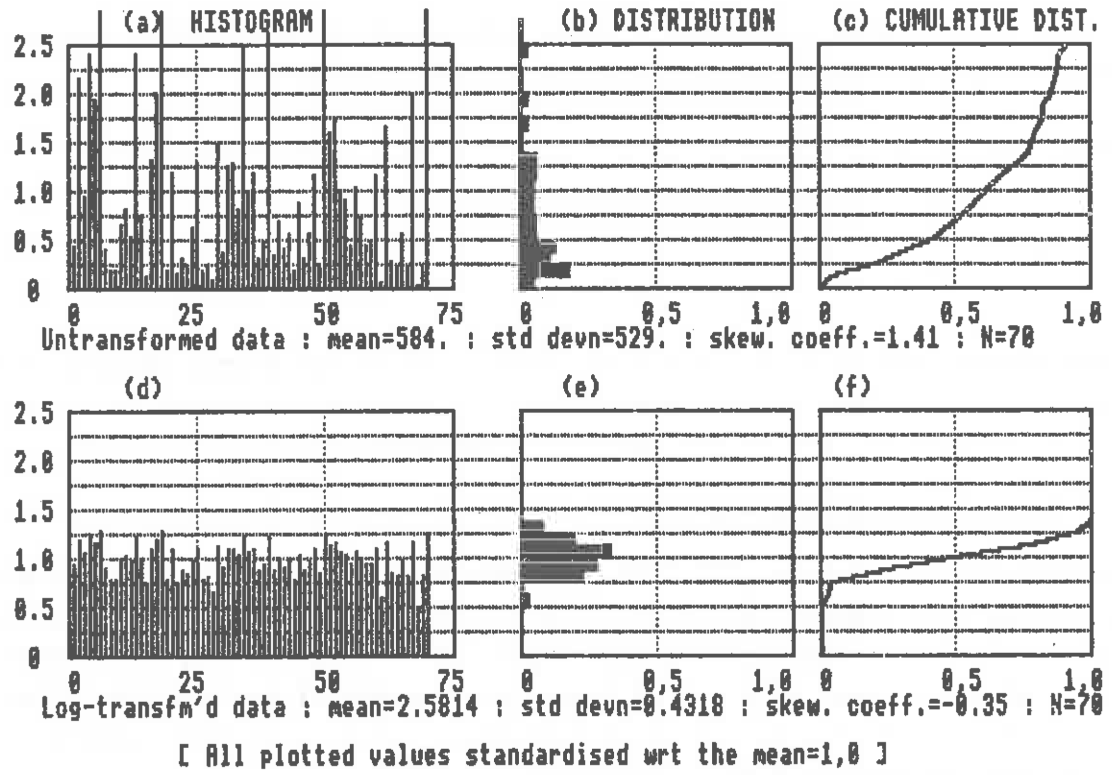

FIGURE 2.1 Determination of the statistical properties of the annual flood peak series of the Vaal River at Standerton.

The essence of statistical analysis of a data set is the determination of the mathematical equations which best fit the data as presented in Figures 2.1(b), 2.1(c), 2.1(e) and 2.1(f), and then use these equations to derive the relationship between the magnitude of an event and the probability of it being equalled or exceeded in any one year. Given this probability the corresponding exceedance probability during the design life is readily calculated.

The mathematical equations which best describe these distributions cannot be determined on theoretical grounds alone, and despite the investigations and proposals over the past century, no universally accepted models have yet been developed, and research and accompanying debate continues. This will be discussed in more detail later.

Figures 2.1(b) and 2.1(e) above are a useful starting point for describing the properties of a data set. The first and most important property of the data set is its mean value. The second is the degree of dispersion of the individual data values about the mean (variability), and the third is the manner in which the values are dispersed about the mean (skewness). For example, in Figure 2.1(e), the logarithms are almost symmetrically dispersed about the mean value of 1,0 while the untransformed values are strongly asymmetrical.

2.4.2 Determination of the mean value

The mean (mean annual flood peak) is simply determined by adding all the values and dividing the sum by the total number of observations. The equation is:

Mean :

$$\bar{x} = \frac{\Sigma x}{N} \tag{2.2}$$

A little thought will show that this is the same as taking the average moment of the histogram values about the origin ($x = 0$).

2.4.3 Determination of variability

It is possible that two rivers may have the same mean annual flood peak, but one (say in the Karoo) would have a much larger range of values than another (say in Natal). This range can be measured by determining the spread of values about the mean. The second moment, this time about the mean, ie $\Sigma(x - \bar{x})^2$ is used as a measure of the variability. This is incorporated in the equation for the standard deviation s which is the most commonly used estimate of the variability of a set of observations.

Standard deviation :

$$s = \left[\frac{\Sigma(x - \bar{x})^2}{N - 1} \right]^{1/2} \tag{2.3}$$

2.4.4 Determination of skewness

The third moment about the mean $\Sigma(x - \bar{x})^3$ can be used to determine the skewness of the set of observations.

Skewness coefficient :

$$g = \frac{N}{(N-1)(N-2)} \quad \frac{\Sigma(x-\bar{x})^3}{s^3} \tag{2.4}$$

2.4.5 Higher moments

The fourth moment is a measure of kurtosis or flatness of the peak and consequently thickening of the tails of the distribution. This and the higher moments are not normally used in the analysis of single data sets because the sample size (number of observations) is too small for reliable estimates. Where they are required for regional analyses, other methods are used for their estimation.

2.4.6 Other measures and methods

The mean, standard deviation and skewness coefficient are not the only measures of location, scale (dispersion) or shape, nor is the method of moments the only method for determining their values. The maximum likelihood method is a strong contender as an alternative for determining these values. In some distributions, including the normal distribution, the method of moments and the method of maximum likelihood will give identical results. In other cases the differences will be small and there is little justification for their use, bearing in mind the additional computational effort and inaccuracies in the data. Other measures of location, scale and shape will be described as they occur below, but in most distributions of hydrological interest their values can be derived from the three properties given above.

2.5 PROBABILITY DISTRIBUTIONS

2.5.1 Distribution functions

Figures 2.1(a) and 2.1(d) are histograms of the data sets; Figures 2.1(b) and 2.1(e) are histograms of the frequencies of occurrence of the data within the specified ranges; and Figures 2.1(c) and 2.1(f) are the cumulative frequencies of occurrence, ie the frequencies with which the data values exceed the specified values defined by the horizontal lines.

If the midpoints of the rows of the frequency histograms in Figures 2.1(b) or 2.1(e) were joined the result would be a frequency polygon. A mathematical equation which best fits this polygon is called a probability density function (pdf) while the equation which fits the cumulative frequency polygon is called a cumulative distribution function (cdf) or simply the distribution function.

If the frequencies in the histograms were divided by the number of observations, ie the frequencies were expressed as a proportion of the total number of observations, then the total of the individual frequencies in Figures 2.1(b) and 2.1(e) would be unity and their cumulative totals would also be unity. An important property of the pdf is that the area beneath the curve must be unity, and similarly the cdf must have an exceedance probability value in the range from zero to one. The cdf is the integral of the pdf.

There are a number of mathematical expressions which have an area under the curve of unity and shapes which are similar to the frequency polygons of hydrological data sets.

2.5.2 Orientation

One thing that practically all texts on statistics have in common is the orientation of the figures showing the various pdf's. This follows the convention of having the magnitude on the horizontal axis (abscissa) and the probability on the vertical axis (ordinate). This is understandable as the pdf itself is usually the centre of attention. However, the conventional orientations for the histograms from which the pdf's are derived, and the cdf's which result from them, both have the magnitudes on the vertical axes. The logical orientation of all three presentations is that shown in Figure 2.1 as this clearly shows their derivation and the inter-relationships.

The orientations in the next section follow the logical rather than the conventional approach. The reason is that this will greatly facilitate the understanding of the equations that are used for analyzing the data; presenting the data graphically; and generating synthetic data sequences. It will also encourage the use of visual presentations which are excellent controls of the accuracies and assumptions used in the analyses, and are easily incorporated in microcomputer programs.

Comment below is limited to a general description of the distributions. Further details of the distributions and methods for applying them are given in the appendices to this chapter.

2.5.3 Uniform distribution

This results when the data are uniformly distributed over an interval with upper and lower limits. Although of no practical value in hydrology it is the basis for generating synthetic sequences in all the other distributions of hydrological interest.

2.5.4 Normal distribution

The normal distribution was first developed by de Moivre in 1753, and a century later was used to explain errors in astronomical measurements and derive the most likely values from a number of observations.

This distribution is widely used in hydrology as well as in other civil engineering applications such as measurement errors (survey), and deviations from specified strengths in concrete and other construction materials.

In general, the normal distribution is applicable where the observed values are the sum of the effects of a large number of independent processes each of which has only a small effect on the total (Chow, 1964).

This distribution is symmetrical about the mean and is therefore only suitable for data where the skewness coefficient *g* is equal to or close to zero. The spread about the mean is a function of the coefficient of variation (C_V). Note that for high C_V values the bottom tail may extend below zero. This is a disadvantage when the distribution is used for examining the minima of a data set or when generating synthetic data (some negative flows may be generated when the distribution is applied to untransformed data). Notwithstanding these deficiencies the normal distribution of log transformed data is still the most widely used distribution in hydrological analyses.

2.5.5 Log normal distribution

Hazen is credited with having observed in 1914 that while hydrological data are usually strongly skewed, the logarithms of the data have a near-symmetrical distribution (see Fig 2.1).

The log normal distribution is a normal distribution using the logarithms of the data. Whereas the normal distribution describes the sum of a large number of independent variables, the log normal distribution best describes the product of a large number of independent variables (Chow,1964).

2.5.6 Exponential distribution

This is the simplest of the one-tailed distributions and is based on the equation $y = e^{-x}$ which is equal to 1,0 when $x = 0$ and decays rapidly to 0,006 when $x = 5$.

It is seldom used directly in hydrological analyses but like the gamma distribution below it is incorporated in the more complex equations derived from it. It is the most common model for a partial duration series.

2.5.7 Gamma distribution

This is a strongly skewed distribution with a lower bound at zero, and makes use of the factorial series $1/n!$ where n need not be an integer.

The gamma distribution is the distribution of the sum of a number of independent exponentially distributed random variables.

2.5.8 Pearson Type 3 distribution

This is essentially a Gamma distribution but with the mean displaced by a constant x_0 from the origin. It includes the normal distribution as a special case when the skew is zero.

2.5.9 Log Pearson Type 3 distribution

As with the log normal distribution, this is the distribution of the logarithms of the values and is the form in which the Pearson Type 3 distribution is most commonly used in hydrological analyses. It will fit most sets of hydrological data.

2.5.10 Extreme value (EV) distributions

In many hydrological analyses, for example flood peak-frequency analyses, one is only concerned with the largest of a number of events that occurred during each year of record. If the distribution of the events within the year is such that the tail of the distribution decays exponentially then the family of extreme value distributions can be applied to the annual maxima. The most commonly used extreme value distributions are described below.

2.5.11 EV1 (Gumbel) distribution

This distribution has a constant positive skewness and is commonly used for hydrological analyses. The maxima from any distribution which converge on an exponential function at the positive tail (normal, Chi-square, log normal, etc.) will have an EV1 distribution if the basic assumptions are satisfied.

The EV1 distribution has a constant positive skewness coefficient of 1,1396 and should only be used when the data set has a value of g close to this figure. The log EV1 distribution was used extensively in the Witwatersrand University Hydrological Research Unit (HRU) publications.

2.5.12 EV2 (Frechet) distribution

This is a positively skewed distribution. If the raw data are EV2 distributed then their logarithms will be EV1 distributed.

2.5.13 EV3 (Weibull) distribution

The Weibull distribution is negatively skewed .

2.5.14 General extreme value (GEV) distribution

This is the generalized form of the above three extreme value distributions and is described in detail in the *Flood Studies Report*.

It is a family of three sub-types of distribution which are classified according to the value of the skewness coefficient g. The EV2 distribution has a value of g greater than 1,1396 and the EV3 distribution has a value of g less than 1,1396. This is a very flexible distribution and is the distribution recommended for use in the United Kingdom (*Flood Studies Report*).

2.5.15 Two component extreme value (TCEV) distribution

The two component extreme value (TCEV) distribution has four parameters and is intended to be applied when the distribution of annual maxima can be approximated by two EV1 distributions. An example of the application of the TCEV distribution is given by Pegram and Adamson (1988). It is not yet in general use in South Africa or elsewhere so no further details are provided in this handbook.

2.5.16 Wakeby distribution

The Wakeby distribution is a recently introduced distribution which was developed by Thomas in 1978 for flood frequency analyses. The distribution has five parameters which makes it very flexible.

2.5.17 Comparison of the shapes of the distributions

The two options available when fitting distributions to the data are to use the raw data (eg Fig 2.1(b)) or the log transformed data (eg Fig 2.1(e)), or some other transform. It is obviously much easier to fit a distribution to the log transformed data in Fig 2.1(e) than the untransformed data in Fig 2.1(b).

The only candidate distributions in general use in hydrology are the log normal, log Pearson Type 3, and log EV1 all using logarithmic transforms; and the extreme value distributions which are which are usually applied to the untransformed data. The two component extreme value and Wakeby distributions have been proposed as serious candidates for regional analyses but are not yet in general use.

The mathematical formulations of these distributions are given in the appendices to this chapter. While these may appear to be widely diverse, in the range of values of interest in hydrology there is often little to choose between them on the basis of fitting the distribution to the historical data. However, there is a very important qualification. The interest in hydrology is in the tails of the distributions and consequently the extrapolation beyond the range of the data. This is where their relative performance must be judged. The goodness of fit to the historical data is not in itself an adequate criterion for determining which is the "best" distribution for a specific application.

2.6 STATISTICAL ANALYSIS METHODS

2.6.1 Introduction

Given a data set comprising measured annual maximum peak flows, the statistical properties of the data set can be calculated; an appropriate probability distribution function assumed; and the probability that a peak flow of selected magnitude will be equalled or exceeded in any one year (the annual exceedance probability) determined. Alternatively the exceedance probability can be selected and the corresponding magnitude determined.

In either case, the end result will be the choice of a design flood peak of a certain magnitude which has the *specified probability of being exceeded.*

A data set comprising the annual maximum flood volumes can be analyzed in the same way, and the relationship between the annual flood peak maxima and flood volume maxima can also be determined.

2.6.2 Logarithmic transforms

Where log transformed data are used in the analyses either natural logarithms (base e) or common logarithms (base 10) can be used. The results will be the same. In this handbook the South African hydrological practice of using $\log_{10}$ rather than $\log_e$ for transforming data is followed. The advantage is the direct relationship with the decimal system which also facilitates the construction and interpretation of graphical presentations. The USA and Australian guidelines also use $\log_{10}$ transforms (Interagency Committee on Water Data, 1982, and Pilgrim, 1987).

However, the logarithms in the distribution equations are natural logarithms. In this handbook natural logarithms are identified as *ln*(..) and base 10 logarithms as *log*(..).

2.6.3 Flood peak-volume-frequency relationships

Hiemstra (1972, 1973 and 1974), Hiemstra, Zucchini and Pegram (1976), and Hiemstra and Francis (1979) pioneered the development of runhydrograph theory for the generation of families of hydrographs for a given return period which would enable designers to determine the hydrograph which would create the most severe conditions at the dam being investigated. An advantage of this method over the unit hydrograph method is that it is based on direct analysis of the flow record at the site. However, this method requires further development and should not be used without first consulting the authors.

2.7 CALCULATION METHOD

2.7.1 Categories of problems

Problems that require the use of direct statistical analysis methods can be divided into four broad categories.

(a) Determine the magnitude of an event given the design risk.

(b) Determine the probability of an event of given magnitude being equalled or exceeded.

(c) Present the magnitude-risk relationship graphically on paper or on a computer screen.

(d) Generate one or more sequences of synthetic data.

A detailed explanation of these methods as they are applied to each type of distribution is given in the appendices below, together with a summary of the underlying theory applicable to the particular distribution.

2.7.2 Format of the appendices

Each appendix is a stand-alone presentation without reference to other appendices or to the rest of the chapter. Equation numbers refer to the particular appendix only, in order to facilitate the development of the subject of the appendix with the minimum of confusion.

2.7.3 Diagrammatic illustrations

Figs 2.2 and 2.3 are graphical representations of the various equations that are used in the appendices and should help in understanding the relationships between the equations.

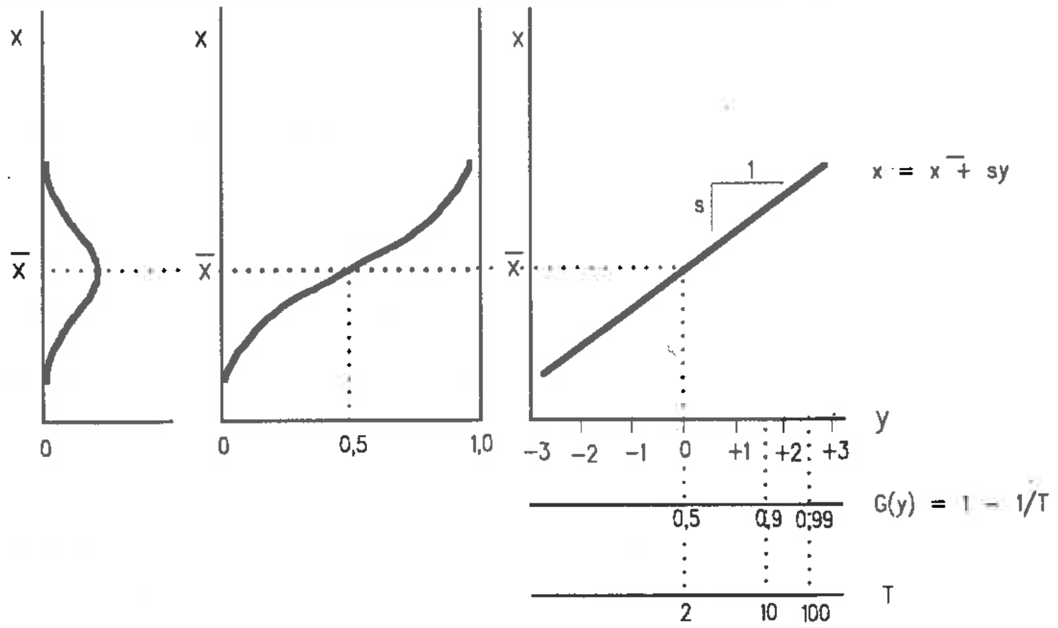

FIGURE 2.2 Relationship between variables and functions, using the normal distribution as an example.

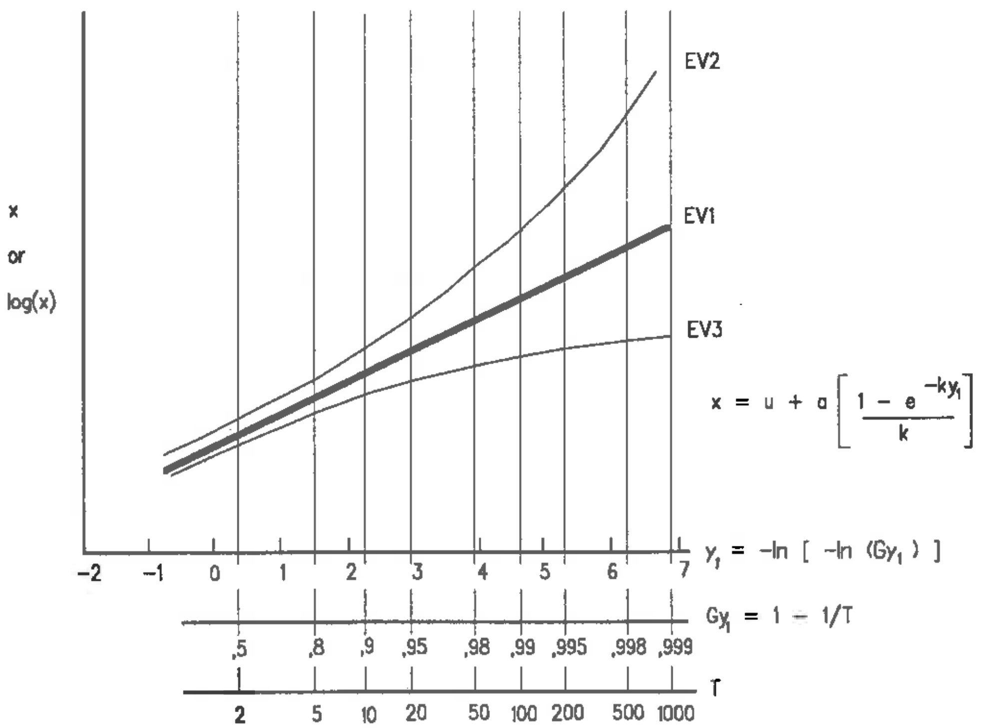

FIGURE 2.3 Relationship between variables and the cumulative probability functions of the three GEV distributions.

2.8 APPENDIX 2A : LOG NORMAL DISTRIBUTION

USING CONVENTIONAL MOMENT ESTIMATORS (LN/MM)

In this distribution the logarithms of the data are assumed to be normally distributed. The first step in the procedure is to convert the observed values to their logarithms, then carry out all the statistical analyses on the log transformed data. Thereafter the procedure is the same as that for a normal distribution which is described below, except that wherever x or y appear, substitute *log x* and *log y*.

The standardized normal distribution has a cdf :

$$G(y) = \int_{-\infty}^{y} \frac{1}{\sqrt{2\pi}} . e^{-1/2[y^2]} .dy \qquad (1)$$

where y is the standardized variable and is related to x by

$$y = (x - \mu) / \sigma \qquad (2)$$

The pdf is :

$$f(x) = \frac{1}{\sigma \sqrt{2\pi}} . e^{-1/2[(x - \mu)/\sigma]^2} \qquad (3)$$

and the cdf is :

$$F(x) = \int_{-\infty}^{x} \frac{1}{\sigma \sqrt{2\pi}} . e^{-1/2[(x - \mu)/\sigma]^2} .dx \qquad (4)$$

where the parameters μ (the mean) and σ (the standard deviation) are the location and scale parameters respectively.

From equation (2) :

$$x = \mu + \sigma y \qquad (5)$$

Analytical procedure for estimating the magnitude of an event x for a given return period T

The value of y for a given value of $G(y)$ cannot be solved directly from equation (1), and published tables have to be used. However, the following numerical approximation from Abramowitz and Stegun (1965, eqn 26.2.23) should be more than adequate for computer applications.

Note that :

$$G(y) = P(X <= x) = 1 - 1/T \tag{6}$$

Use the Abramowitz and Stegun approximation to determine y from $G(y)$.

$$y = -(v - (a + bv + cv^2) / (1 + dv + ev^2 + fv^3)) \tag{7}$$

$$\text{where } v = [\ \ln(1/p^2)\ ]^{1/2} \tag{8}$$

$$\text{and} \quad p = G(y)$$

The values of the parameters are:-

a = 2,515 517	b = 0,802 853	c = 0,010 328
d = 1,432 788	e = 0,189 269	f = 0,001 308

The magnitude of the event x can now be determined from equation (5) for the known values of the mean, standard deviation and y. The absolute value of the error when using equations (7) and (8) is less than $4{,}5.10^{-3}$.

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Analytical procedure for estimating the return period T of an event of given magnitude x

Given the values of the mean and standard deviation, determine the value of y measured in units of standard deviations from equation (2).

The value of $G(y)$ for a given y in equation (1) can be derived from tables. For computer applications there are several approximations that can be used. The following approximation is derived from Abramowitz and Stegun (eqn 26.2.18).

$$G(y) = 1 - 0{,}5(1 + ay + by^2 + cy^3 + dy^4)^{-4} \tag{9}$$

$$\begin{aligned} \text{where } a &= 0{,}196\ 854 \\ b &= 0{,}115\ 195 \\ c &= 0{,}000\ 344 \\ d &= 0{,}019\ 527 \end{aligned}$$

The absolute value of error when using this approximation is less than $2{,}5.10^{-4}$.

Equation (9) is valid for values of y in the range zero to infinity with corresponding values of $P(y)$ from 0,5 to 1,0. The following adjustment has to be made to cover the full range of y from minus infinity to plus infinity :

$$\text{for } y \geq 0 : G(y) = P(y) \tag{10}$$

$$\text{for } y < 0 : G(y) = 1-P(y) \tag{11}$$

When applying equation (11) the sign of y must be changed. Once $G(y)$ has been determined, the return period T can be calculated from the inverse of equation (6) :

$$T = 1 / [1 - G(y)] \tag{12}$$

The absolute value of the error when using equation (9) is less than $2{,}5.10^{-4}$

Graphical method for computer application

Construct a graph with the vertical scale sufficient to cover the spread of the data values (or logarithms of the data values) plus an allowance for extrapolation.

The horizontal scale should be linear and cover the range of at least -3,0 to +3,0. This is the value of the variable y which has the dimension of standard deviations.

The empirical relationship between y and the non-exceedance probability $G(y)$ is given by equations (7) and (8). However, as fixed non-exceedance probabilities are usually shown on the graph these can be included in the computer program as data together with the corresponding return periods if required.

Horizontal lines are drawn to suit the data intervals, and vertical lines to correspond with the return period or non exceedance probability. To plot the data points on the graph proceed as follows :

1. Rank the data from largest to smallest.
2. Determine the associated assumed non-exceedance probability for each value by using the Weibull plotting position. (See Chapter 5 for alternative plotting positions):

   $$F_m = 1 - m/(n+1) \tag{13}$$

   where m is the ranked position and n is the total number of observations in the data set.
3. Determine the corresponding y value from the empirical equations (7) and (8).

4. Plot the observed value x_m against this value y. Note that x_m is in the vertical direction and y in the horizontal direction.

5. Repeat the process for the other values in the data set, until all the points have been plotted.

6. Note from equation (5) that the relationship between x and y is linear. Derive the values of x for $y = -3$, $y = 0$ and $y = +3$. Plot and draw a line through these points and extrapolate to the edges of the graph.

7. Compare the plotted points with this line and note the goodness of fit visually, as well as the presence of outliers, if any.

Model for generating normally (or log normally) distributed data

Values of x cannot be determined directly from $F(x)$ in equation (4). There are a number of empirical models that can be used, however. There is little to choose from them in terms of accuracy, the main objectives for their development being to reduce computer time to a minimum. This is not a consideration in hydrological applications, where the following simple algorithm from Abramowitz and Stegun (1965) should be adequate. More accurate algorithms are also given in that publication, as well as in Rubenstein (1981).

Proceed as follows:-

1. Either select the required values of the mean μ and standard deviation σ, or determine them from the observed data set.
2. The generation model (Abramowitz and Stegun p.953) is :

$$R_x = \left[\sum_{i=1}^{n} U_i - \frac{n}{2} \right] \left[\frac{n}{12} \right]^{-1/2} \tag{14}$$

where U_i is a uniformly distributed random number between zero and one.

When $n = 12$ this reduces to :

$$R_x = \sum_{i=1}^{12} U_i - 6 \tag{15}$$

The maximum errors, assuming no errors in the algorithm generating the uniformly distributed random numbers, are 9.10^{-3} for $-2 < R_x < 2$ and 9.10^{-1} for R_x in the range $2 < |R_x| < 3$.

An alternative model (Knuth, 1969) is an adapted form of the polar method. It is stated that the method is rather slow but accurate and takes a minimum of storage space. Pairs of variates are generated at a time so N must be an even number.

The algorithm is :

$$s = (2\,U_1 - 1)^2 + (2U_2 - 1)^2 \tag{16}$$

IF $s > 1$ reject, otherwise :

$$x_1 = (2U_1 - 1)\,\{-\,(2/s)\,\ln(s)\,\}^{1/2} \tag{17}$$

and

$$x_2 = (2U_2 - 1)\,\{-(2/s)\,\ln(s)\,\}^{1/2} \tag{18}$$

where U_1 and U_2 are $U(0,1)$ uniformly distributed random numbers.

If the simulation is of an exploratory nature use equation (15) as the model, but if greater precision is required use the model in equations (16) to (18).

3. Dimensionalize the random number by multiplying it by the standard deviation and adding the mean.

4. Determine the values of the mean, standard deviation and skewness coefficient of the data set and compare these with the required values (the skewness coefficient should be close to zero). Differences must be expected, but they will decrease with increase in N. Major discrepancies would indicate an error in the program.

5. Plot the results using the graphical method as a check.

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2.9 APPENDIX 2B : LOG PEARSON TYPE 3 DISTRIBUTION USING CONVENTIONAL MOMENT ESTIMATORS (LP3/MM)

In this appendix it is assumed that the logarithms of the data follow a Pearson Type 3 distribution. The first step in the procedure is to convert each value in the raw data to its logarithm, then carry out the analyses on the log transformed data. The values of x and y in the expressions below are therefore the logarithms of the data values.

The cdf of the standardized gamma distribution is :

$$G(y) = \int_0^y [\,(y^{\gamma-1} \,.\, e^{-y}) \,/\, \Gamma(\gamma)\,].dy \qquad (1)$$

where y is the standardized variate of the Pearson Type 3 distribution and is related to x by :

$$y = (x - x_0)/\beta \qquad (2)$$

The pdf of this distribution is :

$$f(x) = [\,(x - x_0)^{\gamma-1} \;.e^{-(x-x_0)/\beta}\,] \,/\beta^{\gamma}.\Gamma(\gamma) \qquad (3)$$

and the cdf is :

$$F(x) = \int_{x_0}^{x} \left[(x - x_0)^{\gamma-1}. \quad e^{-(x-x_0)/\beta} \right] / \,\beta^{\gamma}\,\Gamma(\gamma) \;\; .dx \qquad (4)$$

This is a gamma distribution with parameters β and γ, and the origin at $x = x_o$.

The mean, variance and skewness are :

$$\overline{x} = x_0 + \beta\gamma \qquad (5)$$

$$s^2 = \beta^2\gamma \qquad (6)$$

$$g = 2/\sqrt{\gamma} \qquad (7)$$

Analytical procedure for estimating the magnitude of an event x (=log$x$) for a given return period T

Note that :

$$G(y) = 1 - 1/T \tag{8}$$

The value of y for a known value of $G(y)$ cannot be derived directly from equation (1) and has to be interpolated from tables of y vs $G(y)$ for a series of values of $G(y)$.

Proceed as follows:-

Use equations (5), (6) and (7) to calculate β, γ and x_o from the mean, variance and skewness coefficient of the data set :

$$\gamma = (2/g)^2 \tag{9}$$

$$\beta = (s^2/\gamma)^{1/2} \quad = s(g/2) \tag{10}$$

$$x_0 = \quad \overline{x} - \beta\gamma \quad = \overline{x} - 2s/g \tag{11}$$

From tables, where G(y) and γ are known, interpolate the value of y_T.

From equation (2) :

$$x_T = x_0 + \beta y_T \tag{12}$$

An alternative procedure is to use the mean, standard deviation and skewness coefficient directly instead of via the parameters β, γ and x_0 . The equation is :

$$x_T = \quad \overline{x} + sK_T \tag{13}$$

where K_T is the frequency factor for the return period T in a distribution having the same form as that of x but which has zero mean and unit variance. In the case of the normal distribution K_T is the same as the standardized variate y_T but this is not so in other distributions.

For the Pearson Type 3 distribution replace x_0 and β in equation (12) with x, s and g derived from equations (9), (10) and (11). Then :

$$x_T = \quad \overline{x} - s\{ 2/g - (g/2)y_T\} \tag{14}$$

which is equivalent to equation 13 where :

$$K_T = (g/2)y_T - 2/g \tag{15}$$

K_T is in units of standard deviations from the mean for the return period T. Harter published tables for determining the value of K_T for different values of the return period T and the skewness coefficient g, and these were extended and published by the US guidelines (Interagency Advisory Committee on Water Data, 1982), which also gives the following approximate transformation for skew coefficients g between 1,0 and -1,0 :

$$K_T = \frac{2}{g} \left\{ \left[\left(G(y) - \frac{g}{6} \right) \frac{g}{6} + 1 \right]^3 - 1 \right\} \tag{16}$$

Adamson (1978) gives the following approximation for the full Harter tables :

$$\begin{aligned} K_T = t \quad & - (t^2-1)(g/6) + (1/3)(t^3-6t)(g/6)^2 - (t^2-1)(g/6)^3 \\ & + t(g/6)^4 + (1/3)(g/6)^5 \end{aligned} \tag{17}$$

where t is the standardized normal variate, y_1. This equation is unreliable at the lower tail of the distribution and should not be used.

Having determined the value of K_T, the required value of x can be derived from equation (13).

Analytical procedure for estimating the return period T of an event of given magnitude x

Using the alternative procedure described above, convert x to a standardized variate y by subtracting the mean and dividing by the standard deviation. The value of y is now in units of standard deviations. The skewness coefficient of the data set is also known, and the non-exceedance probability $G(y)$ can be calculated from the inverse of equation (16) :

$$G(y) = \frac{6}{g} \left\{ \left[(g/2)k + 1 \right]^{1/3} + \frac{g^2}{36} - 1 \right\} \tag{18}$$

for values of g between -1 and +1. If g is outside this range and the calculation is critical then the published Harter tables should be used.

The required return period T can be determined from the inverse of equation (8) :

$$T = 1 \,/\, [\ 1-G(y)\ ] \tag{19}$$

Graphical method for computer application.

Construct a normal probability graph as described in Appendix 2A. Plot the data points as described there. Replace step 6 with the following:

6. The relationship between x and y as shown in equation (2) is non-linear. However, equation (13) can be used in the same way as for the normal distribution substituting K_T which is a function of y and g, instead of y alone. As a fixed set of y values in the range -3 to -3 is required, select values of y within this range in increments of 0,5 use equation (15) to determine the corresponding values of K_T, and then substitute these in equation (13) to obtain the value of x. Plot and draw a curve through these points.

7. Compare the plotted data with this line and note the goodness of fit visually, as well as the presence of outliers, if any.

Model for generating log Pearson Type 3 distributed variables

The Pearson Type 3 distribution is a two parameter gamma distribution with shifted origins. The mean is shifted by the value x_0 (equation(5)) while the variance, standard deviation and skewness coefficient all of which are related to central moments, are unaffected by the value of x_o (equations (6) and (7)).

The following method is based on that provided by Landwehr *et al* (1987) except that $Z=\log_{10}(X)$ is substituted for $Z=\log_e(X-v)$.

A random variable X in real space is said to be log Pearson type 3 (LP3) distributed if $Z=\log_e(X-v)$ in log space is distributed as Pearson type 3 where the constant $v<=0$. (v is assumed to be zero in the equations and algorithms below.)

The statistical properties of Z are :

$$\text{mean} = \bar{z} = c+ab \tag{20}$$

$$\text{std devn} = s_z = |a|\, b^{1/2} \tag{21}$$

$$\text{skew} = g_z = 2a[\, |a|\, b^{1/2}\,]^{-1} \tag{22}$$

$$= 2a/s \tag{23}$$

From which the values of the constants a, b and c can be derived :

$$a = 0,5gs \tag{24}$$

$$b = (s/|a|)^2 \tag{25}$$

$$c = z - ab \tag{26}$$

The equation for generating LP3 random variables is :

$$x = \text{antilog}\,\{c + a\,[-\sum_{k=1}^{b} \ln u_k\,]\,\} + v \tag{27}$$

where u is a U(0,1) random variable.

Define B as follows where $[b]$ denotes the greatest integer less than or equal to b.

1. Set $r = b - [b]$ and $s = 1 - r$

2. Generate two U(0,1) randomly distributed variables u_1 and u_2

3. Set $\zeta = u_1^{1/r}$ and $\xi = u_2^{1/s}$

4. If $\zeta + \xi > 1$ return to step 2, otherwise proceed to step 5.

5. Set $B = \zeta / (\zeta + \xi)$.

If b is an integer, $B = 0$, and therefore

$$x = \text{antilog}\,\{c + a\,[-\sum_{k=1}^{b} \ln u_k\,]\,\} + v \tag{28}$$

If $b < 1$ then $[b] = 0$, so that

$$x = \text{antilog}\,\{c + a\,[-B \ln u\,]\,\} + v \tag{29}$$

less.

(24) ## 2.10 APPENDIX 2C : EXTREME VALUE TYPE 1 DISTRIBUTION

(25) ## USING CONVENTIONAL MOMENT ESTIMATORS (EV1/MM)

(26)

This is a double exponential distribution. The general equation for the standardized cdf is :

(27)

$$G(y) = e^{-e^{-y}} \qquad (1)$$

where y is the standardized variate and is related to x by :

$$y = (x-u)/\alpha \qquad (2)$$

The pdf is :

$$f(x) = (1/\alpha) \exp\{ - (x-u) / \alpha. - \exp[-(x-u)/\alpha \} \qquad (3)$$

and the cdf is :

$$F(x) = \exp\{ -\exp[-(x-u)/\alpha] \} \qquad (4)$$

where u is a location parameter,
and α is a scale parameter.
(28) The mode is at the position $x = u$

$$\text{The mean} = u + 0.5772\alpha \qquad (5)$$

(29)

$$\text{The variance} = (\pi^2 \alpha^2)/6 \qquad (6)$$

$$\text{The skewness coefficient is a constant} = 1.1396 \qquad (7)$$

The values of x and y can be derived from equation (4) by simple algebraic manipulation.

$$x = - \alpha \ln[-\ln F(x)] + u \qquad (8)$$

$$y = - \alpha \ln[-\ln F(y)] + u \qquad (9)$$

Estimation of the parameters u and α given the mean x and the standard deviation s of a data set

Calculate the mean and standard deviation of the data set then determine the values of the parameters u and α from equations (5) and (6) ie

$$\alpha = [\ (6s^2)\ /\ \pi^2\ ]^{1/2} \tag{10}$$

$$\text{and } u = \quad \overline{x} - 0.5772\ \alpha \tag{11}$$

Analytical procedure for estimating the magnitude of an event for a given return period

From the inverse equation (9) :

$$y_T = -\ln[\ -\ln(1-1/T)\ ] \tag{12}$$

where T is the return period and $(1 - 1/T)$ is the associated non-exceedance probability.

From equation (2) :

$$x_T = u + \alpha y_T \tag{13}$$

Analytical procedure for estimating the return period T of an event of given magnitude x

From equation (4) calculate $F(x)$ for the known values of x, u and α. The return period T is related to $F(x)$ by :

$$T = 1/[1-F(x)] \tag{14}$$

Graphical method for computer application

Construct a graph with the vertical scale sufficient to cover the spread of the data values plus an allowance for extrapolation. It can be either linear if raw data are being used, or a logarithmic scale if log transformed data are being analyzed. In the latter case all the parameters and properties of the distribution are those of the logarithms of the data values. The horizontal scale should be linear and cover the range from -2,0 to +7,0 which is the value of the variable y measured in units of standard deviations.

The relationship between y and the exceedance probability $G(y)$ is given by :

$$y = -\ln[\ -\ln G(y)\ ] \tag{15}$$

Similarly the relationship between y and the return period T is given by equation (12). Mark off the horizontal scale in steps of the return period T (equation 12) or the exceedance probability y (equation 15), or both. Note that the x values are plotted in the vertical direction and the y values in the horizontal direction.

Proceed as follows :

1. Rank the data from largest to smallest

2. Determine the associated assumed non exceedance probability for each value by using the Gringorten formula :

$$F_m = 1 - (m-0.44)/(n+0.12) \tag{16}$$

where m is the ranked position and n is the total number of observations.

3. Determine the corresponding y_m value from equation (8)

$$y_m = -\ln[\,-\ln F_m\,] \tag{17}$$

4. Plot the observed value x_m against this value y_m. Note that x_m is in the vertical direction.

5. Repeat the process for the other values in the data set, until all the points have been plotted.

6. Note from equations (2) and (6) that when x = mean, y = 0,5772 therefore plot the mean at this value of y.

7. Note also from equation (2) that

$$x = u + \alpha y \tag{18}$$

thus for $y = -2$, $y = u - 2\alpha$ and for $y = +3$, $x = u + 3\alpha$

Plot both of these points and draw a straight line through them and the mean.

8. Compare the plotted points with this line and note the goodness of fit visually, as well as the presence of outliers, if any,.

Model for generating EV1 distributed data

The inverse equation (8) is an obvious candidate for a generation model. Proceed as follows :-

1. Either select the required values of α and u, or determine the values from the values of the mean and standard deviation of the observed data using equations (10) and (11).

2 The generation model derived from equation (8) is :

$$Rx = -\ln(-\ln U)\alpha + u \tag{19}$$

where Rx is the desired random variable and U is a uniformly distributed random number between zero and one.

3. Determine the values of the mean, standard deviation and skewness coefficient from the generated data set and compare these with the required values. Differences must be expected, but they will decrease with increase in N. Major discrepancies would indicate an error in the program.

4. Plot the results using the graphical method as a final check.

2.11 APPENDIX 2D : GENERAL EXTREME VALUE DISTRIBUTION USING CONVENTIONAL MOMENT ESTIMATORS (GEV/MM)

This is a family of distributions which are distinguished by the sign of the shape parameter k :-

Distribution	Value of k
Extreme Value Type 1 (Gumbel)	$k = 0$
Extreme Value Type 2 (Frechet)	$k < 0$
Extreme Value Type 3 (Weibull)	$k > 0$

The following development relates to all three distributions, but if the value of k is close to zero (g close to 1,14), then the more tractable EV1 distribution should be applied as explained in Appendix 2C.

The standardized GEV distribution has a cdf of the form :

$$G(y) = e^{-y^{1/k}} \tag{1}$$

where the standardized variate y is related to x by :

$$y = [1 - k(x-u)/\alpha] \tag{2}$$

The pdf is :

$$f(x) = (1/\alpha)\,[1-k(x-u)/\alpha\,]^{1/k-1} \quad .e^{-[1-k(x-u/\alpha]^{1/k}} \tag{3}$$

and the cdf is :

$$F(x) = \exp\{-[1-k(x-u)/\alpha]^{1/k} \tag{4}$$

The distribution has three parameters :
u which is a location parameter,
α which is a positive scale parameter, and
k which is a negative shape parameter in the case of the EV2 distribution, and positive for the EV3 distribution.

It is the sign of the shape parameter k which distinguishes these distributions from one another. The EV2 distribution has a lower bound at $u + \alpha/k$ which is less than u as α/k is negative. For the EV3 distribution k is positive and the variable values have an upper bound at $u + \alpha/k$ which is greater than u as both α and k are positive. The value of x can be derived from equation (4) and y from equation (1) by simple algebraic manipulation.

$$x = [\ 1-(\ -\ln F(x)\ )^k\ ]\alpha/k + u \tag{5}$$

$$y = \exp\{\ k\ \ln[\ -\ln\ G(y)\ ]\ \} \tag{6}$$

In the case of EV2 :

$$x = u + \alpha/k - (\alpha/k)y \tag{7}$$

(the negative sign becomes positive for EV3).

The distribution of the standardized variate y depends on k, and from equation (6):

$$y = \exp\ \{\ k\ \ln[-\ln\ G(y)\ ]\ \} \tag{8}$$

also :

$$y = \exp\ \{\ k\ \ln[\ -\ln(1-1/T)\ ]\ \} \tag{9}$$

where T is the return period.

Estimation of the parameters α and k given the standard deviation s and skewness coefficient g

The mean of the standardized variates y is given by :

$$\begin{aligned} E(y) &= +\ \Gamma(1+k) && \text{for EV2} \\ \text{and } E(y) &= -\ \Gamma(1+k) && \text{for EV3} \end{aligned} \tag{10}$$

where $\Gamma\ (1+k)$ is a gamma function.

The variance is given by :

$$var(y) = \Gamma\ (1+2k) - \Gamma^2\ (1+k) \tag{11}$$

The relationship between the coefficient of variation c and k is given by :

$$c^2 = \left(\frac{s}{\mu} \right)^2 = \frac{\Gamma(1+2k)}{\Gamma^2\ (1+k)} - 1 \tag{12}$$

As the values of s and u are known, equation (12) can be solved to obtain k. However, this equation is intractable and the value of k for a known value of c cannot be determined directly. The following method is used in the *Flood Studies Report.*

It can be shown that if y_2 is a standardized EV2 variate with parameter k, then the following relationship holds :

$$y_2 = e^{-ky1} \tag{13}$$

where y_1 is an EVl standardized variate. Consequently, in equation (7), y may be replaced by $exp\ (-ky_1)$ to give :

$$x_2 = u + \frac{\alpha}{k} - \frac{\alpha}{k}.e^{-ky1}$$

$$= u + \alpha[\ (1-e^{-ky1})\ /\ k\ ] \tag{14}$$

where x_2 is an EV2 variable and y_1 is a standardized EVl variate.

$$W(y_1;k) = (1-e^{-ky1})\ /\ k \tag{15}$$

$$y_1 = -\ln[\ -\ln(1-1/T)\ ] \tag{16}$$

$$x_3 = u + \alpha\ W(y_1;k) \tag{17}$$

The moments of the standardized variate y_2 are :

$$mean\ =\ U_1\ = \Gamma\ (1+k) \tag{18}$$

$$var(y)_2\ =\ U_2\ =\ \Gamma\ (1+2k)\ -\ \Gamma^2\ (1+k) \tag{19}$$

$$\text{and} \qquad U_3\ =\ \Gamma(1+3k)\ -\ 3\Gamma(1+2k).\Gamma(1+k)\ +\ 2\Gamma^3\ (1+k) \tag{20}$$

$$skewness\ =\ g\ =\ U_3/U_2^{3/2} \tag{21}$$

The mean, variance, standard deviation, coefficient of variation and skewness coefficient can be calculated from these equations for a range of values of k, and the results presented in a table. Having calculated the value of the skewness coefficient from the data set, the corresponding values of k and $var(y)_2$ can be interpolated from the table.

Returning to equation (7), this has the general form :

$$x = A + By \tag{22}$$

$$\text{where } A = u + \alpha/k \tag{23}$$

and $B = -\alpha/k$ (24)

The value of B in equation (24) can be obtained from :

$$B = (s^2/var(y)_2)^{1/2} \tag{25}$$

and from equation (24) :

$$\alpha = -Bk \tag{26}$$

the value of α can be determined.

For computer applications the problem is more difficult because it requires the inverse solutions of equations (19), (20) and (21) involving the gamma function $\Gamma\ (x+1) = x!$. The simplest approach is to use a polynomial approximation of the gamma function, then calculate the values of U_2, U_3 and g in these equations for a range of values of k from -0,25 to +0,25. Another polynomial can then be fitted to the set of values of g corresponding to the selected values of k, but using k as the dependent variable.

The two polynomials are:-

(a) Gamma function :

$$\Gamma\ (1+x) = 1 + ax + bx^2 + cx^3 + dx^4 + ex^5 \tag{27}$$

where
$a = -\ 0.574\ 865$
$b = +\ 0.951\ 236$
$c = -\ 0.699\ 859$
$d = +\ 0.424\ 555$
$e = -\ 0.101\ 068$

for values of x between zero and one. (from Hastings, 1955 in Abramowitz and Stegun, 1965 p257).

For values of x outside this range use the relationships :

$$\Gamma\ (1+x) = x\Gamma(x) \tag{28}$$

for values of x greater than one, and

$$\Gamma(x) = \frac{\Gamma(1+x)}{x} \tag{29}$$

for values of x less than zero. For example :

$$\Gamma(4,5) = 3,5\ .\ 2,5\ .\ \Gamma(1,5) \tag{30}$$

(b) k-g relationships

A polynomial can be fitted to the tabulated values of k and g in Tables 1.7 and 1.10 in the *Flood Studies Report*, including the value of k = *1,1396* for g = *1,00*. The resulting equation using standard polynomial regression methods is :

$$k = a + bg + cg^2 + dg^3 + eg^4 \tag{31}$$

where	a = 0,277 15	b = -0,327 73
	c = 0,090 27	d = -0,012 60
	e = 0,000 70	

The value of r^2 is 0,999, and the standard error of the estimate is 0,004.

The calculation sequence for the analytical determination of the GEV parameters u, a and k from the mean μ, standard deviation s, and skewness coefficient g is as follows :

(a) Determine the value of k from the known value of the skewness coefficient g, using equation (31).

(b) Determine the value of *var(y)* for the value of k in equation (19) using the polynomial approximation of the gamma function in equation (27).

(c) Determine the value of α from equations (25) and (26).

Analytical procedure for estimating the magnitude of an event for a given return period.

Calculate the mean and standard deviation from the data set.

Determine the values of the parameters α and k as described above.

Determine the value of y_T for the required return period T from equation (9). Then from equation (7) :

$$x_T = u + \alpha/k - (\alpha/k)\, y_T \tag{32}$$

(the negative sign is positive for EV3)

Analytical procedure for estimating the return period of an event of given magnitude

Calculate the mean, standard deviation and coefficient of variation from the data set, then determine the values of the parameters α and k as described above.

From equation (4) for the given value of x calculate the non exceedance probability F(x).

The return period T is related to $F(x)$ by :

$$T = 1/[1-F(x)] \tag{33}$$

Graphical method for computer application
Use the same method as that given for the EV1 distribution for constructing the graph. Then proceed as follows:-

1. Rank the data from largest to smallest.

2. Determine the associated assumed non-exceedance probability for each value by using the Gringorten plotting position :

$$F_m = 1 - (m-0,44)/(n+0,12) \tag{34}$$

   where m is the ranked position and n is the total number of observations.

3. Determine the corresponding y_m value from equation (6) :

$$y_m = \exp\{ k \ln[-\ln F_m] \} \tag{35}$$

4. Plot the observed value x_m against the value y_m.

5. Repeat the process for the other values in the data set, until all the points have been plotted.

6. Use equation (7) to determine the value of x for a series of values of y in the range -2 to +7 and draw a curve through these points.

7. Compare the plotted points with this curve and note the goodness of fit visually, as well as the presence of outliers, if any.

Model for generating GEV distributed data

Several methods are available. The logical choice is the inverse transform method making use of the inverse equation (5). Proceed as follows :

1. Either select the required values of α and u, and k or determine them from the values of the parameters of the data set as detailed in the section on parameter estimation above.

2. The generation model derived from equation (5) is :

$$Rx = [\, 1 - (-\ln U)^k \,]\alpha/k + u \tag{36}$$

   where R_x is the desired random variable and U is a uniformly distributed random number between zero and one.

3. Determine the values of the mean, standard deviation, and skewness coefficient from the generated data set and compare these with the required values. Differences must be expected but they will decrease with increase in N. Major discrepancies would indicate an error in the program.

4. Plot the results using the graphical method as a final check.

2.12 APPENDIX 2E : GENERAL EXTREME VALUE DISTRIBUTION USING PROBABILITY WEIGHTED MOMENT ESTIMATORS (GEV/PWM)

All the distributions described so far use conventional moments (mean, standard deviation and skewness coefficient) for determining the values of the parameters of the distribution functions. These moments are functions of the mean, and the dispersion about the mean value raised to the power 2 and 3 respectively and therefore the higher the moment the greater the sensitivity to both high and low values in the data set being analyzed, which are given disproportionately large weights. This is not necessarily a disadvantage and will be discussed in more detail in later chapters.

The maximum likelihood method of parameter estimation was proposed in the *Flood Studies Report* for the GEV distribution. It is an iterative method which is not easy to apply and is not always successful.

Greenwood *et al* (1979) introduced probability weighted moment (PWM) procedures for parameter estimation for several distributions, and Hosking *et al* (1985) presented a PWM procedure for the GEV distribution which is described below. The claimed advantage of the PWM procedures is that as all the moments are linearly related to the data values, they give all values the same weight.

The probability weighted moments proposed by the above authors make use of the following biased plotting position estimator which is referred to as the Greenwood plotting position in this handbook.

$$p_j = (j-0{,}35)/n \qquad (1)$$

This is substituted in the equation for the moments estimator :

$$\beta r = n^{-1} \sum_{j=1}^{n} p^r{}_j \, x_j \qquad r = 0,1,2 \qquad (2)$$

where j is the rank position ($j=1$ for the smallest value and $j=n$ for the largest value).

The moments are scaled by dividing them by the sample mean β_0.

The GEV distribution parameters u, α, and k are derived from the three moments β_0, β_1, and β_2 via the coefficient c as follows :

$$c = [(2\beta_1-\beta_0)/(3\beta_2-\beta_0)] - [\log 2/\log 3] \qquad (3)$$

$$k = 7,8590c + 2,9554c^2$$

$$\alpha = [(2\beta_1-\beta_0)^k] \,/\, [(1-2^{-k})\Gamma(1+k)] \tag{5}$$

$$u = \beta_0 + a[\Gamma(1+k)-1]/k \tag{6}$$

Thereafter the calculations are the same as those for the GEV/MM distribution described in Appendix 2E.

2.13 APPENDIX 2F : WAKEBY DISTRIBUTION

USING PROBABILITY WEIGHTED MOMENT ESTIMATORS (WAK/PWM)

The normal, log normal, and extreme value type 1 distributions all have two parameters and fixed skewness coefficients, and therefore plot as straight lines on the appropriate graphs. The log Pearson type 3 and general extreme value distributions have three parameters and can therefore accommodate variable skewness. They plot as curves on the normal and EV1 graphs with the direction of the curvature dependent on the sign of the skewness coefficients (g for the LP3 and $-k$ for the GEV).

Arising from the need for a more flexible distribution for flood frequency analyses, Houghton (1978) proposed the use of the 5-parameter Wakeby distribution, and a year later Greenwood *et al* (1979) developed a probability weighted moments procedure for estimating the parameter values for this distribution, while Landwehr *et al* (1979) described the algorithms for deriving these moments. The procedures below are based on the information in these three papers.

The Wakeby distribution is described in its inverse form :

$$x = m + a[1-(1-F)^{b}] - c[1-(1-F)^{-d}] \qquad (1)$$

where $F \equiv F(x) = P(X<=x)$
m is the location parameter
and a, b, c and d are the remaining distribution parameters

1. Algorithm for estimating probability weighted moments

The distribution function F(x) may be characterized by the probability weighted moments :

$$M_{l,j,k} \equiv E[X^{l}\, F^{j}\, (1-F)^{k}] \qquad (2)$$

The derivation in which l=1, j=0, and $k \geq 0$ forms the basis for the algorithm for estimating the parameter values of the Wakeby distribution.

2. Estimation of distribution moments

The authors determined that the empirical relationship

$$F_{i} = (\, i - 0{,}35 \,)\, /\, n \qquad (3)$$

provided a better performance of the moments estimator particularly in the upper quantile values of x ($F \geq 0{,}50$) than any of the more common plotting positions. Consequently, in the context of flood frequency analysis, the estimate of $M_{(k)}$ for $k = 0, 1, 2, 3$ was defined to be

$$M(k) = 1 / n \sum_{i=1}^{n} x_i [(n - i + 0{,}35) / n]^k \tag{4}$$

which is the same as

$$M(k) = 1 / n \sum_{i=1}^{n} x_i [1 - (i + 0{,}35) / n]^k \tag{5}$$

For ease of computation and comparison equation (5) can be rewritten

$$M(k) = 1 / n \sum_{i=1}^{n} x_i (p_i)^k \tag{6}$$

where $p_i = [1 - (i - 0.35) / n]$ (7)

All authors using this estimator specified that x_i is the ranked value of x from $i = 1$ for the smallest value to $i = n$ (the total number of observations) for the largest value, thus

$$0 <= p^k{}_n <= \ldots\ldots <= p^k{}_1 <= 1$$

The moments can be expressed in terms of the parameters. This is useful for checking the algorithms during the development of the model.

$$M_{1,0,k} = m/(1+k) + (a-c)/(1+k) - a/(1+k+b) + c/(1+k-d) \tag{8}$$

3. Estimation of coefficients N and C

Two sets of equations for N and C are required for each of $j = 1, 2, 3$. These are for the situations where $m = 0$ and $m \neq 0$ respectively in equation (1).

where $m = 0$:-

$$N_{4-j} = - (3)^j M_2 + (2)^{1+j} M_1 - M_0 \tag{9}$$

$$C_{4-j} = -(4)^j M_3 + 2(3)^j M_2 - (2)^j M_1 \tag{10}$$

where $m \neq 0$:-

$$N_{4-j} = (4)^j M_3 - (3)^{1+j} M_2 - 3(2)^j M_1 - M_0 \quad (11)$$

$$C_{4-j} = (5)^j M_4 - 3(4)^j M_3 + (3)^{1+j} M_2 - (2)^j M_1 \quad (12)$$

4. Estimation of parameter values *a, b, c, d* and *m*

The estimation of the value of the parameter b from the estimates of the moments M_0, M_1, M_2 and M_3 via the coefficients N_{4-j} and C_{4-j} is the entry point for the derivation of the remaining parameter values a, c, d and m.

The procedure is carried out in four stages. If success is not achieved in one stage, then the procedure goes on to the next stage. If no successful combination is obtained by the end of the fourth stage, it is assumed that no WAK/PWM distribution for the data set exists.

The values of the individual parameters as well as their combinations have to meet a number of criteria. These are detailed in Landwehr [1978] and summarized below in the form of error conditions.

Error 1 Invalid value for b. Either b is imaginary or $b > b_{max}$ or $b < b_{min}$. The recommended values of b_{max} and b_{min} are 50 and 0.3 respectively.

Error 2. The mean does not exist if $d \geq 1$.

Error 3. Invalid probability density function if $f(m) = 1/(ab + cd) < 0$.

Error 4. Improperly defined cumulative density function for this combination of parameter signs resulting in
$F(x_1) > F(x_2)$ for $x_1 < x_2$.

Error 5. Improperly defined cumulative density function for this combination of parameter values resulting in
$F(x_1) > F(x_2)$ for $x_1 < x_2$.

Condition 6. While not an error condition and not mentioned by the authors of the method, the parameters a, b, c and d should be positive while m should be close to zero for most flood data sets. A solution where this condition is not met should be treated with caution.

Errors 4 and 5 are detected by examining the combination of parameter signs and parameter values in Table 1.

TABLE 1. Valid and invalid parameter combinations for the Wakeby distribution (based on corrected Table B1 of Landwehr *et al* 1979).

Combination #	Sign of parameter *a*	*b*	*c*	*d*	Valid distribution? Yes	Conditional	No
1	+	+	+	+	Yes		
2	-	+	+	+		(i)	
3	+	+	-	+			No
4	-	+	-	+			No
5	+	+	+	-		(i),(ii)	
6	-	+	+	-			No
7	+	+	-	-	Yes		
8	-	+	-	-		(i),(iii)	
9	+	-	+	+		(i),(iv)	
10	-	-	+	+	Yes		
11	+	-	-	+			No
12	-	-	-	+		(i),(v)	
13	+	-	+	-			No
14	-	-	+	-		(i)	
15	+	-	-	-			No
16	-	-	-	-	Yes		

Conditions

(i) Valid if $ab + cd > 0$, ie, valid pdf

(ii) Valid if $a > c$ and $b <= |d|$

(iii) Valid if $a > c$ and $b >= |d|$

(iv) Valid if either $|b| < |d|$ or $c > a$ when $|b| = |d|$

(v) Valid if either $|b| > |d|$ or $c > a$ when $|b| = |d|$

The errors are tested in the order shown and if an error condition is encountered, the attempt at finding an acceptable combination of parameter values is aborted and the program proceeds to the next stage of calculations.

The equations used for estimating the parameter values are :

$$b = \frac{(N_3C_1-N_1C_3) + [(N_1C_3-N_3C_1)^2 - 4(N_1C_2-N_2C_1)(N_2C_3-N_3C_2)]^{1/2}}{2\,(N_2\,C_3 - N_3\,C_2)} \tag{13}$$

$$d = (N_1 + bN_2)/(N_2 + bN_3) \tag{14}$$

$$m = [\{3\} - \{2\} - \{1\} + \{0\}]/4 \tag{15}$$

where

$$\{k\} = (k + 1)\,(k + 1 + b)\,(k + 1 - d)\,M_k \qquad k = 0,\ 1,\ 2,\ 3 \tag{16}$$

$$a = \frac{(b+1)(b+2)}{b(b+d)}\left[\frac{\{1\}}{2+b} - \frac{\{0\}}{1+b} - m\right] \tag{17}$$

$$c = \frac{(1-d)(2-d)}{d(b+d)}\left[\frac{-\{1\}}{2-d} + \frac{\{0\}}{1-d} + m\right] \tag{18}$$

First stage

In the first stage of the analysis the assumption is made that $m = 0$. The relevant equations for the coefficients N and C are (9) and (10). The values of b, d, m, a and c are determined from equations (13), (14), (15), (17) and (18) respectively. Error condition 1 is tested after the determination of the value of b and error condition 2 after the determination of the value of d. The remaining error conditions are tested after all of the parameter values have been determined.

The authors of the method recommended that the parameter combination be accepted if none of the five error conditions are present. No mention was made of the condition where one or more of the parameter values a, b and d are negative.

Second stage

The second stage also consists of a single analysis. The only difference being that the assumption is made that $m \neq 0$ and consequently equations (11) and (12) are relevant. Otherwise the calculation is the same as that in the first stage.

Third and fourth stages

The third and fourth stages involve the iterative search for a successful combination of parameter values based on a progressively decreasing value of b within the range b_{max} to b_{min}.

The authors of the method recommend that the number of iterations allowed and the increment Δb be fixed. They make no suggestions regarding these two values, although as will be demonstrated below, the outcome of the analysis is sensitive to both of these assumptions. Furthermore, they recommend that if either error conditions 1 or 2 hold that the next lower value of b be attempted, but if error conditions 1 or 2 do not hold, but error condition 3 does, ie $(ab + cd) < 0$ that Δb be reduced by half and a new iteration sequence be started with $b = b_{max}$.

If no acceptable combination is found, the fourth stage is attempted based on $m \neq 0$ and the corresponding equations. If no valid combination is found, the algorithm fails.

The algorithm proposed by the authors exits successfully as soon as an acceptable combination is encountered. This implies the highest value of the parameter b that leads to a successful combination. Their justification for this procedure was that the right tail of the distribution is essentially independent of b when b assumes a large positive value, and b is presumed to be less than the upper bound b_{max} so that the contribution of the term $(1 - F)^b$ in equation (1) is not effectively eliminated.

Important points not made by the authors were that if a solution exists it is not in the form of a single combination of values, but a range of combinations and consequently a range of Q-T relationships, and that their algorithm arbitrarily accepts the combination with the highest b value without demonstrating that this is the optimum solution.

An analyst would require some assurance on these points.

Modified WAK/PWM iteration procedure

The essence of the modified procedure described below is that it does not exit in the third and fourth stages when a satisfactory combination of parameter values is encountered for the first time, but proceeds through the full analysis. The additional programming effort and computer time are minimal.

At each calculation step in stages 3 and 4, the first error encountered is registered and the algorithm proceeds to the next step. If none of the five errors are encountered, the parameter values are printed. If one more of the values other than that of m are negative, this set is identified (condition 6).

The user specifies b_{max} and b_{min} which should be set at 50 and 0.3 initially. The number of iteration steps is set at 18 which is convenient for a display on a 25 line computer screen. The value of Δb is 1/18 of the specified range. If one or more successful combinations are identified on the first run, the user can select new values of b_{max} and b_{min} to bracket the range of successful combinations and carry out another series of analyses.

Another modification is that Q_{10}, Q_{100} and Q_{1000} are determined from the successful combinations and presented together with the parameter values. These can be compared with the theoretical values derived directly from the parameters used for generating the sequence using equation (1).

Caution

Although this is a very flexible distribution which can fit most data sets encountered in flood hydrology, the reliability of the estimates of the parameter values from a single data set is poor, and the method can only be applied with confidence on a regional basis. This application is described in more detail in later chapters.

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Chapter 3

STORM RAINFALL

Contents

Note:

The figures and tables in this chapter should not be used for calculation purposes. Those that are recommended for inclusion in calculation methods are repeated in the appendix to Chapter 13.

3.1 INTRODUCTION

3.1.1 A cautionary note

The application of the information presented in this chapter is discussed in detail in later chapters. However, at this stage it is important to identify the two basic questions relating to the estimation of flood magnitude from storm rainfall and the way that they influence the determination of storm rainfall characteristics described in this chapter. The questions are :-

1. How can the flood magnitude-frequency relationship be determined from the properties of storm rainfall?
2. How can the magnitude of the worst flood that can reasonably be expected at the site be derived from storm rainfall?

The answer to the second question is relatively simple. Storm properties and recorded data have to be examined to determine the most severe combinations of storm rainfall depth, area and duration that have been experienced in the past, and suitable extrapolations made to estimate expected future maxima.

The difficulty lies in developing an answer to the first question.

The principal assumption in deterministic methods of flood magnitude-frequency relationships is that the annual exceedance probability (AEP) of the resultant floods is the same as that of the causative rainfall. Consequently it is important that the methods used in the development of the rainfall-runoff relationship should not violate this assumption.

The magnitude of a flood is a function of :

(a) The properties of storm rainfall, which include the depth-duration-frequency relationship of point rainfall, and the relationship between point rainfall and rainfall over the catchment. These relationships are discussed in detail in this chapter.

It is also a function of the following additional factors which are discussed in Chapter 4 :

(b) Fixed catchment characteristics such as the area, slope, soil permeability and vegetal cover.

(c) Catchment processes which control the conversion of rainfall into runoff. These include interception, infiltration and the sub-surface movement of water.

(d) Antecedant conditions which modify the catchment processes. These are principally antecedant rainfall, antecedant catchment moisture losses due to evaporation, and antecedant river flow.

The assumption that the flood AEP can be derived from that of the causative rainfall AEP rests in turn on the following assumptions :

(i) That the controlling conditions do not vary from storm to storm. The catchment characteristics fall into this category.

(ii) That there is a direct, unique relationship between the variables that change from storm to storm. This implies that they are statistically dependent and consequently any one of them can be used to predict the flood AEP. The properties of storm rainfall, catchment processes and antecedant conditions fall into this category.

Other than the area of the catchment, most of these assumptions do not hold. The issue then becomes the extent of the deviation from the assumptions, and the magnitude of the influence on the derived flood value. This in turn depends on the departure from linearity of the relationship, particularly in the extreme conditions which are of interest in flood frequency analyses.

When variation in a key relationship is present, it is essential that the mean relationship and not the upper or lower envelope values be used in rainfall-runoff models where the objective is to retain the rainfall AEP in the model.

3.1.2 Overview

The severity of a flood is a function of the properties of the storm rainfall that generates it. These properties include the depth, areal spread and duration of the rainfall, as well as variations in rainfall intensity in space and time over the catchment during the storm.

The meteorological approach to storm rainfall is to study the development and movement of rain producing systems, whereas the hydrological interest is in the precipitation pattern that these systems leave on the ground. If the hydrological application assumes that the rainfall intensity is constant in time and space over the catchment then a straight forward analysis of rain gauge data will be adequate. However, it is very difficult to estimate time and space variations of rainfall during the storm from available rain gauge data, and therefore these have to be derived from a qualitative understanding of the meteorological mechanisms which produce the rain.

Two recent publications *Climatic change and variability in southern Africa* (Tyson, 1987), and *The atmosphere and weather of southern Africa* (Preston-Whyte and Tyson, 1988) describe in detail the atmospheric processes which result in storm rainfall over southern Africa. They are valuable references, but neither of the publications provides quantitative analyses of storm rainfall, nor have southern African meteorologists concerned themselves with analyses of the depth-area-duration-frequency (DADF) relationships of flood producing rainfall events, or with procedures for estimating maximum rainfalls.

The most extensive quantitative analyses of storm rainfall by hydrologists in South Africa were those undertaken by the Hydrological Research Unit of Witwatersrand University (1969 through to 1981), and by the Department of Water Affairs in its technical report series. Both of these sources relied primarily on the analysis of daily rainfall data. Estimates of the properties of short duration precipitation have been based on the limited amount of recording rain gauge data, while estimates of the areal and time distribution of storm rainfall have had to be based on the very limited southern African data and tested against relationships developed overseas.

In more recent years further large scale analyses of storm rainfall have been undertaken by Schulze and his colleagues in the Department of Agricultural Engineering of the University of Natal, Pietermaritzburg.

3.1.3 Principal properties of flood producing rainfall

The properties of flood producing rainfall of interest to the hydrologist are :-

(a) The area over which the precipitation occurs,

(b) The depth of the precipitation over this area (from which the volume can be derived),

(c) The duration of the precipitation over the area (from which the intensity can be derived), and

(d) The frequency with which precipitation properties are equalled or exceeded.

These properties cannot be determined directly and have to be inferred from rain gauge records which are measurements at fixed points. The following relationships have to be developed for determining catchment rainfall from point rain gauge data.

- The depth-duration-frequency relationship of precipitation measured at a single point.
- The area reduction factor (ARF) which is applied to point precipitation to obtain the mean precipitation over a catchment.
- The time distribution of point precipitation, which is how the intensity of precipitation at a point varies with time.
- The time distribution of precipitation over a catchment, which is a function of a number of factors such as the speed and direction of the storm and its rates of growth and decay.
- The upper limit to the maximum depth of precipitation that can occur over a catchment for a specified duration if such a limit is assumed to exist.

- The possible effect of long term persistence (measured in decades) on all of the above relationships.

These properties have to be derived from available data, the principal source of which is the Weather Bureau's network of some 6 000 daily rainfall stations, of which approximately half are currently in operation, and its recording rain gauges of which only 64 have sufficiently long records to be included in these analyses, but many more are currently in operation.

3.2 PRECIPITATION

The mean annual precipitation (MAP) over southern Africa varies from less than 25 mm in the Namib desert to over 3 000 mm in the mountains of the southern Cape and eastern Transvaal. The percentage of the MAP falling in the summer months (October to March) varies from 10% in the south-western Cape Province to 90% in the northern Transvaal. There is no apparent correspondence between these two rainfall characteristics.

Meteorological conditions which produce high intensity precipitation vary from the influx of broad streams of warm equatorial air from the north which are the source of most of the long duration precipitation over large catchments in the interior of South Africa, to short duration, small area precipitation from local convective thunderstorms which are the principal cause of floods in small catchments.

Topography also plays a role. The escarpment around the eastern and southern rim of South Africa provides an effective orographic barrier. This produces much of the country's runoff, although precipitation intensities tend to be less than those produced by convective activity.

The overall scene in southern Africa is one of a great variety of climatic, meteorological and topographical conditions which control flood producing precipitation.

3.2.1 Factors which control the precipitation process

Precipitation can be in the form of rain, hail, sleet or snow, and it occurs when moist air cools to below dew point or below freezing point in the presence of condensation nuclei. The water droplets coalesce, and when large enough fall to the earth. When condensation takes place at sub-zero temperatures hail may be formed. Snow is not hydrologically significant in southern Africa.

The following is a brief, general description of the major factors which control the depth, area and duration of precipitation.

Precipitable moisture. The moisture carrying capacity of air is dependent on its temperature. Warm air has a significantly greater potential moisture carrying capacity than cold air. The oceans are the major source of moisture which is subsequently precipitated over the continental areas when favourable conditions occur. The southern African subcontinent projects into the southern oceans reaching 35° south latitude, with the cold Atlantic ocean on its west coast and warmer Indian ocean on its east coast. The surface temperatures of these oceans have an appreciable influence on the precipitable moisture content of the air masses moving across the subcontinent.

Rate of influx. As depth of precipitable moisture in a vertical column of air rarely exceeds 25 mm, the rate of precipitation at a point will depend on both the initial moisture content of the air as well as the rate of influx from outside the area.

Condensation nuclei enhance the initial formation of cloud droplets which are very small and readily supported by the natural turbulence in the air. Under favourable conditions the droplets will coalesce and fall as rain. When condensation takes place at sub-zero temperatures freezing nuclei fulfill the same function.

Release of thermal energy. Thermal energy gained during the evaporation of water at its source is released back into the environment during condensation and freezing processes. This accelerates the vertical movement of the air and consequently precipitation intensity.

Cooling mechanisms. Air temperatures and pressures decrease with altitude, although conditions may occur when there are local increases in temperature with altitude (temperature inversions). The primary cooling mechanisms are those which cause air to rise and therefore to expand and cool. The rate of cooling of the air will determine the rate of precipitation. Lifting (and therefore cooling) mechanisms include:-

(i) Topographic features in the path of incoming moist air which cause the air to rise and cool (orographic lifting).

(ii) Solar heating of the ground surface warms the air close to the ground and pockets of warm air rise through the overlying cooler air. Under favourable conditions vertical, turbulent air streams develop in this unstable environment (convective lifting).

(iii) A number of different processes can cause air masses to converge, and then rise when their forward movement is constrained. Similarly, because air masses of different temperature and moisture content do not mix readily, opposing streams of air will cause the warmer air to rise over the colder air (convergence lifting).

(iv) Cyclones are low pressure areas where divergence is taking place in the upper layers drawing air in at lower levels and so causing it to rise and flow outwards at higher levels (cyclonic lifting).

3.3 TYPES OF PRECIPITATION

3.3.1 Orographic precipitation

Orographic precipitation occurs when winds force air up a mountain or escarpment slope. The rate of precipitation (precipitation intensity) will depend on the vertical wind velocity which in turn is a function of the steepness of the slope relative to the wind direction, and the horizontal velocity of the prevailing wind. Precipitation is usually heaviest at the level of the cloud base, and where this is below the summit the intensity may decrease with increase in elevation above the cloud base. Over the lee slopes the air is drier and hotter than the air at the same altitude over the windward slopes. This warm, dry rain-shadow area is due to the loss of precipitable moisture on the windward slope as well as the decrease in relative humidity as it warms up when descending the lee slope.

Orographic lifting can also take place when the velocity of onshore winds is decreased by frictional drag when they flow across the coast. Often orographic lifting may trigger off convective or cyclonic precipitation in unstable air conditions. Orographic precipitation is the major source of rainfall along the southern and eastern escarpments and mountain ranges in southern Africa. Generally the intensity of orographic precipitation is low but it continues for as long as the wind direction is sustained. Typically, orographic precipitation will therefore be of low intensity but long duration (often several days), and over an area which is controlled by the local topography and therefore predictable in its location and extent.

When large scale systems produce extensive rain the orography may enhance the precipitation rates considerably.

3.3.2 Convective precipitation

Most solar radiation passes through the atmosphere and is absorbed by, and warms the earth's surface. Long wave radiation from the heated surface as well as contact with this surface warms up the lower air layers. This warm air expands and becomes more buoyant than the overlying air. Under these unstable conditions, cells of warm air break through the cooler air blanket above them and create a pathway for more warm air to follow in their wake. As these pockets of warm air rise they expand and cool. The cooling progressively reduces the air's capacity to carry moisture until saturation point is reached. Further cooling causes condensation to take place, clouds to form and moisture to precipitate out of the clouds in the form of rain, sleet or hail and thermal energy to be released to the environment.

The lifting, and consequentially cooling and precipitation will continue until the temperature of rising air is the same as that of the ambient air. The total depth of precipitation on the ground will depend on the initial moisture content and the ultimate temperature of the rising air. The intensity of the precipitation will depend on the rate of cooling which is a function of the vertical velocity which in turn is largely determined by the temperature difference between the rising and ambient air.

Under favourable conditions pockets of warm, moist air will rise to heights of several thousand metres. In the process large volumes of water and ice are released. Once the raindrops or hail grow to sizes too large to be supported by the updraught, they fall back to earth dragging cold air down with them. This causes the rising air cell to collapse, but the cold air is forced outwards when it reaches ground level and displaces surrounding warmer air upwards, so initiating the growth of another cell.

Once started, a convective storm is self-sustaining as it sweeps up the warm moist air in its path and deposits precipitation behind it. Occasionally two storm systems will merge and cause extremely severe rain. Fortunately these violent systems soon exhaust the local moisture laden air and they are therefore only short lived phenomena.

Preston-Whyte and Tyson (1988) describe super cells which are convective storms which usually exhibit a massive single cell structure, circular to elliptical in shape, and extending 20-30 km in horizontal and 12-15 km in vertical dimensions. A rainfall intensity of 86 mm in one hour was recorded at Jan Smuts Airport on 22nd December, 1987.

The characteristics of convective precipitation are therefore almost the opposite of those of orographic precipitation. Convective storm precipitation is far more intense but only of short duration (measured in minutes whereas orographic precipitation may last for days). In convective precipitation there are very abrupt changes in intensity in both time and space whereas orographic precipitation is fixed in space and almost uniform over long time spans. Storm movement is characteristic of convective precipitation whereas orographic precipitation is static.

3.3.3 Cyclonic precipitation

Zones of low pressure which develop over the interior of southern Africa in the summer months are typically located along a trough extending north-westwards from the south-eastern Cape coast through to western Botswana. They are not fixed in this position. Precipitation occurs in the north eastern sector of the trough due to the influx of warm, moist air from that direction. Under favourable conditions these troughs give rise to heavy prolonged rainfall over a wide area, and can cause widespread flooding over the interior of the sub continent.

Cut-off lows are unstable systems associated with strong vertical air motion and consequently intense rainfall. They have been the cause of flooding of many parts of southern and south-eastern South Africa.

Tropical cyclones (called hurricanes or typhoons in the northern hemisphere) differ from the extratropical low pressure zones in that they originate in and are mostly confined to oceanic areas, and the precipitation is far more intense. They move slowly through an air mass of uniform temperature rather than between air masses of different temperatures. They are generated in the hot air masses over the ocean and their principal driving force is the same as that of convective thunder storms. However, the influx of fresh supplies of moist air is not limiting as it is in the case of convective storms over the interior, and the systems are self sustaining for as long as they move through an area which has a large mass of warm moist air to feed them. For this reason tropical cyclones quickly subside when moving over land. Fortunately, these cyclones only rarely reach South Africa, and then only in their dying stages, although even they are capable of producing large volumes of high intensity rainfall, as demonstrated by the tropical cyclones Domoina and Imboa which caused severe flooding over south-eastern Transvaal and northern Natal in January and February 1984 respectively.

3.3.4 Frontal precipitation

A front is a zone of discontinuity between converging but dissimilar air masses. Pronounced gradients in temperature, pressure, wind and moisture content therefore occur across a front. A cold front is the result of cold air moving beneath and displacing warmer air upwards. The interface is a steep wedge having a backward slope. The steepness of the front and speed of its advance together produce an abrupt lifting of the warm air, and if this air is unstable with a high moisture content, violent storms with high intensities but short durations can result.

A warm front is formed where a warm air mass displaces colder air. Ground friction drags the bottom edge of the retreating cold air into a thin wedge with a flat slope. The flatter slopes of the front and slower movement produce less violent changes and consequently less intense but longer duration precipitation than that associated with a cold front. Precipitation intensity will depend on the moisture content and temperature stability of the warm air. Where the warm air is stable precipitation will be light at first, then moderate and fairly prolonged. If the warm air is unstable, more violent weather results as ascending currents create thunderstorms ahead of the front. Heavy downpours are interspersed with lighter showers.

Not all fronts produce rain. For example a warm air mass moving over cool ground is thermodynamically stable in that the lower layers are cooled by contact with the surface and tend to remain there, and not generate vertical turbulence as would happen if a cold air mass moved over ground warmer than itself.

When the forward movement of a front is slow, the rain is usually prolonged but less intense than would be caused by a rapidly moving front.

Generally there is an inverse relationship between intensity and duration of precipitation associated with the passage of frontal systems over an area. These vary from intense but short duration precipitation when a cold front passes through an area of warm, moist unstable air, to long duration but only moderate intensity precipitation associated with a warm front moving through local stable air.

3.4 PRECIPITATION DATA

Although many of the hydraulic principles familiar to the engineer also apply to the movement of air masses, because air is compressible its properties are also strongly influenced by temperature and pressure gradients in addition to the elevation gradients which dominate the flow of water.

The meteorological processes at work in a storm are highly complex and it is not as yet possible to derive storm precipitation characteristics from meteorological data. Nor is it possible to calculate the general depth-area-duration-frequency relationships of storm precipitation from meteorological information. These relationships have been derived almost exclusively from rain gauge data - primarily daily precipitation observations supplemented by recording rain gauges having a time resolution of ten to fifteen minutes.

Radar observations are likely to provide a powerful tool for the study of the areal distribution of high intensity, short duration precipitation. Radar assisted weather modification studies currently in progress will greatly assist in the understanding and quantification of storm precipitation within the next decade or so. These studies are already under way in South Africa and elsewhere.

Hydrologists therefore have to rely on largely empirically derived depth-area-duration-frequency properties of storm rainfall, plus a broad understanding of the basic precipitation causative processes.

The major source of precipitation data is that collected by the Weather Bureau from its nation wide rain gauge network of daily and recording rain gauges.

3.4.1 Format of available data

Published information for the daily observation stations includes :-

- Monthly and annual precipitation for each observation year.
- Mean annual precipitation (MAP)
- Maximum recorded annual precipitation.
- Maximum recorded observation day precipitation for each observation year.
- Average, maximum and mean number of days per year in which precipitation
- occurred which was :-
  - more than or equal to 0,2 mm

  - more than or equal to 1,0 mm
  - more than or equal to 10,0 mm
- Number of days per year during which thunder was heard.
- Number of days per year on which hail occurred.

The latest publication at the time of writing is publication WB40 on the climate statistics of South Africa up to 1984 (Weather Bureau, 1986).

Published information derived from *recording rain gauges* includes the following:

- Highest recorded precipitation for each month of the year for durations of 15, 30, 45 and 60 minutes, and 24 hours.

- Expected maximum monthly precipitation for the above durations and return periods of 25, 50 and 100 years.

This information is published in WB36 (Weather Bureau, 1976).

These extreme values derived from autographic data are not in a format suitable for direct application to hydrological problems, so the data were reprocessed by the Department of Water Affairs into the following format (Division of Hydrology Technical Note No. 78).

- Mean annual precipitation.

- Expected maximum annual precipitation for durations of 15, 30, 45 and 60minutes, and 24 hours, and return periods of 5, 10, 15, 20, (25), 50, 100, 200 and 500 years as well as mean values.

- Statistical properties of the recorded data.

3.4.2 Correction factors

Correction factors have to be applied when converting data recorded at specific times of the day to independent durations of the same length (for example the maximum observational day rainfall measured at 08h00 every day will be less than the maximum observed during any 24-hour period). The commonly used correction factors are :-

TABLE 3.1 Clock correction factors

From	To	FSR	Hershfield
Quarter hour	15 minutes	?	?
One clock hour	60 "	1,15	1,13
Two clock hours	120 "	1,06	-
Six clock hours	360 "	1,015	1,04
One observation day	24 hours	-	1,13
Twenty four hours	1 440 minutes	-	1,01

The correction factors used in the *Flood Studies Report* were derived directly from the examination of 50 data sets. Hershfield (1962) derived his correction factors by adding one-half of the maximum adjacent clock-hour quantity (adjacent 2 hours for 120 minutes etc.). He found that this relationship gave results differing by not more than 5% in the total of 400 storms that were examined.

In South Africa the pre-electronic data capturing recording rain gauges had recorder charts which were graduated in clock quarter-hours, and the data were visually extracted for these time intervals. The 30, 45 and 60 minute maximum rainfalls were obtained from the maxima of 2, 3 and 4 successive quarter-hour totals respectively. For example the derivation of the maximum observed precipitation depths for these durations at Port Elizabeth were those recorded during the storm of 1st September, 1968 as shown in Table 3.2:-

TABLE 3.2 Port Elizabeth storm of September, 1968. Derivation of maximum precipitation depths from fixed clock time interval observations.

Time interval	Precipitation	Maxima			
		15 min	30 min	45 min	60 min
10:00 - 10:15	15,5 mm				
10:00 - 10:30	26,7		26,7		26,7
10:30 - 10:45	34,7	34,7	34,7	34,7	34,7
10:45 - 11:00	22,0			22,0	22,0
11:00 - 11:15	28,7			28,7	28,7
11:15 - 11:30	24,0				
11:30 - 11:45 etc.	27,5				
Total		34,7	61,4	85,4	112,1

However, the maximum 24 hour precipitation at autographic stations coincides with the observation day and is not the maximum of 96 consecutive quarter-hour observations. A correction factor of 1,13 must be applied to the maximum daily rainfall to obtain true 24-hour precipitation.

No correction factors were applied to the data in the Weather Bureau's publications nor in the Department of Water Affairs Technical Note 78.

3.4.3 Recent developments

At the time of writing this handbook the autographic data are in the process of being redigitised. This will provide a more accurate and more extensive data base. No clock correction factors need be applied. However, the principles given below remain valid and should be applied directly when the problem justifies the greater effort. Note that the description "autographic" refers to chart recorded data as distinct from electronically recorded data.

3.5 DEPTH-DURATION-FREQUENCY RELATIONSHIP STUDIES

In most methods used for deriving flood magnitude from storm rainfall, the interest is in the depth-area-duration-frequency (DADF) relationship. In this relationship it is assumed that the frequency can be attached to the rainfall depth, and that the depth-duration and depth-area relationships are unaffected by the frequency of occurrence. The implication is that the **average** (or median) depth-duration and depth-area relationships are unaffected. However, it does not imply that deviations from the average will not affect the assumed DADF relationship. (See cautionary remarks at the beginning of this chapter).

These simplifying assumptions were adopted in the various methods developed by Witwatersrand University's Hydrological Research Unit as well as in overseas methods including the Australian guidelines (Pilgrim, 1987), but the approach has been criticized on two grounds :-

(i) The use of the median rather the mean has been debated over the years (eg Foster, 1949 p 107, and Irish, 1975). The issue has not yet been resolved (Pilgrim, 1987 p6).

(ii) The basic premise of the assumption has been challenged (Irish, 1975 and Colton, 1975). Eagleson (1970 p 196) describes an alternative approach making use of the joint probability density function of storm duration and total storm precipitation depth. Pilgrim (1987), lists this as a possible alternative although it was not used in the development of design data for the Australian guidelines.

Several routes can be followed to determine the full DADF relationship, the simplest being :-

Step 1. Determine the depth-frequency relationship for daily rainfall by choosing a single probability distribution and assuming that it is valid for the full DADF relationship.

Step 2. Derive the relationship between daily rainfall and shorter duration rainfall.

Step 3. Derive the relationship between point rainfall and areal rainfall.

Alternatively, two-way relationships can be examined. These are usually the depth-duration-frequency relationship and the depth-duration-area relationship.

Finally, the full DADF relationship for the whole spectrum of depth, area and duration values can be determined.

The basic assumption in the traditional development of the DADF relationship is that the individual relationships are continuous across the whole spectrum of values and can be described by continuous mathematical functions. This is unlikely when the properties of the different types of flood producing storms are considered. This being so, the next question that has to be addressed is whether the mathematical approximations are adequate for the intended application.

Table 3.3 is a depth-duration-frequency matrix which shows the principal durations and return periods of interest to the hydrologist. The right-hand side of the table contains information which can be derived from the records of the Weather Bureau's daily observation stations (Weather Bureau, 1986). Data for the whole matrix can only be derived from the records of stations which have recording rain gauges.

It seldom happens that a recording gauge with a long record is located within the catchment which is being investigated. In most cases use has to be made of a general depth-duration-frequency relationship derived from daily precipitation observations in, or in the vicinity of the catchment, or (and sometimes preferably) derived from general relationships developed for the region.

TABLE 3.3 Determination of the maximum depth of precipitation for a given duration and return period by developing a relationship between data from quarter-hour observations from a small number of stations, and daily observations from a large number of stations.

	DATA FROM QUARTER-HOUR OBSERVATIONS	DATA FROM DAILY OBSERVATIONS			
RETURN PERIOD IN YEARS	DURATION OF PRECIPITATION	DURATION OF PRECIPITATION	OTHER VARIABLES		
	minutes hours 1 2 4 6 12 24 2 5 10 15 30 45 60	days years 1 1 2 3 7	days with thunder	max 24 h precip	days with > 10 mm
1					
2	A				
Mean	D	X	X	X	X
5	C	(MAP)			
10	B				
20					
50					
100					
1000					
10000					
PMP					

The procedure for deriving an estimate of precipitation depth for a required duration and return period from daily observation data can be carried out in three steps:-

(a) Develop a relationship between mean annual data from daily observations and (say) the mean annual 60 minute rainfall and thereby determine the value of point D in the matrix.

(b) Develop a relationship between the 60 minute rainfall depth and depths for other durations. Fill in the values in the horizontal lines in the short duration matrix.

(c) Develop a relationship between mean annual 60 minute rainfall depth and rainfall depths for other return periods. Fill in the vertical values in the short duration matrix.

3.5.1 Early South African investigations

Early South African estimates of the DDF relationship were based on American experience. Vorster (1945) used regional relationships developed by Yarnell (1935) for the USA and applied them to South Africa. He made use of maximum 24-hour data provided by the South African Weather Bureau which was subsequently published (Schumann 1943, 1956) to regionalize the relationships. Levinkind (1947) also used Yarnell's DDF curves and Schumann's 24-hour maximum rainfall to obtain similar relationships to that of Vorster, but did not find it necessary to develop separate regional DDF curves.

3.5.2 Hershfield, Reich and Bell

Reich (1962, 1963) was the first author to use data from autographic rain gauges in South Africa to develop DDF relationships for the country as a whole. He did this by testing relationships previously proposed by Hershfield (1962) for the USA and found them applicable to South Africa. Bell (1969) refined the model and showed that it fitted data from the United States, USSR, South Africa, Australia, Hawaii, Alaska and Puerto Rico. The reason proposed for the applicability of the model in all these countries is that high intensity precipitation for durations of up to two hours is primarily the result of convective thunderstorm activity, and that the meteorological processes within these storms are independent of their locality.

The relationships proposed by Bell are shown in matrix form in Table 3.4.

TABLE 3.4 Matrix showing factors to be applied to 60-minute/2-year return period precipitation to obtain precipitation depths for other durations and return periods proposed by Bell (1969).

Duration (minutes)		5	10	15	30	60	120
Depth/duration ratio		*0,29*	*0,45*	*0,57*	*0,79*	**1,00**	*1,25*
Return period years	Depth/return period ratio						
1	*0,86*						
2	**1,00**						
5	*1,35*						
10	*1,60*						
25	*1,87*						
50	*2,10*						
100	*2,32*						

Once one value (precipitation depth) within the matrix is known, all other values can be derived by applying the appropriate ratios. Bell (1969) found depths derived from the 10-year return period, 60 minute precipitation depth gave more consistent results than those derived from a 2-year return period, 60 minute precipitation. However, the estimate of the 10-year return period value was itself less accurate than the estimation on the 2-year return period value derived from the same data.

The final step was the calculation of the 60-minute precipitation from daily precipitation data. Hershfield, Weiss and Wilson (1955) developed a relationship for deriving the 1-hour/2-year precipitation from mean annual precipitation, mean annual number of thunderstorm-days, mean of annual series of maximum daily precipitation amounts and the mean annual number of days with rain.

Hershfield and Wilson (1957) reduced the number of independent variables to two : the mean of the annual series of maximum daily precipitation amounts and the mean annual number of thunderstorm days.

Reich (1963) determined and mapped the 1-hour/2-year maximum precipitation from 210 daily stations in South Africa. Using this information and Bell's relationships Reich produced maps and diagrams from which the precipitation depth P for any required duration t and return period T could be derived. Although not produced in equation form by Reich, the following equation can be derived from his diagrams:-

$$P_{T,t} = (0{,}35\ lnT + 0{,}76)\ (0{,}54\ t^{0{,}25} - 0{,}50)\ (1{,}83\ M^{0{,}67}\ N^{0{,}33}) \qquad (3.1)$$

where $P_{T,t}$ = the required precipitation for duration t (minutes) and return period T (years)

N = average number of days per year on which thunder occurred.

M = average 24-hour annual maximum precipitation in the range 50 to 115 mm.

If the average 24-hour annual maximum precipitation is in the range 0 to 50 mm, the parameters of M and N become $(4{,}32\ .\ M\ .\ N^{0{,}33})$. The value in the last pair of parentheses in equation 3.1 is the one hour/once in two years depth.

3.5.3 Witwatersrand University Hydrological Research Unit

The Hydrological Research Unit (HRU) undertook separate studies on large and small area storm precipitation in South Africa. Pullen, Wiederhold and Midgley (1966) reported on the depth-area-duration (DAD) properties of 170 large-area storms, while van Wyk and Midgley (1966) based their work on the records of 22 autographic rain gauge records varying in length from 5 to 26 years. The results of these two investigations were subsequently incorporated in the HRU reports 1/69 on design storm determination, and 1/72 on design flood determination. Although the method has the advantage of simplicity, its weakness is that the DDF relationships are related to MAP alone. This cannot be expected to be a good index of the DDF relationships over the wide range of meteorological conditions experienced within the three regions for which the relationships were developed. This is particularly so where most of the precipitation is of orographic origin.

The revised DDF relationships were published in HRU 2/78 (Midgley and Pitman 1978). The variables used in deriving the relationship were mean annual precipitation (MAP), locality (coastal or inland), and the published Weather Bureau autographic rain gauge data (le Roux 1974). A log EV1 distribution was fitted to the annual maximum series. Chi-square tests were used to confirm the goodness of fit, but neither partial duration series nor other distributions were tested. No other variables (daily maxima, thunderstorm data etc.) were tested for inclusion in the model.

The relationship used for various durations was :-

$$I = I_o/(1 + B.D)^n \tag{3.2}$$

where I is the intensity (mm/h) associated with a duration of $D(h)$

I_o, B *and* n are parameters associated with the given region, MAP and return period

also I_o is the intensity associated with an event zero duration.

The values of these parameters for a return period of 20 years and MAP of 500 mm are :

	I_o (mm/h)	B	n
coastal region	*122,8*	*4,779*	*0,7372*
inland region	*217,8*	*4,164*	*0,8832*

The results are tabulated in Table 3.5. The relationship between 20-year return period value and that for other return periods is shown in column 2 and the corresponding relationships for 5-year and 2-year return periods in columns 3 and 4.

TABLE 3.5 The relationship between precipitation depths for various return periods from HRU 2/78

Return period	Conversion ratios		
T years	T = 20	T = 5	T = 2
2	0,47	0,73	1,00
5	0,64	1,00	1,36
10	0,81	1,27	1,72
20	1,00	1,56	2,13
50	1,30	2,03	2,77
100	1,60	2,50	3,40

The values in the last two columns are not in the HRU publication, but are included here to facilitate comparison with Bell's relationship in Table 3.4. For example Bell's conversion ratio from T = 2 to T = 100 is 2,32 compared with the HRU ratio of 3,40.

The 15, 30, 45, 60 and 1 440 minute duration precipitation values were analysed independently using the log EV1 distribution. This resulted in the calculated daily maxima for the 100-year return period being less than the 60-minute duration for the same return period in the case of 8 of the 59 stations analysed. This introduced doubts about the suitability of the log EV1 distribution, particularly for long return periods.

Curve fitting procedures were used to develop the final relationship between MAP and the independent variables, and a coaxial diagram was constructed. The full mathematical equations used in the construction of the diagram were not given in the report, and the only method for determining the required storm precipitation is by using the coaxial diagram.

The following equations approximate these relationships and are used in the computer programs accompanying this handbook.

The form of the general equation is :-

storm rainfall = (storm duration) (regional factor) (frequency factor) (MAP factor)

These factors are:

storm duration	= t
factor for coastal region	= $122.8/(1.0 + 4.770t)^{0.7372}$
factor for inland region	= $217.8/(1.0 + 4.164t)^{0.8832}$
frequency factor	(second column in Table 3.2 above)
MAP factor	= $(18.79 + 0.17\ MAP)/100$

3.5.4 Van Heerden (1978)

Van Heerden (1978) proposed the use of precipitation intensity (mm/hour) rather than precipitation depth (mm) as the dependent variable. He accepted the Weather Bureau autographic rain gauge data and compared the relationship with that in overseas publications.

He modified Bell's relationships somewhat subjectively to fit the Weather Bureau data. In the discussion on his paper he revised the relationships, again somewhat subjectively.

3.5.5 The UK Flood Studies Report (1975)

This report is probably the most comprehensive analysis of storm precipitation properties yet undertaken. Its major drawback for applications in southern Africa is that it was only based on data from the United Kingdom where both the climate and precipitation causing mechanisms are less varied than those found in South Africa.

The records of approximately 200 recording rain gauge stations, 101 of which had 20 or more years of record, and 600 daily observation stations with an average of 60 years of record were analysed.

The basic statistics used for determining the precipitation depths for all the other durations were the 60-minute and 2-day rainfalls. The reason for the choice of 2-day instead of the conventional 1-day interval was because in the UK, the 09:00 reading frequently separated the precipitation into two less significant daily falls. The difficulty is that this statistic has to be derived by calculation, and not simply from direct observation as is the case with 1-day precipitation.

A significant time saving simplification was introduced by avoiding the full depth-return period calculation. This could be done because the properties of the GEV distribution used in the analyses are such that several key return period values can be derived directly by the simple method of dividing the ranked set of annual maxima into four quartiles. The mean of the middle two quartiles has a return period of 2 years, the mean of the upper two quartiles has a 5-year return period, and the mean of the fourth quartile has a 10-year return period. The four highest values can be used to estimate with less confidence values for rarer events up to n years where n is the number of years of record.

The depth-return period relationship was based on the value of the 5-year return period precipitation depth. Depths for other return periods were derived by applying a growth factor, which was found to be a function of the precipitation depth. This relationship was not found to be significant in the American studies.

Another factor r was determined. This is the ratio of the 60-minute/5-year period value to the 2-day/5-year return period value. The independent variables used for deriving this relationship were the average number of days on which thunder was heard; the 5-year return period precipitable water depth corresponding to a saturated column of air whose base temperature is the 5-year return period value of dew point persisting for at least 6 hours; and the 2-day/5-year return period precipitation. Isolines of both the 2-day/5-year return period and *r* values were mapped. Both values could therefore be extracted from the maps, and the 1-hour/5-year precipitation depth derived directly by applying the ratio to the 2-day value. Values for other durations were obtained by interpolation. Another factor was applied to this value to reduce it to the required return period.

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3.5.6 Department of Water Affairs

In 1974 the Weather Bureau (Le Roux 1974) published data obtained from its 64 autographic rain gauges for the period ending 1972. This was a considerable increase in the data base compared with that available to earlier authors. However, the data was in monthly format without annual maxima, and the longest return period was 25 years. Adamson (1977) reprocessed the Weather Bureau's tapes and derived annual maxima for durations of 15, 30, 45 and 60 minutes and observation days; and for return periods mean, 5, 10, 15, 20, 25, 50, 100, 200 and 500 years.

Adamson also used the EV1 distribution, which was used by the Weather Bureau but did not test any other distributions. The main shortcoming in this analysis is that it is based on a relatively small number of autographic stations and their sparse distribution over South Africa.

Van Heerden (paragraph 3.5.4 above) suggested the grouping of stations having similar intensity-duration relationships, while the HRU (paragraph 3.6.3 above) used the autographic data to develop relationships with mean annual precipitation on the assumption that these relationships are constant within two broad regions in South Africa (inland and coastal areas).

While both the Van Heerden and HRU methods are useful for preliminary investigations, neither of them can be expected to give accurate estimates due to the shortness of the records from which they were derived as well as the poor coverage over South Africa.

Encouraged by the results in the *Flood Studies Report* Alexander (unpublished course notes) decided to pursue the earlier methods developed by Hershfield, Reich and Bell. In this way, it was hoped that short duration precipitation could be derived from daily precipitation data. As there are some 3 000 current daily rainfall stations in operation and an equal number of closed stations, if such a relationship could be developed it would be more stable due to the much larger data base, and would be more applicable to the particular site being investigated as the data from several daily rainfall stations in the vicinity could be used instead of national averages.

Table 3.3 above shows the type of information that can be derived from recording rain gauge and daily rainfall data. The problem is to develop a relationship that links the two sets of data. Assuming that relationships between the various durations and between return periods can be found (see Table 3.4) all that is necessary is to relate daily rainfall properties to a single value within the recording rain gauge data matrix on the left hand side of Table 3.3.

A period of one hour is the obvious choice for the duration but there are several considerations which have to be taken into account when selecting the key return period. Hershfield recommended the use of a 2-year return period on the grounds that this gave the best correlation with daily data. Bell found that a 10-year return period gave a better base line for extrapolation within the matrix. A 5-year return period was used in the *Flood Studies Report* because this could be derived by taking the mean of the values within the upper two quartiles of a set of data without having to fit a specific extreme value distribution to it.

All three of these proposals are dependent on the assumed distribution. Hershfield, Reich and Bell as well as the HRU reports all used the EV1 distribution which has a fixed skewness, whereas the more flexible general extreme value distribution was used in the *Flood Studies Report*. Other authors have found that a log normal distribution gives satisfactory results.

It was decided to follow the example of the *Flood Studies Report* and use the 1-hour/5-year return period depth as the point of entry into the short duration matrix. This was also a compromise between the return periods used by Hershfield and Bell.

The next step was to find a general relationship between daily observation data and the 1-hour/5-year return period depth as the point of entry into the short duration matrix. This was also a compromise between the return periods used by Hershfield and Bell.

Convective storms are the main contributor to intense short duration rainfall in southern Africa. The frequency of occurrence of these storms can be mapped by using one of the following:

- Average number of days on which thunder was heard. [Fig. 3.1(a)]
- Average number of days per annum during which lightning ground strikes were recorded. [Fig. 3.1(b)]
- Average number of days per annum when more than 10mm of rain was recorded. [Fig. 3.1(c)]
- Average number of days per annum on which hail was reported. [Fig. 3.1(b)]

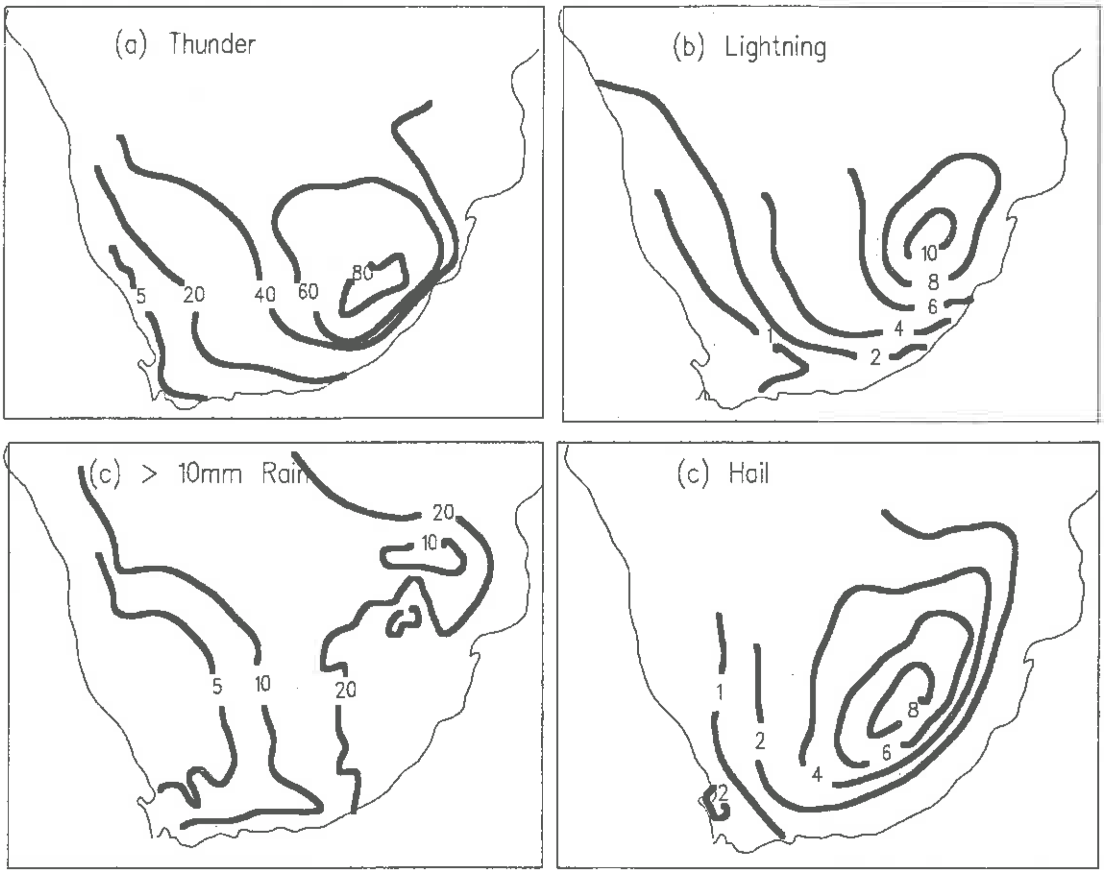

FIGURE 3.1 Generalized maps of the average number of days per annum on which thunder was heard (a); on which lightning ground strikes were recorded (b), from WB28 (South African Weather Bureau, 1965), and Kröninger 1981. Average number of days per annum when more than 10mm of rain was recorded (c), and on which hail was reported (d), from WB28 (South African Weather Bureau, 1965).

Candidate variables that can be considered for developing a method for deriving short duration rainfall depths from daily observations are :-

1. Mean annual precipitation.
2. Mean of the annual series of maximum daily precipitation amounts.
3. Mean annual number of thunderstorm days.
4. Mean annual number of days on which precipitation occurred.
5. Mean annual number of days on which precipitation exceeded a given threshold (say 0,1 mm, 1,0 mm or 10,0 mm).
6. Precipitable moisture in a saturated column of air with a base temperature associated with a specified return period.

Mean annual precipitation is not a good index of short duration precipitation in situations where a large proportion of the annual precipitation is of non-convective storm origin. It was used in the HRU studies, but omitted by Reich, Bell and the *Flood Studies Report.*

The average of the annual maximum daily precipitation depths was found to be a good indicator in the overseas analyses as well as by Reich, as was the average number of thunderstorm days per year. This indicates that convective precipitation is the main source of high intensity, short duration precipitation.

Variables 4 and 5 are possible contenders although the interdependence between them and the number of thunderstorm days should not be overlooked.

Variable 6 was used in the *Flood Studies Report*, but this information is not readily available for South Africa.

A stepwise multiple regression analysis was carried out with the 60-minute/5-year precipitation depth as the dependent variable, and the following independent variables :-

Dependent variable :

$P_{60,5}$ = Maximum 60-minute/5-year return period precipitation depth

Independent variables :

A = Mean annual precipitation

M = Mean of the annual series of maximum daily precipitation depths

R = Average number of days per year on which thunder was heard

S = Average number of days per year on which precipitation equalled or exceeded 10 mm.

The logarithm model was found to be a more efficient estimator than one using untransformed data. The simple correlation matrix of the log transformed data is shown in Table 3.6 :-

TABLE 3.6 Correlation matrix for variables used to determine $P_{60,5}$

Variable	A	R	S	M	$P_{60,5}$
A	1,000	0,708	0,978	0,823	0,748
R	0,708	1,000	0,736	0,583	0,755
S	0,978	0,736	1,000	0,828	0,783
M	0,823	0,583	0,828	1,000	0,854
$P_{60,5}$	0,748	0,755	0,783	0,854	1,000

The right hand column of the matrix shows that the best predictors for $P_{60,5}$ are M, S, R and A in that order.

The improvement in the explained variance by stepwise inclusion of the four variables was :-

M only	72,9%
M + R	82,8%
M + R + A	83,7%
M + R + A + S	84,3%

The two most significant variables are the maximum daily precipitation *M* and number of thunderstorm days *R* which confirms the findings of Hershfield and other authors. The introduction of mean annual precipitation and number of days with more than 10 mm precipitation does not significantly improve the model because of their strong interrelationships and correlations with the other two variables. They were therefore omitted from the model which then had the form : -

$$P_{60,5} = 1,55\ M^{0,63}\ R^{0,20} \tag{3.3}$$

This is very similar to the equation proposed by Hershfield which for the 5-year return period becomes :-

$$P_{60,5} = 1,36\ M^{0,67}\ R^{0,33} \tag{3.4}$$

In view of the confirmation of Hershfield's relationships between depth, duration and frequency by Reich and Bell, this was accepted for South African conditions. The full equation for determining the precipitation depth *P* mm for a duration of *t* minutes and return period of *T* years then becomes :-

$$P_{t,T} = 1,13\ (0,27\ \ln T + 0,56)\ (0,54\ t^{0,25} - 0,50)\ (1,55\ M^{0,63}\ R^{0,20}) \tag{3.5}$$

In this equation the factor 1,13 is the clock time correction; the second factor is Hershfield's relationship corrected to provide the required 5-year return period; the third factor is Hershfield's relationship relating the required duration to a 60-minute duration; while the last factor is the calculated 60-minute/5-year return period precipitation derived from daily data.

Table 3.7 shows a comparison of calculated short duration precipitation derived from daily data using equation (3.5) with data from the 64 recording rain gauge records published by the Weather Bureau. The differences expressed as percentages of the recording rain gauge data are also shown. *This does not imply that the depth derived from recording rain gauge data is necessarily more reliable than that from daily observations*, particularly when the long return period depths are derived from short records. This is well illustrated by stations 6, 8, 9, 10, 13, 22 etc. where good relationships exist between the daily and recording rain gauge derived $P_{60,5}$ depths but the reliability deteriorates when $P_{15,50}$ depths are compared. In these cases the $P_{15,50}$ depths derived from daily data may well be more trustworthy than those from the recording rain gauge record.

In Fig 3.2 the $P_{t,T}$ values derived directly from the recording rain gauge data are compared with those derived from equation 3.5. The dotted lines show the 20% difference bounds. The relationship for the $P_{60,5}$ precipitation is a function of the regression analysis used to derive it. The relationship for the $P_{15,50}$ precipitation confirms Hershfield's factors for deriving this from the $P_{60,5}$ estimates. This close agreement must, however, be tempered by the fact that both Hershfield and the Weather Bureau used the EV1 distribution in their analyses, so the close agreement is not unexpected, and does not necessarily provide proof of the accuracy of the method.

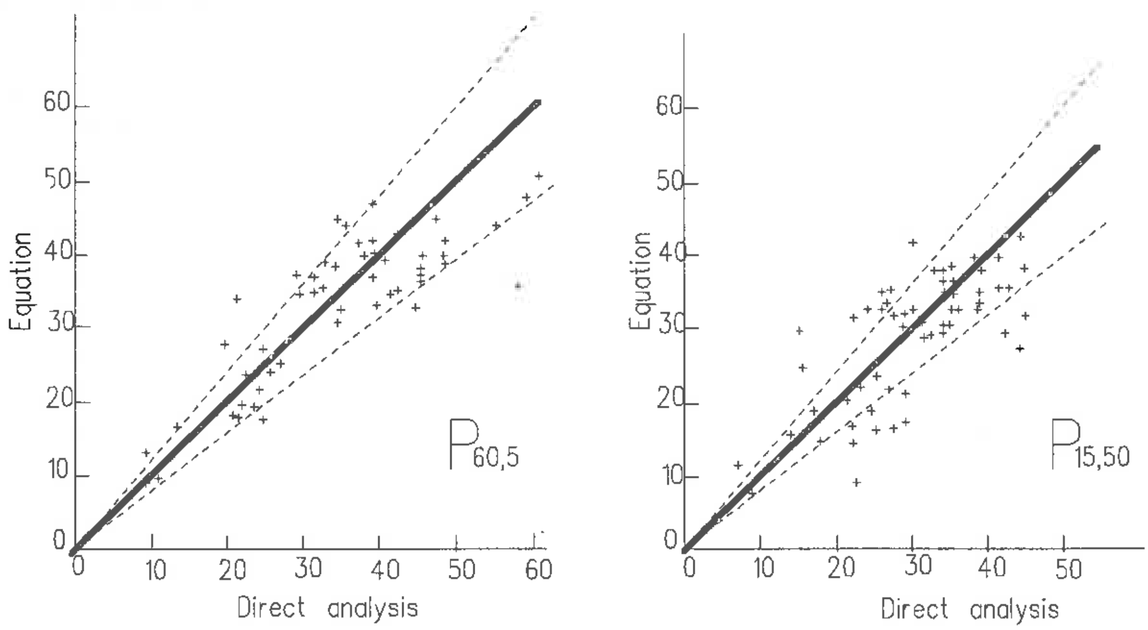

FIGURE 3.2 : Comparison between short duration precipitation depths derived from recording rain gauge and daily observation data.

3.5.7 Subsequent studies by the Department of Water Affairs

Adamson (1980) analysed the daily rainfall records of some 2 000 stations in South Africa and South West Africa to derive 1, 2, 3 and 7 day point precipitation depths for return periods of 2, 5, 10, 20, 50, 100 and 200 years. He found that the partial duration series using the truncated log normal distribution gave statistically more reliable results than the partial duration series with truncated exponential distributions, or annual series using two and three parameter log normal, EV1, EV2, log Pearson Type 3, log EV1, normal with Box-Cox transformation, or the modified Wakeby model estimated via incomplete moments.

Table 3.8 is an extract from Adamson's study and shows the analyses for eleven stations in the Pretoria area. The results vary over a narrow range despite the differences in length of record, mean annual rainfall and observed maxima. This stability plus the large number of stations and their long length of record gives additional confidence in the results. The coverage is such that the positions of the stations in, or close to an area of interest can be plotted and isolines drawn to obtain more accurate information at a particular point. Alternatively average values for a number of nearby stations can be used instead of a value from a single station which may not be representative of the area of interest due to outliers in the observations or due to topographical or other anomalies in its location.

Undoubtedly, Adamson's study is the most reliable source for 1-day to 7-day precipitation estimates so far available in South Africa. This will also give a more stable base from which to derive shorter duration precipitation estimates.

TABLE 3.8 Extract from Department of Water Affairs' technical report TR 102 (Adamson 1980) showing one to seven day precipitation depths for stations in the Pretoria area.

WEATHER BUREAU NUMBER.	STATION	YEARS OF RECORD	MEAN ANNUAL RAINFALL	DURATION	MAXIMUM ANNUAL MAXIMUM RECORDED	RECURRENCE INTERVAL. - YEARS - 2	5	10	20	50	100	200
513350	LYTTLETON	44	644	1 DAY.	202	56	75	99	119	146	169	201
				2 DAY.	202	69	93	111	130	157	180	204
				3 DAY.	229	77	104	125	146	176	201	228
				7 DAY.	275	102	143	173	205	251	289	330
513382	IRENE	61	660	1 DAY.	123	56	78	99	123	151	177	207
				2 DAY.	160	69	97	119	142	176	205	236
				3 DAY.	164	78	110	134	161	200	233	269
				7 DAY.	238	104	152	189	229	288	338	393
513404	PRETORIA (BRYNTIRION)	71	701	1 DAY.	169	62	88	108	130	163	191	223
				2 DAY.	237	78	112	138	167	210	246	287
				3 DAY.	246	88	128	158	191	241	283	330
				7 DAY.	299	111	156	190	226	278	321	368
513404A	PRETORIA (RIETONDALE)	47	686	1 DAY.	109	60	83	101	129	157	181	207
				2 DAY.	147	74	101	122	144	175	201	230
				3 DAY.	199	86	121	149	178	221	258	298
				7 DAY.	222	108	150	181	214	261	299	341
513413	DOORNKLOOF	50	604	1 DAY.	124	58	81	99	128	157	182	212
				2 DAY.	157	73	102	124	147	182	211	243
				3 DAY.	172	83	118	146	176	219	256	297
				7 DAY.	182	105	150	184	221	274	318	366
513417	OLIFANTSFONTEIN	51	598	1 DAY.	142	58	79	95	117	152	188	223
				2 DAY.	173	71	99	121	144	177	206	236
				3 DAY.	201	79	111	135	160	198	229	263
				7 DAY.	325	102	146	179	214	264	306	352
513437	PRETORIA (WATERKLOOF)	63	761	1 DAY.	141	69	98	121	146	183	215	250
				2 DAY.	188	86	121	149	178	222	259	301
				3 DAY.	200	96	137	169	204	255	299	347
				7 DAY.	277	120	170	208	247	305	353	405
513464	PRETORIA (KOEDOESPOORT)	48	616	1 DAY.	196	56	81	101	124	158	188	222
				2 DAY.	214	66	93	114	137	171	199	231
				3 DAY.	222	73	101	122	144	177	203	233
				7 DAY.	239	95	130	165	202	250	291	334
513494	PRETORIA (SILVERTON)	41	663	1 DAY.	193	59	83	103	124	155	182	211
				2 DAY.	213	72	99	119	140	170	196	224
				3 DAY.	223	81	111	133	156	189	217	247
				7 DAY.	247	108	152	185	220	270	311	356
513496	PRETORIA (LYNNWOOD)	53	675	1 DAY.	198	58	80	97	131	158	171	207
				2 DAY.	231	72	100	120	142	174	201	229
				3 DAY.	241	81	111	133	157	190	218	248
				7 DAY.	319	106	144	173	203	246	281	318

Adamson also provided an alternative algorithm for the modified Hershfield equation where lightning ground strike density is substituted for days on which thunder was heard. This is used in the recommended methods detailed in Chapter 13.

3.5.8 Australian guidelines (Pilgrim, 1987)

Australian hydrologists are in a more fortunate position than their South African counterparts in that they were able to analyse daily rainfall data from approximately 7 500 stations with more than 30 years of record, and short duration data (down to 6 minutes) from approximately 600 recording rain gauges with more than 6 years of analyzed record.

Several investigations showed that for site specific analyses the log normal distribution fitted by conventional moments (LN/MM) was the most suitable and convenient distribution to use. The log Pearson Type 3 (LP3/MM) distribution was found to be the most suitable when long return periods estimates were required which had to be based on regionally derived distribution parameters. The log normal distribution was used for analysing the primary rainfall analyses with "fine tuning" to the LP3/MM distribution where regional skewness of the logarithms was significantly greater then zero (on average about 0,2 but ranging to 0,8 in limited areas). The log normal distribution remained appropriate for regional parameter estimation over large areas where the skewness coefficient was close to zero.

Because of the larger data base it was found possible to use multiple regression techniques which included meteorological, geographical and topographical variables to reduce spurious local variations in the frequency-intensity relationship. (Similar anomalies in southern African data can be seen in Table 3.7)

Regional charts of 1-hour, 12-hour and 72-hour duration for 2 and 50 year return periods were prepared. For durations between one hour and 72 hours the best interpolation procedures were found to be those using the method of principal components applied to the geometric mean of the 2 year and 50 year intensities of 20 long term recording rain gauge records.

The procedure for determining the intensity-frequency-duration design curves for a given location consisted of five steps :

1. Read the log normal design rainfall intensities for 1, 12 and 72 hours for 2 and 50 year return periods.
2. Read the appropriate skewness coefficients from the regionalised skewness map.
3. Read the short duration geographical factors for durations less than one hour from maps.
4. Convert the partial series adjusted log normal distribution obtained in step 1 to LP3/MM distribution estimates.
5. Use the provided interpolation and extrapolation procedures to determine rainfall intensities for other durations and return periods.

The Australian relationships are site specific and cannot be applied directly to situations in southern Africa. The demonstrated greater reliability of the LN/MM and LP3/MM distributions in preference to the extreme value distributions confirms the conclusions reached by Adamson (1980) and throws doubt on the relationships based on the EV1 distribution in previous South African analyses.

3.6 AREA REDUCTION FACTOR

3.6.1 General

With the possible exception of orographic precipitation, it would be most unusual if the precipitation from flood producing storms was evenly spread in time and space over a catchment. Most storms have one or possibly two or more centres of maximum precipitation, with the depth of precipitation decreasing with increasing distance from the storm centre.

The usual method for estimating the average depth of precipitation over an area is to express it as a percentage of the point precipitation for the same duration and return period. This is called the area reduction factor (ARF) which decreases in value with increase in area.

Unfortunately there is no uniformity in the methods used to derive the ARF. Basically, the methods vary from the analysis of all storms occurring over a fixed area or array of rain gauges on the ground (fixed location analysis), to the analysis of the isohyets of a complete storm without regard to its geographical location (variable location analysis). The precipitation depth can be expressed as an average over the area or as the depth equalled or exceeded. The analysis may or may not have a statistical basis ie area mean depths are measured, ranked, and analysed using an assumed distribution in the same way as point precipitation is analysed.

Two diagrams are presented in HRU 1/72. For catchments up to 800 km^2 the ARF is given as a function of area and point intensity, while for larger catchments of 500 to 30 000 km^2 the ARF is given as a function of area and duration (Fig 3.3). The ARF for small catchments was obtained from studies of storms in the Pretoria area while those for larger catchments were derived from 170 major storms well distributed over South Africa. In both cases, the curves represent envelope boundary values and not mean values.

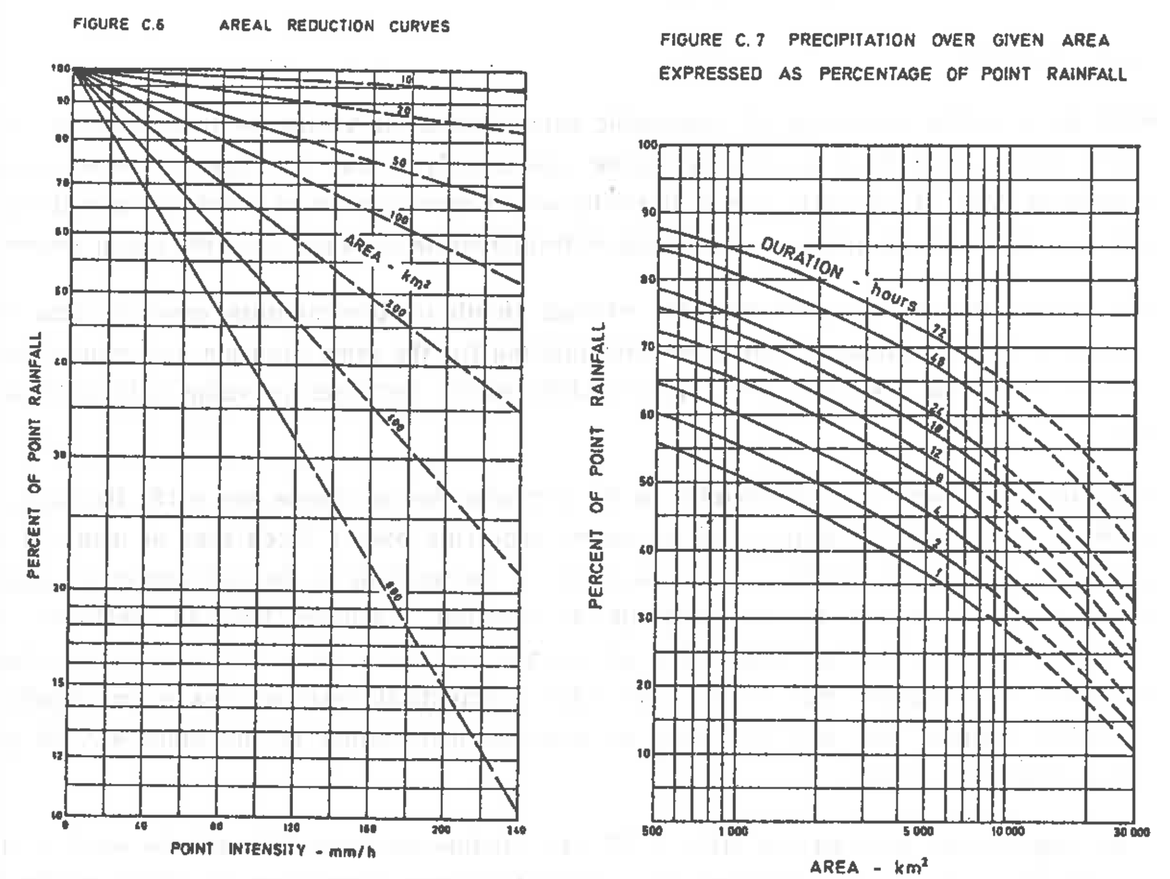

FIGURE 3.3 Area reduction factors in HRU 1/72

The ARF shown in these two figures is the variable location, storm centred ARF. The area reduction factors were derived by selecting high rainfall producing storms for which good records were available. Isohyets of total storm precipitation depth were drawn and the isohyet enclosing the greatest depth was chosen as the reference point in both time and space. Although this reference point was fixed for a particular storm it was not related to a fixed catchment on the ground. The areas within the isohyets were planimetered, and relationships developed between isohyetal area and the depth of precipitation *equalled or exceeded* (ie not the average depth of precipitation within the isohyet). *This method can be used for reconstructing the depth-area relationships for design storms, but it should not be used when uniform precipitation depth over a catchment is assumed in the design.* Its use also poses conceptual problems when this variable location, storm centred ARF has to be applied to a geographically fixed catchment.

Two other assumptions implicit in the method are that the ARF is independent of return period and geographical location. The *Flood Studies Report* confirmed the validity of these assumptions for the United Kingdom.

Fig 3.5 shows the form of the relationship presented in the *Flood Studies Report* and gives the ARF as a function of catchment area from 10 to 10 000 km² and duration from 15 minutes to 8 days, while Fig 3.6 shows the relationship developed by the US Weather Bureau in 1960, and which is still in use internationally.

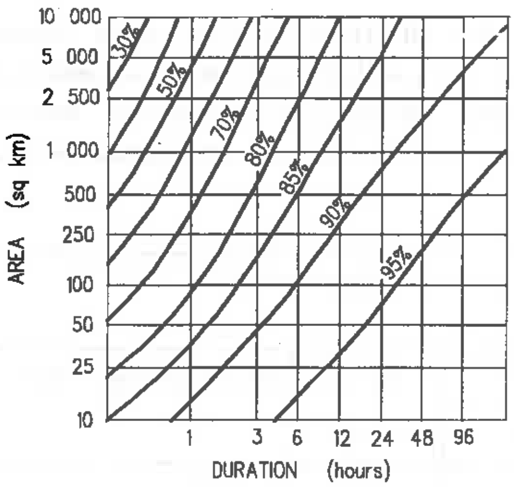

FIGURE 3.4 Area reduction factor derived from fixed location storm data showing average precipitation depths (after Flood Studies Report, 1975).

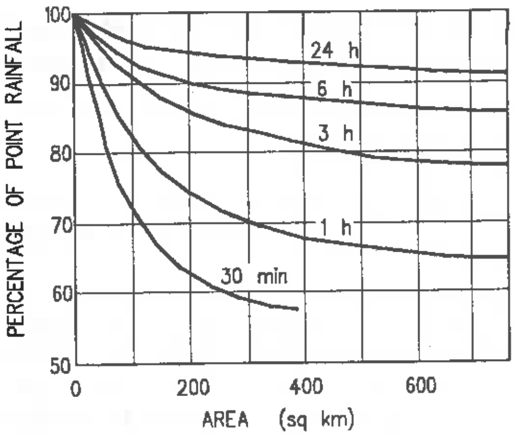

FIGURE 3.5 Depth-area diagram for the continental United States of America (after US Weather Bureau).

In the first supplementary report (1977) to the *Flood Studies Report* users were cautioned to make a clear distinction between the different definitions of ARF. The ARF used in the FSR is the percentage reduction which relates to the *statistics* of point and area precipitation, and is the relationship *used by the designer who has to determine the average precipitation over an area from point rainfall statistics.*

For example, in this method a predetermined area is considered as one large rain gauge by taking the average precipitation depths within the geographically fixed boundaries of the area for each storm. Thereafter these values are treated in the same way as for point precipitation from individual station. The ARF is the percentage reduction of the areally averaged depth for a specified area, duration and return period, divided by the point depth for the same duration and return period. In the *Flood Studies Report* it was found that the ARF was sensibly independent of return period although some slight reduction in ARF was observed with increase in return period.

The second definition of ARF relates to the way in which rainfall intensity decreases with distance from the centre of the storm in individual events.

If the designer wishes to assume a uniform distribution of precipitation in time and space over a catchment for the duration of the storm, then the first method for deriving the ARF should be used. However, this ARF may *not* be used for determining changes in the areal distribution of precipitation intensity during a storm.

Conversely, if the ARF has been derived from storm-centred data, it should not be used when assuming uniform distribution of precipitation intensity over a catchment.

The following hypothetical example illustrates the difference between these different approaches. There are six recording rain gauges (A to F) with 20 years of concurrent record located within a catchment of known size (say 160 km²). The observed maximum quarter-hour precipitation depths for each station for the 1988-89 observation year are shown in Table 3.9

TABLE 3.9 Calculation of area reduction factor

Storm No	Maximum 15 min rainfall (mm)						Areal	Storm centred ARF
	A	B	C	D	E	F	avr (mm)	
1	5	3	4	<u>7</u>	2	6	4,5	7/4,5 = 0,64
2	<u>9</u>	3	<u>9</u>	3	5	4	5,5	9/5,5 = 0,61
3	7	6	6	<u>11</u>	6	8	<u>7,3</u>	11/7,3 = 0,67
4	3	<u>8</u>	4	5	3	4	4,5	8/4,5 = 0,56
5	1	2	6	4	<u>7</u>	6	4,3	7/4,3 = 0,61
all other storms	<9	<8	<9	<11	<7	<8	<7,3	

From Table 3,9 :

Fixed location ARF for 1988-89

Maximum observed areal average rainfall = 7,3

Average of point maxima observed at the six stations

= 1/6 (9+8+9+11+7+8) = 8,7

ARF for 1988-89

= 7,3 / 8,7 = 0,84

Storm centred ARF for 1988-89

Average of values in last column = 0,62

his analysis for all other years. If the ARF is assumed to be independent of return ake the average of the 20 ARF values, and plot this against the size of the nt (160 km²). Repeat for other durations to find out how ARF changes with , and for other size catchments to determine how it changes with area.

y published general ARF relationships based on South African data are those in the HRU publications. A correction factor should be applied when using this ARF for ning the mean depth of precipitation over the catchment. Hershfield (1962) found an difference of approximately 15% at 2 500 km² for durations from 6 to 24 hours storm-centred and geographically fixed ARF. A further correction must be applied verage area depth is required from curves showing depths equalled or exceeded. e boundary values were used in the HRU curves and not mean values.

.10 shows the methods used in HRU 1/72, the Flood Studies Report and the US Bureau for deriving the ARF.

ABLE 3.10 : Methods used for the various authorities for determining the ARF.

ority	Figure	Location		Precipitation depth		Statistical basis
		Geographically fixed	Storm centred	Average over area	Equalled or exceeded	
HRU 2 etc.)	3.4		Yes		Yes	No
Flood ies ort	3.5	Yes		Yes		Yes
Weather eau	3.6		Yes	Yes		?

is the recommended ARF for southern African conditions in those situations where signer wishes to determine the average precipitation over the catchment from point statistics (Alexander 1980). This relationship is based on the *Flood Studies Report* hown in Fig 3.4 which in turn gives values which are in agreement with the US r Bureau's recommendation. However, it is adjusted in the short duration, small ent area region of the diagram. The reason for this change is that severe storms ing very high intensity precipitation have cell cores with areas exceeding 10 km² and ns (time of passage across a point on the ground) exceeding 10 minutes. Ten minutes finest practical time resolution of most recording rain gauges, and estimates of shorter duration precipitation based on extrapolation from longer durations are suspect when viewed in the light of the storm mechanisms which produce high intensity rainfall for durations less than 10 minutes.

There seems to be little justification for assuming an ARF of less than 100% in this region of the diagram, and the *Flood Studies Report* factors have been adjusted upwards accordingly. A comparison of the ARF values using various methods is given in Table 3.11. The FSR and US Weather Bureau values are in surprisingly close agreement bearing in mind the different methods used and different meteorological conditions, and are both appreciably higher than the HRU values.

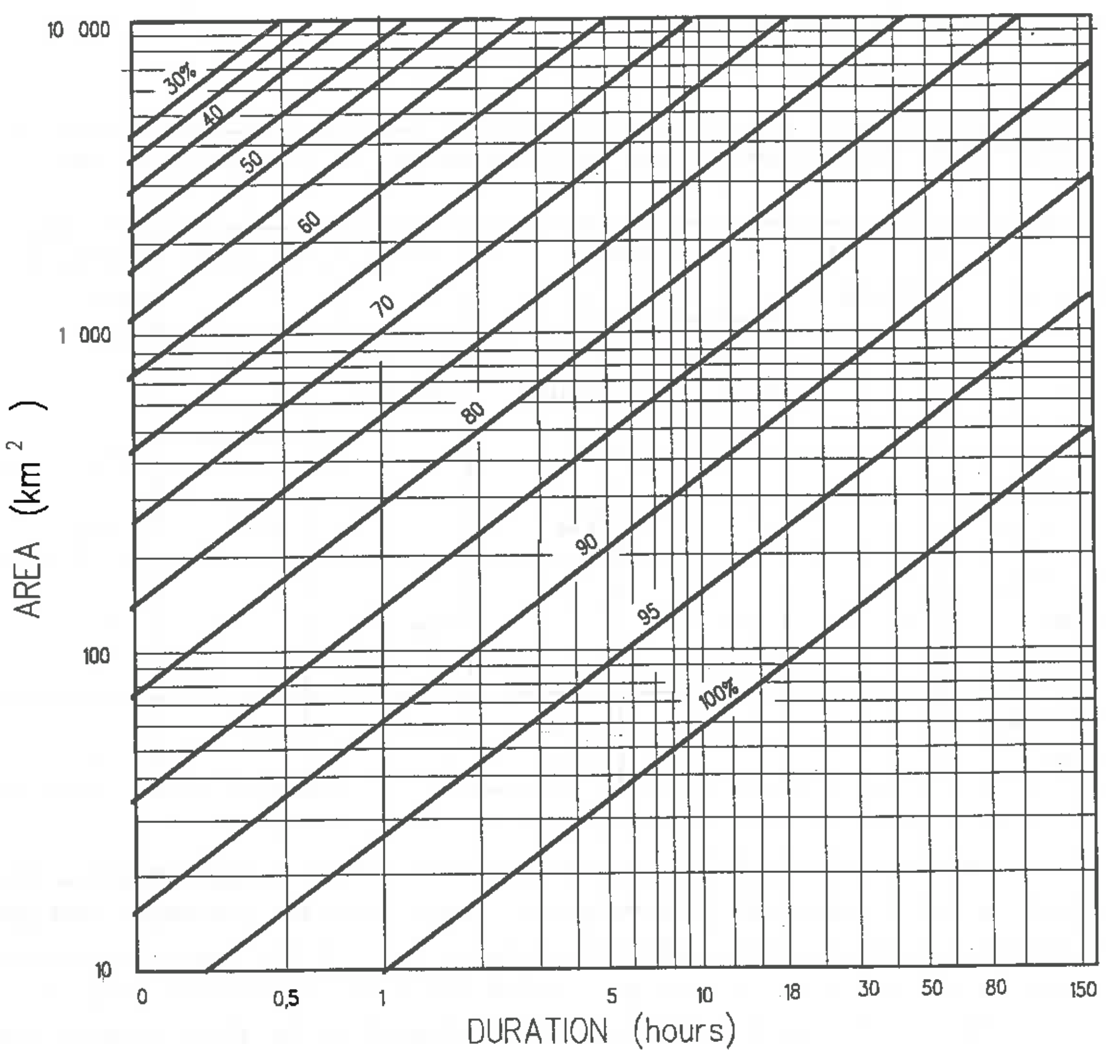

FIGURE 3.6 Recommended percentage area reduction factor for estimating average precipitation over a catchment.

TABLE 3.11 Comparison of area reduction factors:

Area km^2	t hours	ALEXANDER (Fig 3.7)	HRU 1/72 (Fig 3.4)	FSR (Fig 3.5)	US W Bur (Fig 3.6)
100	1	89	52 - 100	79	82
	3	93	52 - 100	87	92
	24	97	52 - 100	94	96
500	1	76	56	67	67
	3	82	62	81	80
	24	92	78	91	92
1 000	1	70	51	62	66
	3	78	58	76	78
	24	90	74	89	91
10 000	1	45	28	44	-
	3	62	35	61	-
	24	80	52	83	-

3.7 TIME PROFILES OF POINT RAINFALL

The intensity of storm rainfall measured at a point on the ground will vary with the rate of growth and decay of the storm as well as its rate of movement. Convective and frontal storms tend to have their peak rates near the beginning of rainfall, while cyclonic events have the peak rainfall somewhere in the central third of the storm (Eagleson, 1970). This tendency is shown in Fig 3.7 based on US Weather Bureau recommendations, and Fig 3.8 from the *Flood Studies Report*. However, there are large variations about these average intensity changes with time.

Pilgrim and Cordery (1975) examined mass curves for the highest 50 observed rainfalls in Sydney, Australia for 20-minutes and 4-hours duration. These showed a very wide scatter although the mean curves tended to have a slight S-shape. They ascribed the wide divergence to the fact that very heavy rainfalls are generally associated with highly turbulent unstable airstreams. They also pointed out that uniform mass curves were highly atypical of the variations of intensity that occur during the great majority of heavy rains, and that the assumption of almost uniform time distribution of storm rainfall is entirely unsatisfactory for design patterns, as it does not reflect the variations in intensity during the majority of heavy rains.

Van Wyk and Midgley (1966) could not detect a significant pattern in the dimensionless mass curves of numerous storms in the Pretoria area. The noticeable difference in the shapes of the HRU curves for South Africa (Fig 3.9) on the one hand and those of the USA (Fig 3.7) and the United Kingdom (Fig 3.8) is due to the different way in which the data are presented. The HRU curves represent the lower envelope boundaries of the mass curves for various durations and not the mean values shown in the other two figures. The reason for using boundary values was based on the assumption that the critical time distribution of storm input is likely to be that which exhibits an increasing intensity with increase in duration (Wiederhold 1969). The use of boundary values will affect the assumed exceedance probability but this was not investigated by the authors.

Most data, including that of the US Weather Bureau and in the *Flood Studies Report* relate the time profile of *point* precipitation for *complete storms* having durations of 15, 30, 60 minutes etc. For example it is assumed that the critical 15 minute precipitation will be generated by a 15 minute storm. This is an unlikely condition. A good example is the Port Elizabeth storm of 1st September, 1968, which had a duration of 25 hours yet generated the highest 15, 30, 45 and 60 minute rainfalls as well as the highest daily rainfall observed during the 35 years of record at the station. (See Table 3.2)

The authors of HRU 1/72 looked at this aspect and derived the diagram C5 (Fig 3.9). The method for using the diagram is given in van Wyk and Midgley (1966).

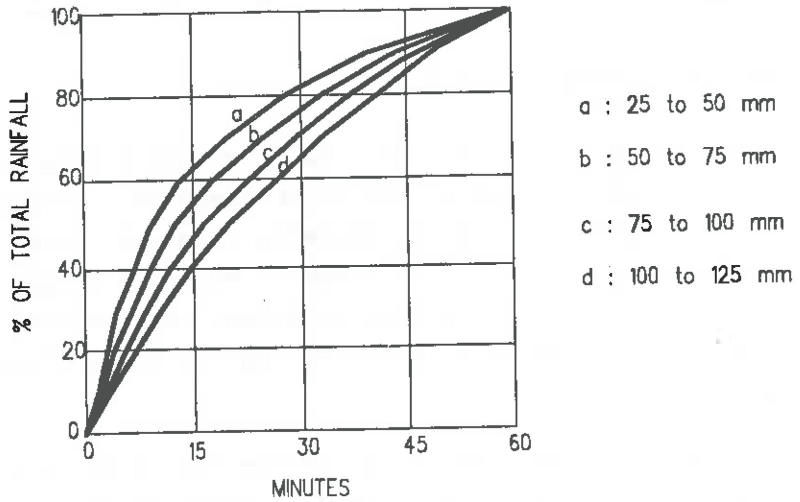

FIGURE 3.7 Typical mass curves of point precipitation for 1-hour thunderstorm type rain in the USA (US Weather Bureau)

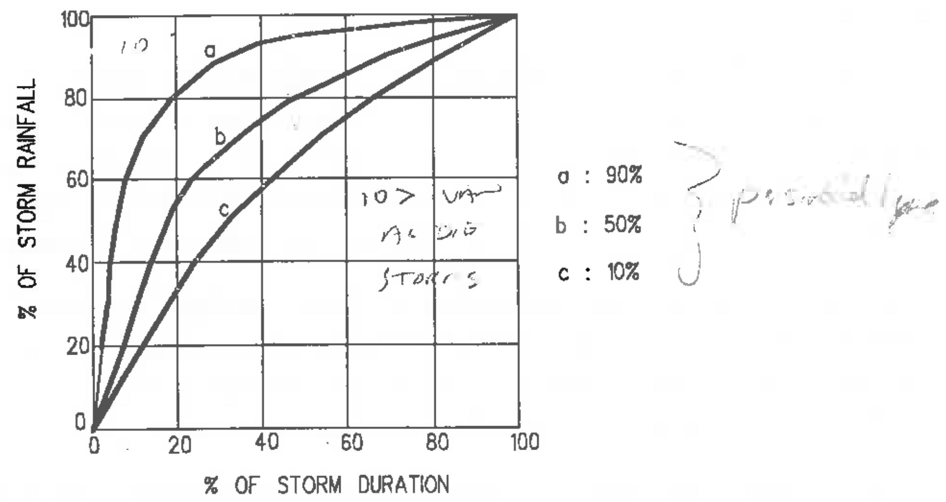

FIGURE 3.8 Summer storm profiles for the United Kingdom (after Flood Studies Report)

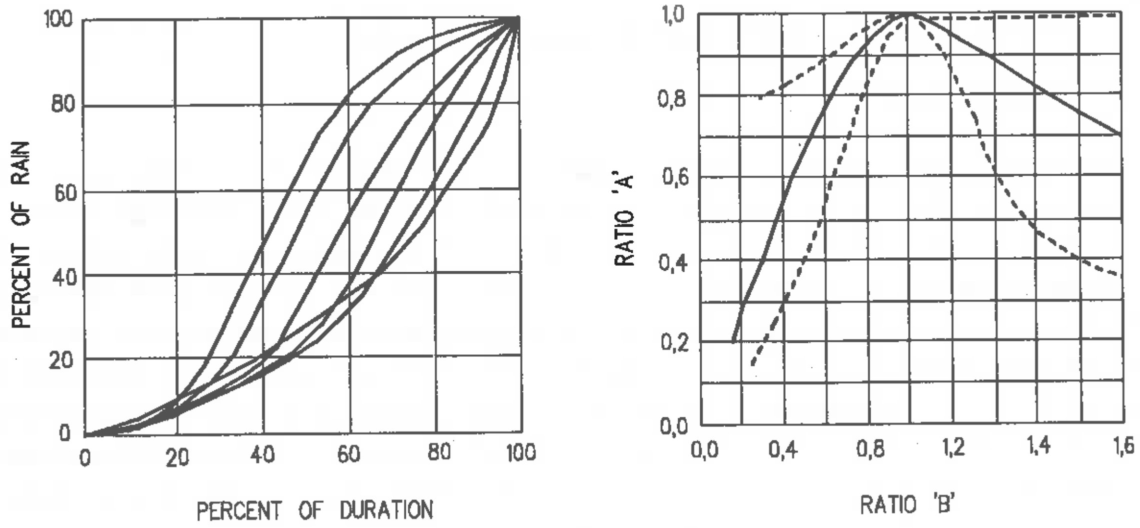

FIGURE 3.9 Time distribution of intense storms (after HRU 1/72)

3.8 STORM MOVEMENT

3.8.1 General

The final property of storm precipitation of hydrological interest is that of the speed, direction, growth and decay of storms. The time profiles of point rainfall dealt with in the previous section cannot be transposed to a larger area on the assumption that simultaneous changes are taking place over the whole catchment, as there are very steep gradients of precipitation *in both time and space* during high intensity convective precipitation.

In the case of long duration precipitation (24 hours) for large catchments (8 000 km^2) in the United Kingdom the area mass precipitation curves were found to be in close agreement with those of point storms (*Flood Studies Report*) but this is unlikely to be representative of southern African precipitation even for long duration storms.

The lack of design information on these properties associated with storm movement is due to the difficulty of obtaining the relevant data rather than its unimportance.

It was not until the advent of weather radar that meaningful studies could be made of the time-space changes in storm precipitation intensity.

3.9 STORM RAINFALL FROM RADAR OBSERVATIONS

3.9.1 Nelspruit studies

When a cloud seeding operation for hail suppression was mounted in the Nelspruit area in the summer of 1972, one of the requirements of the licensing authorities (Departments of Water Affairs and Transport) was the establishment of a sophisticated radar system. The contractor to the Lowveld Tobacco Corporation which financed the project was the Cansas International Corporation. At the request of the Department of Water Affairs, the contractor captured and stored all radar data on magnetic tape. This was subsequently processed by M.J. Dixon of the Department of Water Affairs who developed a tentative mathematical model for the estimation of the areal properties of high intensity short duration storms based on radar data collected over a period of two years (Dixon 1977). A summary of Dixon's conclusions plus some additional comment follows.

Small catchment runoff response is highly dependent on :

- the size of the storm
- its rate of growth and decay
- its speed and direction of travel
- the relationship between the storm path and the catchment orientation.

It is very difficult to determine these characteristics of convective storm rainfall for complete storms from an array of fixed point recording rain gauges, whereas the changes in radar reflectivity can be observed and recorded over a wide area (75 000 km^2), at close time intervals (approximately 16 seconds), and small area segments (less than half a square kilometre). Approximately 90 000 reflectivity values are stored with each sweep of the radar antenna.

The major problem is the determination of the relationship between radar reflectivity and precipitation intensity (the Z-i relationship). This is because reflectivity is a function of droplet numbers, a power function of droplet diameter, the moisture phase (water, ice, or graupel), and a number of meteorological variables. Dixon assumed that the Z-i relationship for high intensity convective storm rainfall would be sufficiently stable to allow him to relate reflectivity directly to rainfall intensity. The relationship he used in his calculations was

$$Z = 486 \; . \; i^{\,1{,}37} \tag{3.6}$$

Although the choice of reflectivity contour intervals was not a constraint in the data, he chose those used by the contractors for their operational requirements, viz

25 dbZ (≈ 0,7 mm/h)
35 dbZ (≈ 3,9 mm/h)
45 dbZ (≈ 21 mm/h)
and 55 dbZ (≈ 113 mm/h)

This is a logarithmic progression and must be borne in mind when interpreting the results. This range allowed him to excise reflectivity values less than 25 dbZ in that this precipitation intensity could safely be ignored for flood design purposes. Very few storm echoes exceeding 55 dbZ were observed, so by using this upper cut-off value ground clutter which has a higher reflectivity value could be eliminated.

The general instability of the air masses in which thunderstorms are generated, as well as the highly turbulent nature of the storm systems is most unlikely to lead to uniform precipitation in space or time. Storm systems are multi-cellular in composition, with a number of cells in varying states of growth and decay at any one time. Typical characteristics found by Dixon were that single cells were usually between one and ten kilometres in diameter; moved at between 30 and 50 kilometres per hour and had a typical lifetime of between 20 and 40 minutes. Storm systems containing a larger number of cells can reach 30 kilometres in diameter and can last for a number of hours. Vertical heights can range from moderate cumulus structures of up to 6 km producing light showers, to towering cumulonimbus clouds 20 km high with precipitation intensities exceeding 100 mm per hour. The upper level driving winds which steer the storms in the Nelspruit area can vary from 20 to 200 km/h, the preferred directions being from westerly, through anti-clockwise to north-easterly. Heating over high ground causes about 80 percent of the Lowveld's severe thunderstorms, occurring between 40 and 70 days per year. Frontal squall lines occur on about 10 days per year.

The general picture of convective storm rainfall is therefore one of a series of short bursts of heavy rainfall from the individual cells in the storm, but enclosed within an isohyetal pattern for the whole storm which has a zone of maximum depth and corresponding maximum areal development.

During the two-year period of observation, 198 storms were captured, of which 98 proved suitable for further analysis.

The following characteristics were derived from the data :

Storm direction. The storm track was almost never straight, and sometimes doubled back towards the starting point. For calculation purposes, the direction was taken to be that of a straight line joining the first and last points of the storm track. The mean storm direction was found to be 60° east of north with standard deviation of 56 °. The data could satisfactorily be assumed to be normally distributed. Storm direction was found to be independent of other storm characteristics.

Storm speed. The calculated speed was taken to be the straight line distance between the first and last points on the storm track divided by the elapsed time. The mean speed was 22 km/h with a standard deviation of 11,5 km/h and lower limit of zero. The speed could also be satisfactorily approximated by a normal distribution, and like storm direction, it was independent of the other storm characteristics.

The remaining variables of interest were the area development of the storm with time, for the three selected reflectivity intervals :-

25 - 35 dbZ (0,7	- 3,9	mm/h)
35 - 45 dbZ (3,9	- 21	mm/h)
45 - 55 dbZ (21	-113	mm/h)

The *areal development of a single storm with time* could be approximated by a Gaussian curve. The *mean areas of the three reflectivity intervals* were approximately normally distributed while the *index of storm duration* (standard deviation of the area-time curves) was found to follow a three parameter Kappa distribution. The parameters for the three reflectivity intervals were not only interrelated, but also correlated with storm area parameters.

Given this information, it became possible to generate a large number of synthetic storms by using the following procedure for each storm :-

1. Storm speed was generated to follow a normal distribution, independent of other characteristics.

2. Direction was generated to follow a normal distribution, independent of other characteristics.

3. The duration of 35-45 dbZ storm intensity values was generated to follow a Kappa distribution.

4. The duration of 25-35 and 45-55 dbZ storm intensity values were computed from the regression on 35-45 dbZ duration plus a random component.

5. The maximum area of the 35-45 dbZ intensity value was generated to follow a normal distribution but held within limits posed by the regression on the duration of the 35-45 dbZ value calculated in step 4.

6. The maximum area of the 25-35 and 45-55 dbZ storm intensity values were computed by regression on the 35-45 dbZ area plus a random component.

All of the above properties were related to a moving storm, with the result that storm duration was the length of time during which the storm was active, whereas the hydrologist requires these relationships for an area on the ground. Dixon generated 530 storms and derived ground relationships for duration, maximum point depth and mean area depths of average intensities within the contour intervals :-

25 - 35 dbZ	= 2	mm/h
35 - 45 dbZ	= 10	mm/h
45 - 55 dbZ	= 55	mm/h

A plot of the area reduction factor versus storm area is shown in Fig 3.10 and intensity-duration-frequency curves in Fig 3.11.

Dixon also found some confirmation of Bell's DDF relationships although his synthetic data base was too short for an adequate test.

3.9.2 Further development of a storm precipitation model

Dixon's work has been quoted in some detail because it paves the way for the development of a new methodology for the study of DADF relationships of high intensity storm precipitation. This is an important subject for further research.

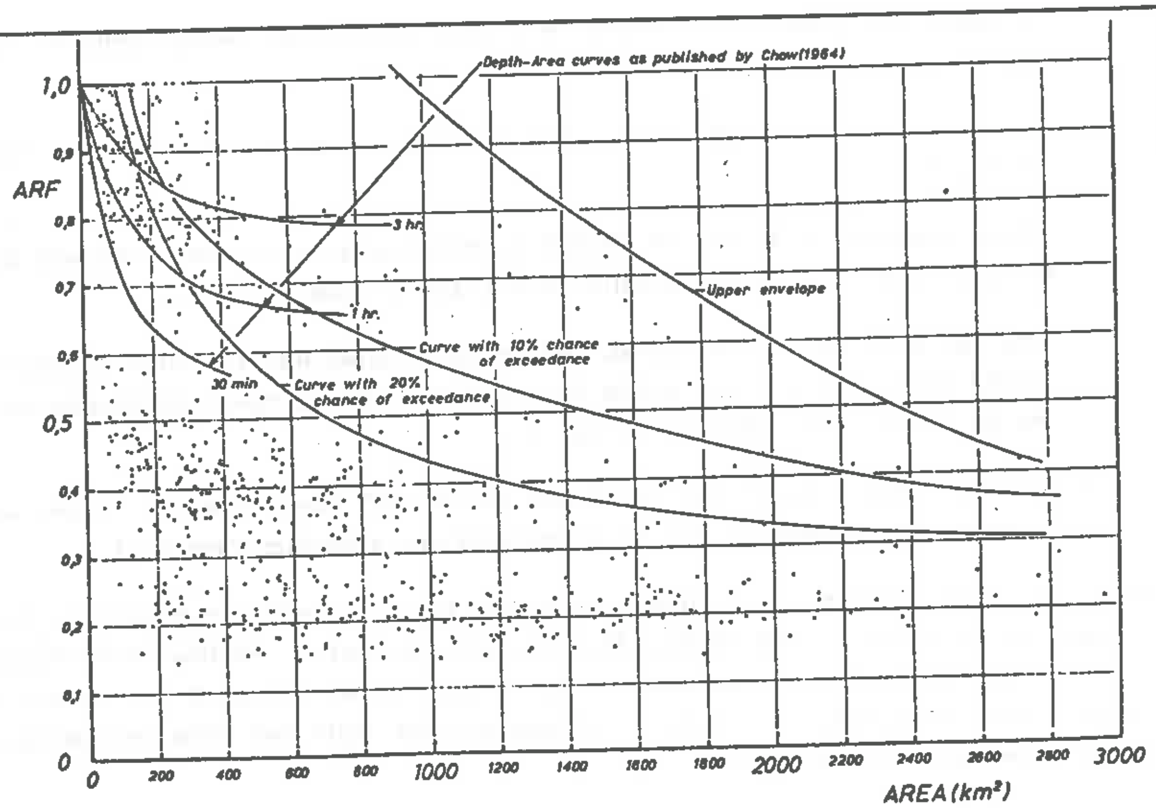

FIGURE 3.10 Area reduction factor vs storm area based on 530 synthetically generated convective storms (after Dixon 1977).

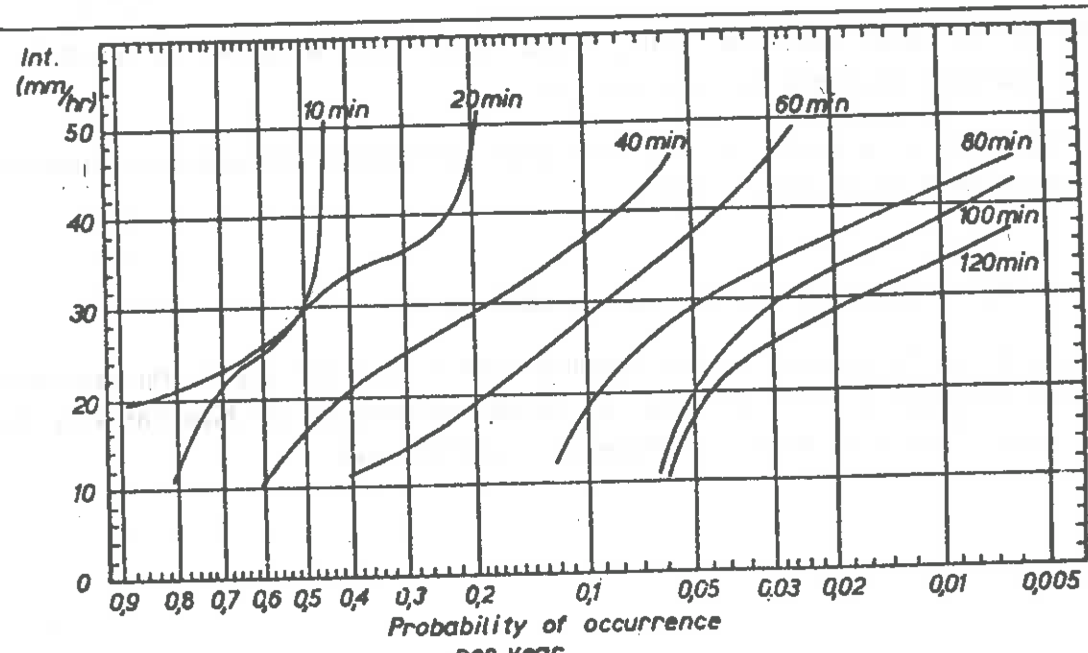

FIGURE 3.11 Intensity vs annual exceedance probability for various durations based on 530 synthetically generated convective storms (after Dixon 1977).

3.10 ARTIFICIAL STIMULATION OF PRECIPITATION

3.10.1 General

The effects of deliberate or inadvertent man-induced changes in the DADF relationships through changes in cloud behaviour cannot be ignored although they cannot as yet be quantified. A cloud seeding project has been in operation in the Nelspruit area since 1972 and a research project in the Vaal River catchment in the Bethlehem area since 1976. If these prove to be successful widespread weather modification activities can be expected.

Large urban areas create *heat island* effects and also increase the concentration of precipitation nuclei in the air.

So far very little information is available on the possible range of changes in precipitation properties of interest to hydrologists due to weather modification processes. Simpson *et al* (1971) stated that seeded clouds rained more than the controls because they were bigger, longer lasting clouds not because their rainfall rates were significantly higher. In the 1970 operations in Florida, USA seeded clouds averaged 19% taller, 75% in area and their echoes lived 56% longer than control clouds. Subsequently Woodley *et al* (1977) reported on more detailed studies at the Florida area cumulus experiment where the effect of cloud seeding on precipitation intensity was examined in some detail. The data for 1973 suggested a substantial mean *increase* in rain intensity associated with seeding but the data for 1975 indicated a modest mean *decrease* in rain intensity associated with seeding. For both years combined the mean result was a 7 percent increase in intensity but the results were still inconclusive. Alteration of cloud size and duration accounted for most of the volumetric increase in output from the seeded clouds.

Meteorologists involved in rainfall stimulation projects have noted that very high intensity precipitation is produced when separate storms merge into a single system. Alexander (1975) reported on a well-documented cloud merger that occurred in the Nelspruit area which produced 101 mm of precipitation in 16 minutes, which may well be the highest intensity yet recorded in South Africa. (The highest 15 minute precipitation recorded at one of the Weather Bureau's autographic recorders up to that time was 56,0 mm at Towoomba in the Transvaal).

Woodley *et al* (1977) describe the cloud merger process. In the Florida experiment cloud mergers have produced an order of magnitude more precipitation than isolated clouds on the same day. In one instance the total rainfall from the merged system was 10,8 million cubic metres compared with 0,3 million cubic metres for the random control clouds. It would have taken 36 isolated clouds to equal the precipitation of the merged system!

The possibility of using cloud seeding to induce cloud mergers is still being investigated.

Urban stormwater drainage systems are very vulnerable to high-intensity, short-duration storms. The possibility that urban and industrial development may enhance the mechanisms that trigger cloud mergers should not be overlooked.

The Metropolitan Meteorological Experiment METROMEX was undertaken in the St Louis area of the USA to investigate the effect of urban industrial development on meteorological conditions including changes in rainfall. Results from the first five years of observation showed significant downwind increases in precipitation. Huff and Vogel (1977) found an urban-induced increase in precipitation of approximately 30% in an area located about 35 km downwind of the centre of the city. The urban rainfall enhancement was found to be most pronounced in heavy rainstorms. The frequency of the storms was approximately twice the network average. Weather radar observations showed that light to moderate storms intensified as they crossed the urban area and this intensification culminated in more rainfall a few kilometres downwind. This was apparently produced by the formation of new cells over the urban area which reinforced the storm system, and by intensified input from the urban environment which caused existing cells to grow and hence increase their water yield. In addition, the birth of new cells favoured the merging of cells downwind of the city resulting in storm intensification.

Analyses of heavy storm rain cells showed that those exposed by the urban environment produced about 70% more rain volume on average than those produced in the adjacent rural area.

Schickendanz (1977) summarized information from eight major cities in the USA :-

TABLE 3.12 Observed effects of urban areas on storm rainfall (after Schickendanz, 1977).

City	Observed effect	Percentage change	Approximate location
St Louis	Increase	15	16 - 19 km downwind
Chicago	Increase	17	48 - 56 km downwind
Indianapolis	Indeterminate	-	-
Cleveland	Increase	15	40 - 80 km downwind
Washington	Increase	9	Urban area
Baltimore	Increase	15	Urban and north eastward
Houston	Increase	17	Near urban centre
New Orleans	Increase	10	Northeast side of city
Tulsa	None	-	-

Six cities were found to have thunder-day increases ranging from 13% to 47% above the climatic background. The percent increases in hail-days were much larger than thunder-days with hail increase generally occurring in the region 24 - 40 km from the city.

3.11 PROBABLE MAXIMUM PRECIPITATION (PMP)

3.11.1 General

The probable maximum flood (PMF) is the only method that was specifically developed for the determination of upper limit floods for dam design but it has been at the centre of controversy ever since its introduction fifty years ago.

The method was developed in the 1930's in response to public pressure following severe loss of life due to dam failures in the USA. The public demanded that the design of the dams should be such that there should be no risk of failure, much the same as the present public protests against the construction of nuclear power stations where the designers are unable to provide such a guarantee.

The intention was clear in the original name of the method which was the *maximum possible flood*.

As it was not possible to derive an upper limit to possible floods from hydrological data, it was assumed that there must be upper limits to the precipitable moisture content of a column of air, and the rate of influx of precipitable moisture over the catchment, and consequently an upper limit to the depth and rate of precipitation over a catchment.

Because this upper limit could not be determined accurately, the term *probable maximum precipitation* (PMP) was introduced in 1950, the word '*probable*' being used to demonstrate the uncertainty relating to its determination. The protagonists continued to maintain that a maximum value existed but could not be determined accurately. However the word *probable* has a very definite connotation in hydrological statistics, and the opposition to the concept thus became one of the semantics as well as of philosophy.

It may seem logical to assume that there must be some upper limit to the amount of precipitation that can fall over a given area in a given time, but many hydrologists maintain that the concept of some definite physical limit which could occur but could not be exceeded is untenable - for example " ... the idea of an upper limit to any natural phenomenon is incompatible with the present day framework of scientific thought." (Benson 1964), and " ... the thickness of an air mass that produces the maximum precipitation can always be one metre greater than was assumed ..." (Yevjevich 1968).

The debate on the concept of a probable maximum flood continues and will be discussed again in Chapters 9 and 12. Details of methods used to determine the probable maximum precipitation are described below.

From a more pragmatic point of view, meteorologists (understandably) find it impossible to determine absolute upper limits to factors such as the moisture content in a vertical air column which is in turn a function of temperature, degree of saturation and barometric pressure; or the absolute maximum rates of influx and cooling of these air masses either on synoptic or local thunderstorm scales. The procedure now used is to attempt to derive maxima from observed data rather than from theoretical considerations.

The PMP calculation methods proposed in available publications do not and cannot give a *no risk* value of precipitation, but may give some indication of the *very low risk* precipitation depths. Users of proposed methods should study all the assumptions used in the development of the method being considered and draw their own conclusions regarding the uncertainties involved.

3.11.2 World Meteorological Organization manual

In 1973 the World Meteorological Organization (WMO) published its manual for the estimation of probable maximum precipitation (World Meteorological Organization, 1973). A revised edition was published in 1986. The manual describes the more common meteorological approaches for estimating PMP and has become the internationally accepted basis for the estimation of the PMP in orographic and non-orographic regions with and without adequate data.

The 1986 edition defines PMP as:

> *"... theoretically the greatest depth of precipitation for a given duration that is physically possible over a given storm area at a particular geographical location at a certain time of year."*

The description continues :

> *"Such is the conceptional definition of PMP. This definition is a description of the upper limit of precipitation potential that is storm centred, ie, related to the centre of the precipitation pattern of the storm irrespective of the configuration of the boundaries of a particular basin. This leads to a second important concept. The storm centred PMP values cannot be applied directly to a drainage, but must be modified to develop a drainage-averaged PMP estimate. This is defined as the average PMP depth over the drainage after the storm centred PMP value has been distributed across the drainage in accordance with the PMP storm pattern and appropriate computational procedures. ---- The values derived as PMP under these definitions are subject to change as knowledge of the physics of atmospheric processes increases.*
>
> *"There is no objective way of assessing the accuracy of the magnitude of PMP estimates derived by the procedures described here or by any other known procedures. Judgment of meteorologists, based on meteorological principles and storm experience, is most important."*

Current knowledge of storm mechanisms and their precipitation-producing efficiency is inadequate to permit precise evaluation of limiting values of extreme precipitation. PMP estimates must be considered therefore, at least for the present, as approximations. The accuracy, or reliability, of an estimate depends basically on the amount and quality of data available for applying various estimating procedures".

The underlying assumption is therefore that an upper limit exists, ie that there is an upper bound to storm precipitation.

The procedure described in the manual consists of precipitable water maximization; maximization of observed storms in place; wind maximization; and the transfer of the maximized storms from the locations where they occurred to the catchment of interest. A number of adjustments have to be made to the maximized in-situ storm in the storm transposition process to accommodate synoptic conditions, the regional influence of storm type, and topographic and elevation controls.

The manual contains a number of cautions to the users. These include the following:-

"The manual was written under the assumption that the user would be a meteorologist".

"Different, but equally valid, approaches may yield different estimates of PMP."

"There is no objective way of assessing the general level of PMP estimates derived by the procedures described here or by any other known procedures."

"Judgment based on meteorology and experience is most important."

"--- the importance of meteorological studies in preparing PMP estimates cannot be over-emphasized."

"--- the various meteorological procedures described here are considered most applicable to basins up to about 50 000 km^2, although they have been used for *much larger basins."* (The area of the Vaal Dam's catchment is 38 000 km^2).

3.11.3 Other overseas methods.

Hansen (1987) provides a good overview of the state of the art for PMP estimation.

Cudworth (1987) describes the policy of the US Bureau of Reclamation. Until recently the Bureau developed estimates of the PMP on a site specific basis based on a separate analysis that included maximization and transposition of historical storms to the basin in question and enveloping the appropriate depth-duration data. The Bureau has since adopted regionalized approaches which provide improved regional consistency, better definition of transportation limits for orographic areas and the use of state of the art models for areas influenced by orography. Since 1980 the Bureau's hydrometeorologists have undertaken joint studies with the personnel of the National Weather Service to develop new regional criteria.

The Australian guidelines (Pilgrim, 1987) provide general information only and refer users to Australian publications. The guidelines recommend that PMP estimation should only be undertaken by specialist hydrometeorologists experienced in PMP studies.

3.11.4 Situation in southern Africa

In most overseas countries meteorologists have been actively involved in the determination of methods for estimating the PMP from which the PMF can be determined. This has not been the case in South Africa where South African meteorologists have not concerned themselves with PMP estimation and the South African Weather Bureau has not published any information on this subject.

The only established PMP estimation procedure developed for southern Africa is that given in Report 1/72 of the University of the Witwatersrand's Hydrological Research Unit (HRU 1/72). The authors of HRU 1/72 produced envelope curves for regions experiencing similar extreme point rainfalls for South Africa as a whole, and for world record rainfalls. They also produced depth-area-duration-frequency relationships and probable maximum precipitation *vs* area curves for each of these regions based on the maximisation of observed storms according to the ratio of maximum to actual dew point at the time of the storm. Although these estimates are equated to PMP, they are not true probable maximum precipitation estimates in the generally accepted sense. These are upper envelope values of observed rainfall maxima in South Africa and are used in conjunction with the unit hydrograph method to estimate the probable maximum flood. This simple but not necessarily inferior procedure involves reading off PMP values either from an envelope curve for South Africa as a whole, or for curves presented from a number of regions. The isohyets of recorded severe storms in each region up to the time of publication (1969) are detailed and can be transposed to the catchment of interest.

As with the case in overseas methods the HRU 1/72 method assumes that the PMP can be included in standard storm rainfall-runoff models to produce the PMF. The validity of this assumption is discussed in Chapters 9 and 12.

3.11.5 Alternative approaches to maximum precipitation estimates

One alternative approach that has been suggested is to estimate the 0,01% risk (return period of 10 000 years), or alternatively to calculate the mean plus six or more standard deviations and use these as upper limit values. For these statistically based estimates use should be made of regional data rather than data from one or two stations.

A second alternative approach is to plot observed depths of extreme precipitation against either catchment area or storm duration, and draw upper envelope curves encompassing the observations.

3.12 PROPERTIES OF HISTORICAL STORMS

3.12.1 Applicability

Most of the relationships described so far in this chapter are based on large numbers of observations and of necessity these represent averaged characteristics, which are then amenable to statistical analysis and extrapolation.

The question that must be addressed is whether or not relationships developed from country-wide averaged relationships can be universally applied over the sub-continent. This question will be raised again later in this handbook. At this stage it should be noted that no depth-area-duration-frequency relationships have yet been developed for the different storm types experienced in southern Africa, nor have multiple storm events been sufficiently examined.

To illustrate this point, the properties of several different types of severe storms that have been observed in South Africa are described in the annexures to this chapter. While these cannot be used to test the *accuracy* of the relationships developed earlier in this chapter, they can be used to test the *applicability* of these relationships to the specific events.

All the storms were severe and cannot be considered as being typical, or as having 'average' characteristics. The calculated return periods of the storms were in excess of 100 years, but the realism of the calculations is subject to doubt. The storms have been ranked from smallest to largest in appendices 3A to 3G, and described briefly as follows:

3A The Nelspruit storm produced 101 mm of rain in 16 minutes over an area of about 50 km^2. This is the most intense precipitation yet documented in South Africa. Extensive hail damage was caused but no floods were generated because of the small area over which the storm occurred.

3B A maximum of 520 mm was recorded during the Port Elizabeth storm, most of it falling uniformly over a four hour period. Similar although less severe storms occur frequently along the southern and eastern coastline and adjacent interior.

3C The depth-duration characteristics of the Pretoria storm were similar to those of the Port Elizabeth storm. These storm types are frequently encountered.

3D The tropical cyclone Domonia was a rare, intense storm. Tropical cyclones are limited to the north eastern coastal areas and adjacent interior. The duration of the storm rainfall was three days but the severe rainfall at any fixed site occurred over a period of less than 12 hours.

3E This storm over the south western Cape also occurred over a three day period and the rainfall was more evenly distributed over the three days than the Domoina storm. The storm produced floods with peaks having return periods well in excess of those of the storm rainfall.

3F With increasing catchment size, major floods are more likely to be produced by a succession of closely spaced storms than by a single event. This is illustrated by the storms which fell over the Vaal Dam catchment in February, 1975.

3G Prolonged seasonal rains are necessary for the generation of severe floods over large catchments in the semiarid interior of southern Africa. This is because of the large antecedant precipitation depths required to raise the soil moisture status to the point where significant proportion of subsequent storm rainfall is available for generating runoff. The 1987-88 summer season rainfalls are an example.

3.12.2 Intensity-duration relationships for extreme storms

Fig. 3.14 shows the intensity-duration relationship for the Nelspruit, Port Elizabeth, Pretoria and Domoina storms described in the appendices. The ideal would be to build up a library of flood producing storms such as these, categorize them according to the meteorological conditions which produced them, and develop full depth-area-duration-frequency relationships for each storm category in each region in southern Africa.

In time it might well be possible to develop these relationships, but until then users will have to rely on the largely empirical relationships based on overseas experience modified in the light of the scarce local data.

Extreme events such as those in the appendices below do give information on historically experienced maxima which is useful even if it can not be coupled with probabilities of occurrence.

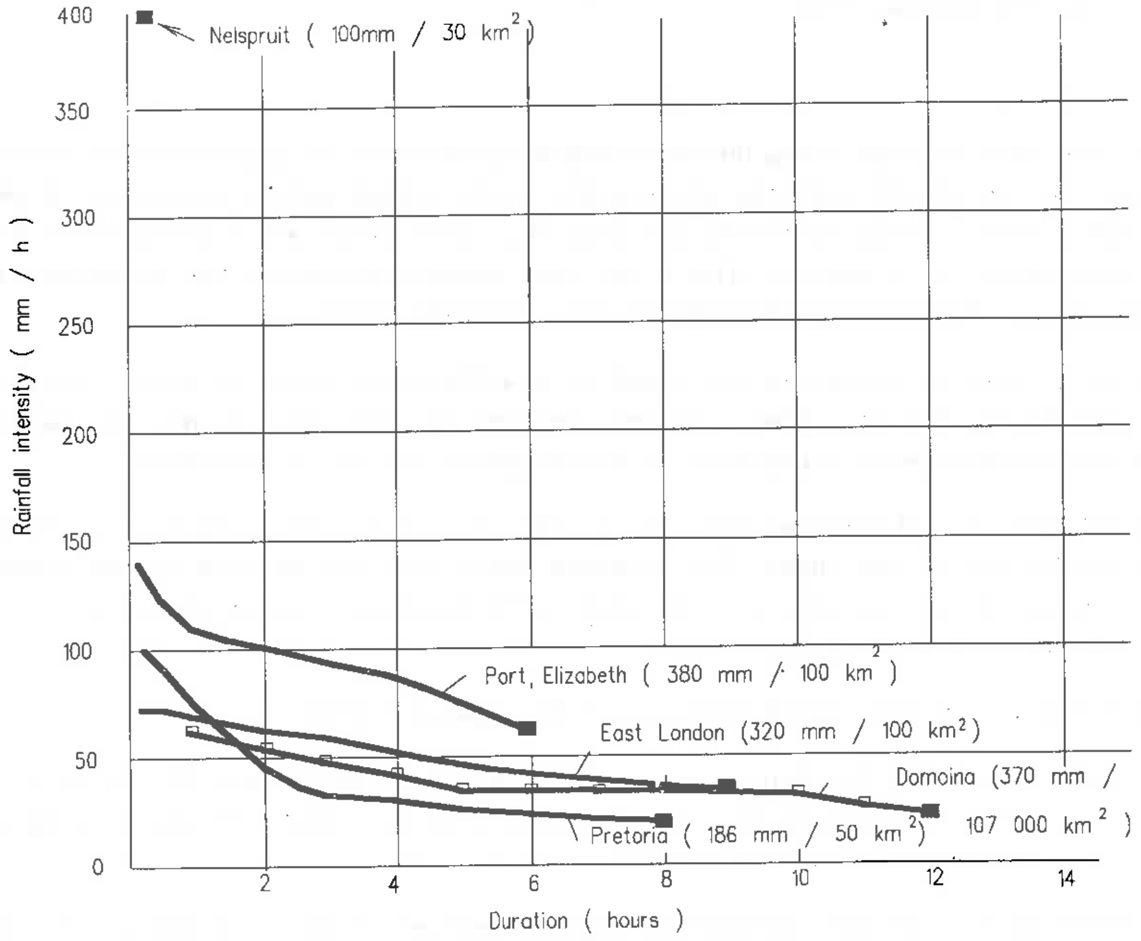

FIGURE 3.12 Rainfall intensity-duration relationship for the extreme storms described in the appendices.

3.13 OTHER SOURCES OF USEFUL INFORMATION

The monthly newsletter issued by the South African Weather Bureau provides a valuable source of information on monthly weather in southern Africa. The series of technical reports on surveys conducted after severe floods by the Flood Studies section of the Department of Water Affairs provides excellent information on all aspects of these floods. The technical report TR102 (Adamson 1980) also has information on severe historical floods up to 1980, as well as another perspective on the subjects covered in this chapter.

3.14 APPENDIX 3A : NELSPRUIT STORM

30 NOVEMBER, 1975

The radar data captured during the cloud seeding operations at Nelspruit provided valuable insight into the DADF properties of convective storm rainfall (section 3.9 above). It also provided the basis for an interesting case study of a cloud merger which produced 101 mm of precipitation in 16 minutes. This is the most intense precipitation yet documented in South Africa. The following information is from Alexander (1975).

Fig 3A.1, shows the location of the operations as well as areas where the rainfall exceeded 20 mm during the day. Many observers reported no rain. This is not unusual for convective rainfall. What is unusual is the intense rainfall just east of White River.

Fig 3A shows pairs of horizontal and vertical radar scans taken at 15 minute intervals during the development of the storm. The horizontal scans show the direction of the vertical sections, and the vertical scans show the planes of the horizontal scans which are taken with the radar set to 3^{0} above the horizontal.

The following is an abbreviated description of the sequence of events.

14:30 Complex III (the third in the afternoon) developed rapidly some 100 km north of Nelspruit. The Lear Jet which had been observing complex II west of Nelspruit was despatched to the new threat.

14:46 On the way the pilot reported vigorous new growth half way between the two complexes and immediately started seeding the new formation which at that time was not yet visible on radar. The vertical section taken 8 minutes later shows the early stages of the growth of the new complex.

15:00 Seeding of the new complex (complex IV) which was now visible on radar continued, and a second seeding aircraft was despatched to the area. There were now signs of rapid cloud development along the 100 km long NE-SW orientated front. By 15h07 the Lear Jet had completed its 14th seeding run, having released 104 silver iodide flares in 109 minutes. Then the starboard engine ingested ice which had formed on the engine nacelle during the cloud penetrations and the jet had to return to base on one engine.

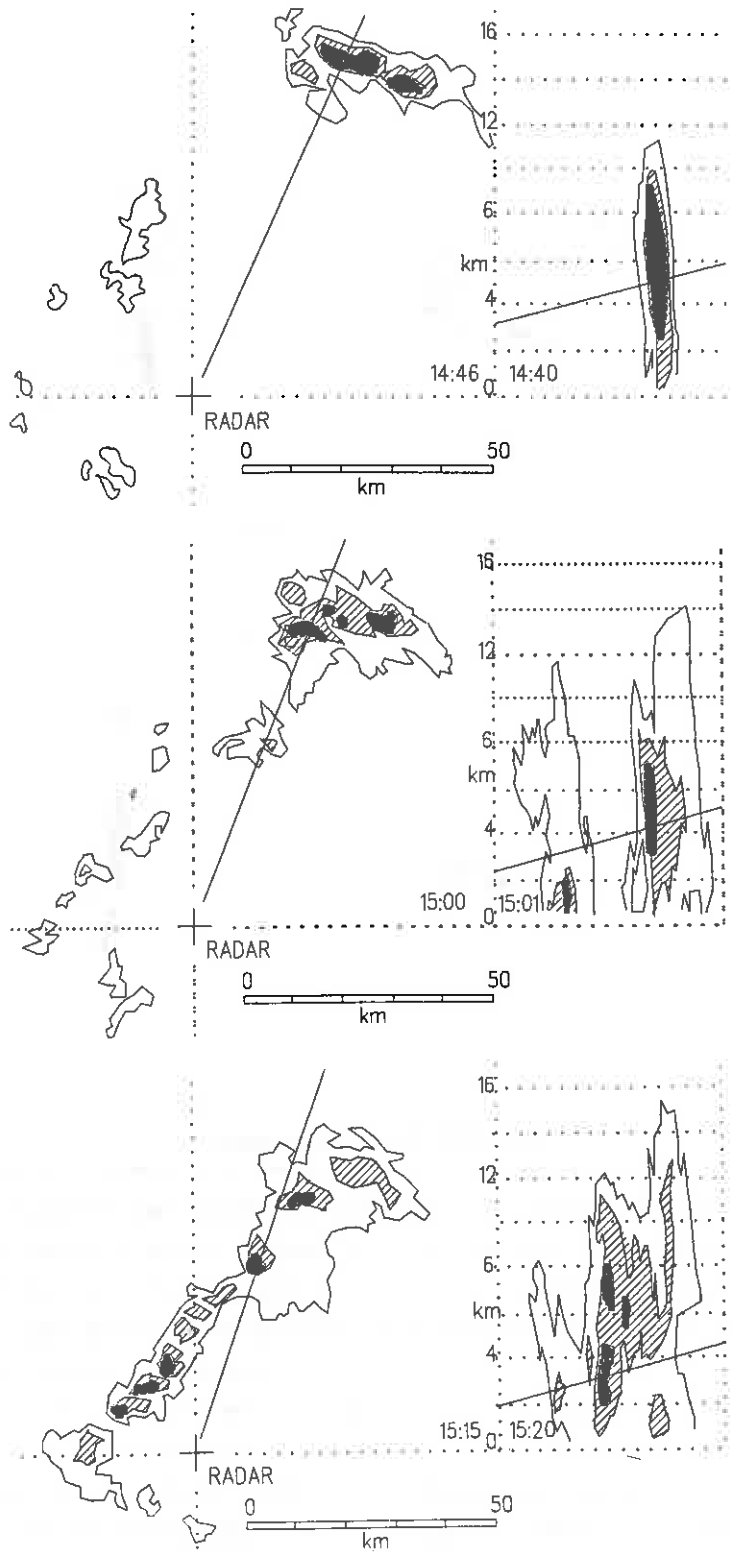

FIGURE 3A.1 **Radar images of the development of the Nelspruit storm of 30 November, 1975. (continued overleaf)**

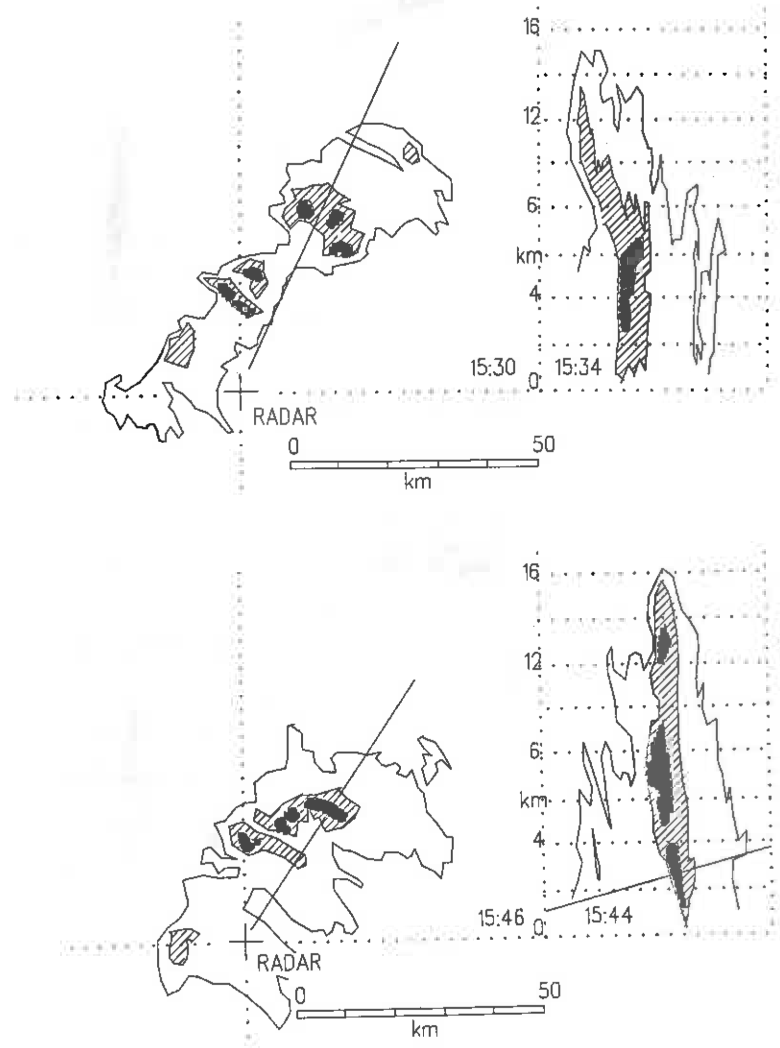

FIGURE 3A.1 (continued)

15:15 In less than 30 minutes a solid front of cloud had developed with numerous 45 dbZ (reflectivity units measured in decibels which is a logarithmic scale) areas showing on the horizontal scan. The second aircraft commenced seeding in the same area as the first aircraft at an altitude of 4 500 m and continued seeding along the front as it gained altitude. The vertical scan clearly shows the merging of complexes III and IV, with a single massive central 35 dbZ core with 45 dbZ inclusions up to 10 km and cloud tops to 15 km. Simpson et al (1971) used cloud seeding to encourage the formation of cloud mergers with their corresponding large increase in rainfall compared with single cloud systems (see section 3.10 above). In the Nelspruit storm the rapid cloud growth along a long front and the south easterly movement of complex III prior to seeding complex IV indicate that merging may well have taken place even had there been no seeding, but seeding may have accelerated and magnified the growth.

15:30 The general intensification and shrinking of the radar reflectivities is evident in both the horizontal and vertical sections. There were numerous reports of hail at this time.

15:46 The cloud complex had shrunk further with a single area of high reflectivities. An observer at "The Fountains" which is in the middle of the complex reported that 101 mm of rain fell during the 16 minute period from 15:43 to 16:11. Two other observers reported 58 mm and 64 mm of rain.

16:00 -16:30 The storm waned as it continued its movement but the activity rejuvenated in the Barberton area where several reports of more than 20 mm of rain were received. By 17:18 the system began dissipating.

An examination of the radar data showed that the storm centre moved southward at 21 km/h. The cell diameter was 6 km, and the time taken for the passage of the cell was therefore 17,1 minutes based on radar data which compares well with the 16 minutes recorded by the observer.

Weather Bureau report WB36 (Weather Bureau, 1974) gives the following information derived from the recording rain gauge record at station 555/837 at Nelspruit for the period 1960 - 1972 :-

TABLE 3A.1 Observed and expected rainfall maxima

Duration	Observed maxima	Expected maxima for 100y return period
15 min	37	42
30 min	53	63
60 min	65	74

An extrapolation of the Weather Bureau tables indicates a return period exceeding 10 000 years for 100 mm of rainfall in 15 minutes at Nelspruit! This indicates that statistically cloud mergers cannot be considered to be part of the family of convective rainfall events that feature in the annual maxima series.

Comment

The possibility that rainfall intensities of this magnitude may occur over urban areas, particularly where the possibility of inadvertent cloud seeding is present (section 3.10 above) should not be ignored.

3.15 APPENDIX 3B : PORT ELIZABETH STORM

1 SEPTEMBER, 1968

Details of this storm are provided in the Weather Bureau newsletter No 234 of January, 1968 (Hayward and v.d. Berg 1968), and the statistical analyses of the recording rain gauge record at the airport in the Weather Bureau publication WB 36.

Intense rain averaging a sustained 20 to 30 mm per 15 minutes fell over the city of Port Elizabeth during a four hour period from about 08:00 on Sunday 1st September, 1968. Severe damage was caused and nine lives were lost, eight of them by drowning. The intense rainfall was associated with the passage of an upper air cut-off low and high vertical air velocities aggravated by the topography of the city as well as other factors.

A total of 429 mm of rain was recorded on the recording rain gauge at the airport, of which 417 mm fell between 08:00 and 12:00, with a maximum of 552 mm further to the north.

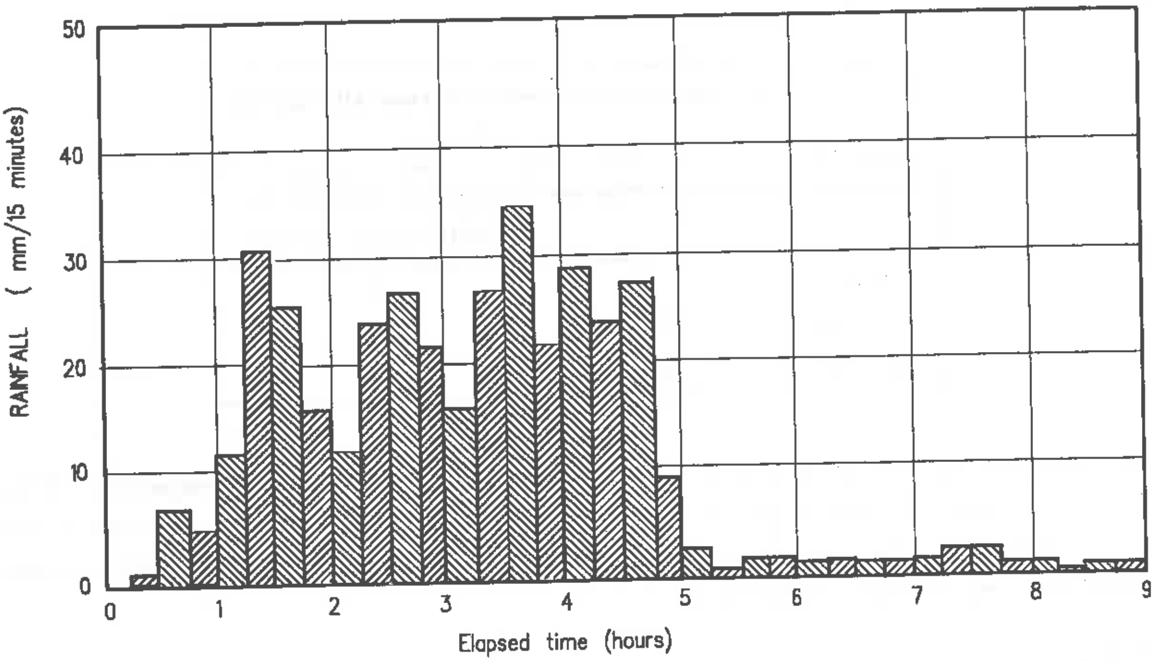

FIGURE 3B.1 Storm rainfall in 15-minute intervals recorded at the Port Elizabetn airport.

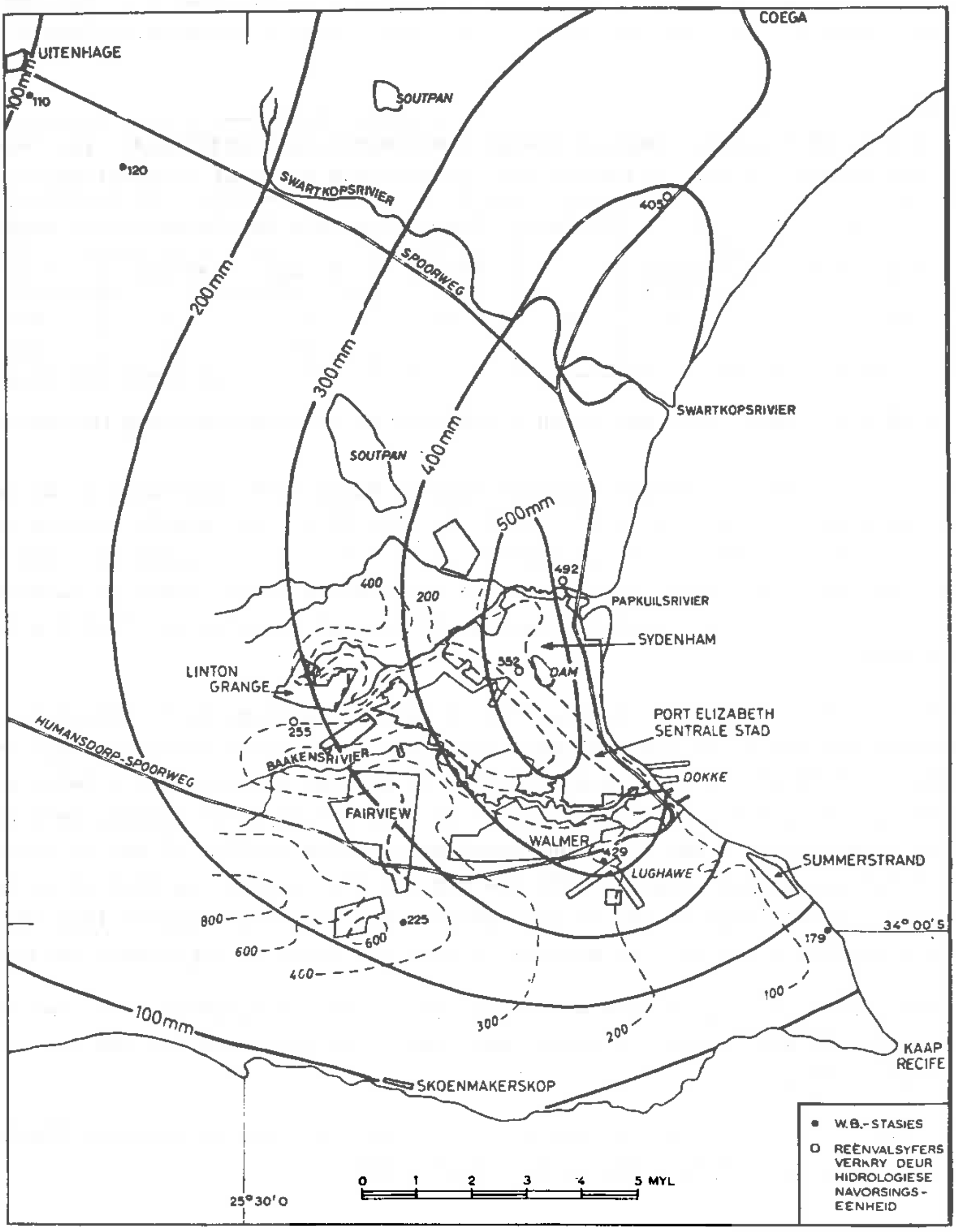

FIGURE 3B.2 Isohyets for the Port Elizabeth storm of 1st September, 1968. Dotted lines indicate contours in feet. (From Hayward and v.d. Berg, 1968)

The 15, 30, 45 and 60 minute rainfalls associated with this storm were the highest on record at this station for the period of record from 1938 to 1972. (See table 3.2). The expected maxima for a 100 year return period derived by various methods are shown in Table 3B.1:

TABLE 3B.1 Extreme values of rainfall in millimeters for Port Elizabeth: (a) from WB 36 (based on data for each month), (b) from WB 36 (based on annual maxima)

Highest on record			Method	Expected maxima for 100 year return period			
15 min	30 min	60 min		15 min	30 min	60 min	1 day
35	61	112	(a) (b)	16 33	22 47	31 69	103 179

The table of extreme values for rainfall in WB36 has the following interesting footnote :-

"For Port Elizabeth the expected maximum rainfalls based on the distribution of the annual maximum values are given as well". (line (b) in Table 3B.1). *"The monthly expected values are too small compared with the monthly recorded because an exceptional single high maximum rainfall occurred in almost every month with a return period of hundreds to thousands of years. The second maximum rainfall was approximately one third of the first maximum"*.

However, Hayward and van den Berg (1968) in their discussion on the Port Elizabeth storm comment that heavy falls over the coastal and adjacent inland areas of southern and eastern Cape and Natal are not at all unusual and can be expected to occur once or twice a year somewhere along this coast, although not of the same severity as the Port Elizabeth storm. They quote rainfalls of 360 mm in the Outeniqua mountains in 1932, 312 mm in Durban in 1953, 265 mm in East London in 1953, 459 mm near Port Shepstone in 1959. Other heavy one-day falls were those of 590 mm at Eshowe in 1940, 475 mm at Kaapsche Hoop and 460 mm at Mariepskop both in 1939, with the Eshowe figure being the largest up to that time.

Further falls exceeding 250 mm were experienced in Port Elizabeth in 1981. This storm produced 372 mm of rain at Loerie Dam which was overtopped but did not breach (du Plessis, 1984).

The East London maximum was exceeded in 1970 when 447 mm was recorded. The South African maximum at Eshowe has also since been exceeded.

Comment

There are several points of interest.

The time distribution in Fig. 3B.1 shows a fairly even spread of rainfall over the 4-hour period. It is obvious that for any catchments with response times less than about three hours, the rain falling towards the end of the storm would have been most critical as the catchments would have been saturated, streams would have been in flood, and the drainage systems would have been full while the storm continued to produce rain at the sustained intensity.

The precipitation pattern and possibly the intensity as well, were influenced by the local topography (Fig 3B.2)

3.16 APPENDIX 3C : PRETORIA STORM

27-28 JANUARY, 1978

This storm and the resulting floods are documented in the Department of Water Affairs technical report TR88 (Kovács 1978) and the Weather Bureau's newsletter No 346 of January, 1978. This information is summarized below.

Fig 3C.1 shows the time distribution of the rainfall in clock quarter hours at the University of Pretoria Experimental Farm where 187 mm was recorded, and the estimated corresponding rainfall distribution at Bryntirion. Fig 3C.2 shows the isohyetal pattern of the rainfall with a maximum of 280 mm recorded at Bryntirion near the Union Buildings.

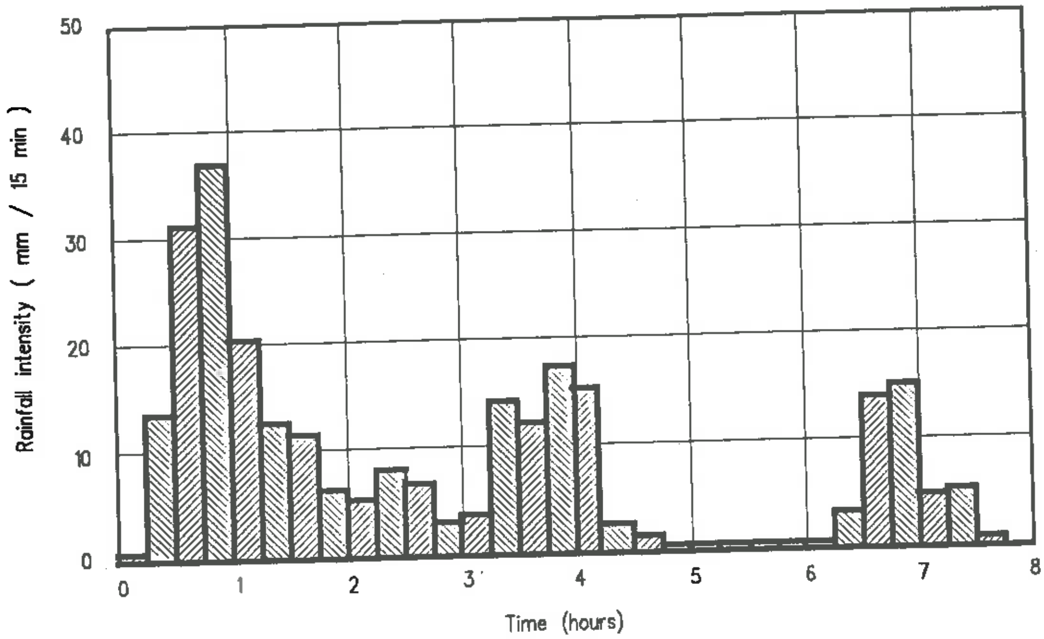

FIGURE 3C.1 Time distribution of the storm rainfall at Bryntirion based on the recording rain gauge record at the University of Pretoria experimental farm.

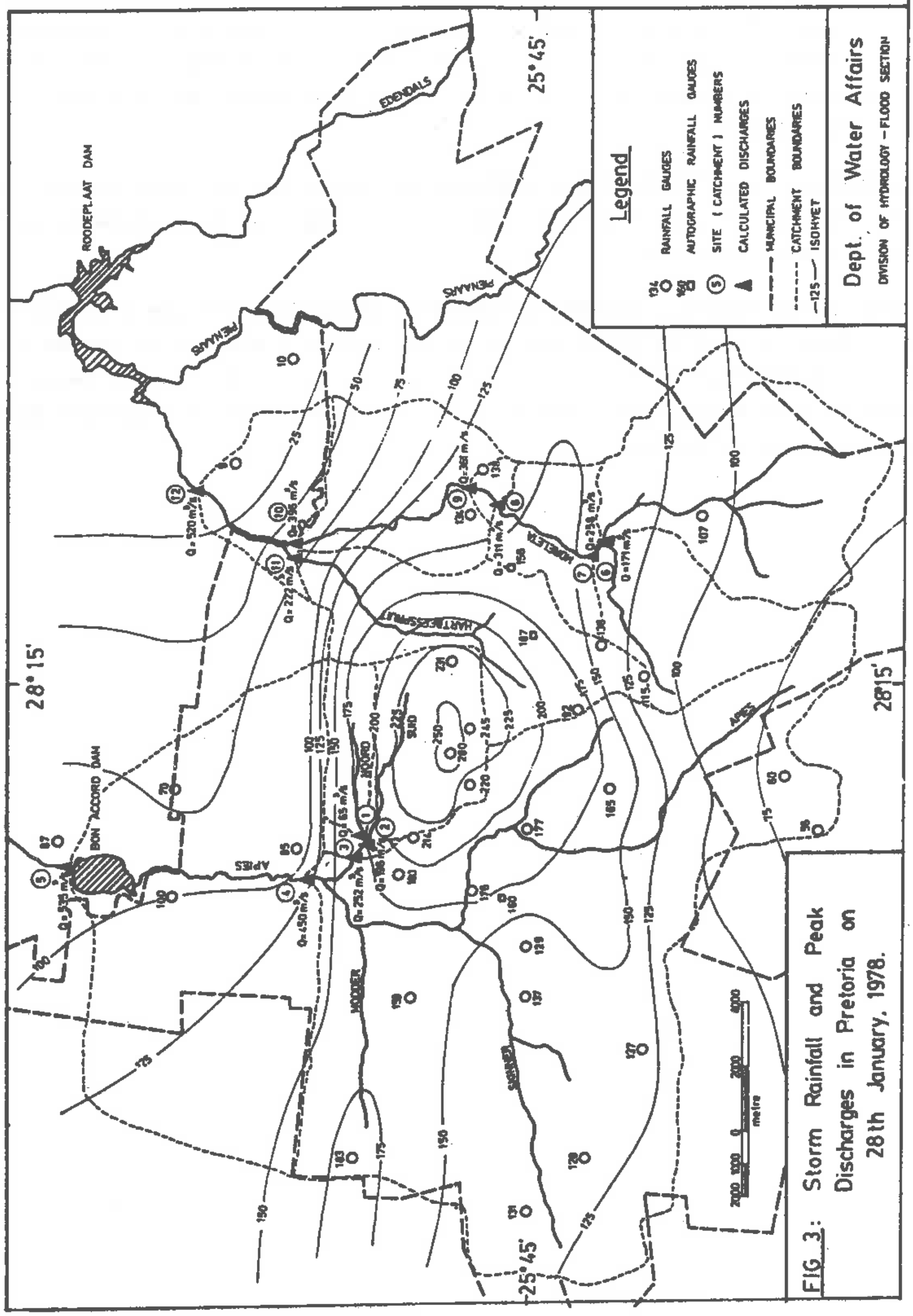

FIGURE 3C.2 Isohyets of the Pretoria storm of 27-28 January, 1978. From TR88 (Kovács 1978)

The high rainfall was caused by the convergence of warm and cool air masses associated with a trough of low pressure over the interior which became pinched between the high pressure systems to the east and west. The elongation of the rainfall isohyets coincides with the orientation of the parallel ranges of the Magaliesberg which may have played a role in the rainfall intensity.

The technical report TR88 contains additional information on flood peaks resulting from the storm which can be used to test the applicability of flood magnitude estimation methods in urban and semi-urban areas.

A long rainfall record is available at Bryntirion where the storm centre was located. The annual daily maximum series for the period 1906 to 1978 is given in the sample data set in the case studies (Chapter 14). A plot of this data on log-probability scales shows a distinct change in slope indicating the possible presence of distinctly separate causative mechanisms for daily rainfall at this site.

3.17 APPENDIX 3D : TROPICAL CYCLONE DOMOINA

28 JANUARY - 1 FEBRUARY, 1984

This is a summary of the information published in Department of Water Affairs technical report TR 122 (Kovács et al 1985).

The tropical cyclone Domoina originated over the Indian Ocean to the west of Madagascar. It moved in a south-westerly direction crossed over Madagascar and the Mozambique channel, and then crossed the African coastline in the vicinity of Maputo ten days later. During the five-day period from 28th January to 1st February, 1984, it moved in an arc westwards over Swaziland, southward over north-eastern Natal and then exited near Cape St Lucia (Fig 3D.1). The resulting floods were the largest in memory over large areas. More than 200 persons lost their lives in South Africa alone. Less than three weeks later a second tropical cyclone Imboa reached the Zululand coast, but moved out to sea again the same day.

Rainfall data were collected from more than 450 locations including 8 stations with recording rain gauges, and flood measurements were made at 85 sites on 45 rivers.

The isohyetal map of the rainfall recorded from 08:00 on 28th January through to 08:00 on 2nd February is shown on Fig 3D.1. The Drakensberg escarpment contained the heavy rainfall to the catchments of the steep east flowing rivers although some moderately heavy rain occurred to the west of the escarpment.

The long, narrow band of isohyets further to the east was the result of the influence of the lower Lebombo mountains.

There are distinct cells of rainfall exceeding 600 mm. The largest of these had maximum diagonal lengths of about 100 km. This generated devastating floods in the Great Usutu, Ngwavuma, Pongolo and Mkuze rivers. The maximum point rainfalls within this cell were between 730 mm and 850 mm. In the northern cell 906 mm was recorded at Pigg's Peak in Swaziland. Up to 924 mm was recorded in the southern cell over the upper catchment of the Black Mfolosi catchment and up to 950 mm in the coastal cell. Except in the coastal cell the maximum rainfall cells were associated with mountainous relief. The highest 1-day rainfall was 615 mm at Pigg's Peak.

In Table 3D.1 daily rainfall depths of stations situated close to the course of the storm are listed in their order along the route of the storm. Higher rainfalls were previously recorded along the coastal area of Zululand and Mozambique, but the Domoina rainfalls were the highest in memory over the inland areas between 150 and 250 km from the shore.

The previously most severe floods in the Mfolosi, Mkuze and Pongolo catchments were those produced by general rain in July, 1963. The maximum 4-day rainfall was 641 mm near Hluhluwe.

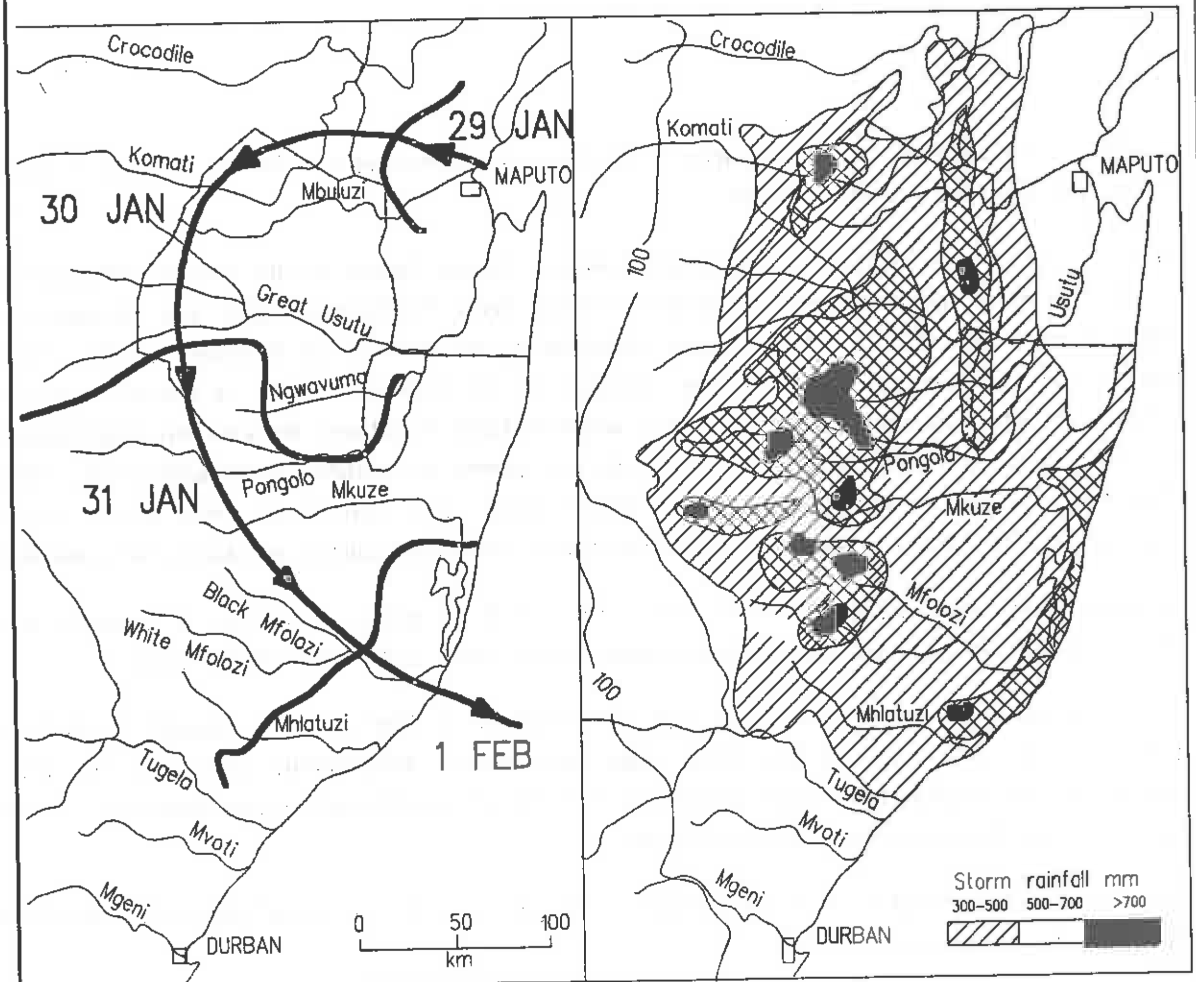

FIGURE 3D.1 Path taken by the tropical cyclone Domoina (left) and the resulting rainfall (right).

TABLE 3D.1 Daily rainfall at selected stations close to the course of the storm.						
	Daily rainfall at 08:00 (mm)					
Station	29 Jan	30 Jan	31 Jan	1 Feb	2 Feb	5-day total rainfall (mm)
Maputo	96	99	9	35	-	239
Komatipoort	85	84	52	37	-	258
Pigg's Peak	225	615	60	6	-	906
Mbabane	45	393	52	17	3	510
Piet Retief	3	186	185	140	10	524
Louwsburg	-	181	245	113	-	539
Nongoma	10	75	164	125	28	402
Hluhluwe. Dam	14	23	87	197	26	374
Cape St Lucia	-	11	20	548	123	702

The 3-day, 200-year return period rainfall was exceeded in 55 of the 83 catchments studied. In the catchments of the Umbeluzi, Great Usutu, Ngwavuma, Pongola and Mfolosi rivers, the Domoina rainfall was of the order of the expected 3-day PMP precipitation given in HRU 1/72. These comparisons are shown in Table 3D.2.

TABLE 3D.2 Domoina rainfall versus PMP in selected catchments				
Site No	River	Catchment area (km^2)	Areal rainfall (mm)	
			Domoina	3-day PMP
12	White Mfolozi	3 939	460	590
16	Black Mfolozi	1 635	610	655
19	Mfolozi	9 248	500	530
28	Pongola	5 788	630	560
32	Pongola	7 831	570	540
33	Ngwavuma	1 660	590	655
48	Great Usutu	17 974	460	440
49	Maputo	31 600	500	340
51	Umbeluzi	3 250	495	610

Fig. 3D.2 shows the distribution of the rainfall at the Umfolozi Game Reserve. At this site the heaviest rain fell during the seven and a half hour period when 278 mm was recorded. However, this was preceded by 115 mm over the previous 24 hours and followed by another 128 mm over the next 24 hours.

Date	Time	12 – hour rainfall	max 30 min rainfall	
1988	hours	mm	mm	
30 Jan	0 – 12	71	9	
	12 – 24	18	5	
31 Jan	0 – 12	311	36	(below)
	12 – 24	66	7	
1 Feb	0 – 12	55	5	
TOTAL		521	36	

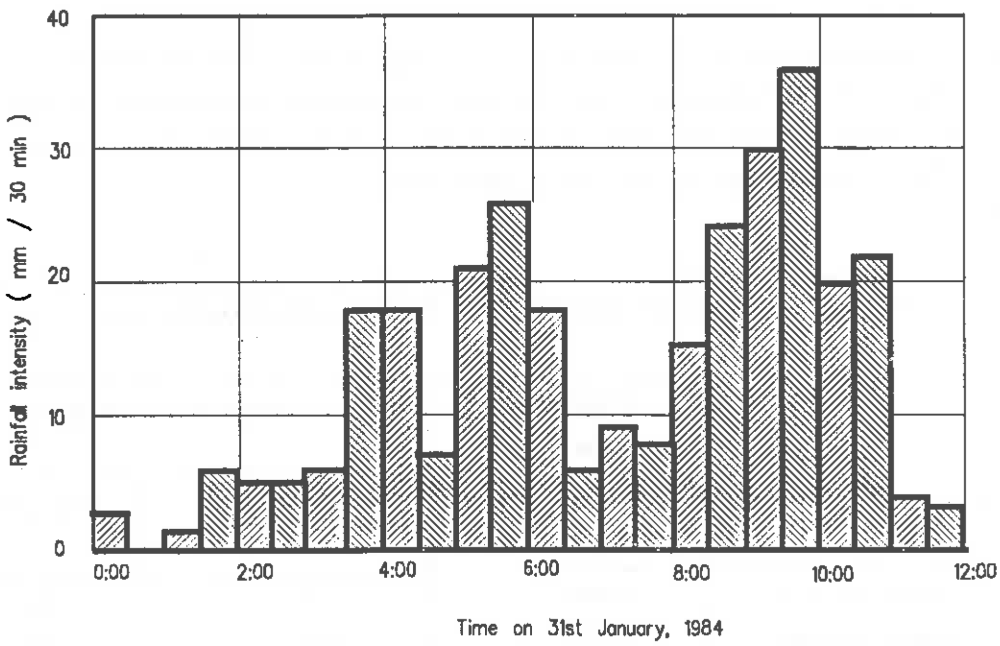

FIGURE 3D.3 Rainfall recorded at the recording rain gauge in the Umfolozi Game Reserve

The cyclone crossed the Usutu, Pongolo and Mkuze River catchments at right angles, but moved directly downstream along the Mfolosi River catchment. In the case of the Mfolosi River this movement plus the effect of the saturating rainfall during the previous 24 hours, resulted in severe flooding along the course of the river.

Fig 3D.4 shows some typical hydrograph shapes produced by the floods including the flood into Pongolapoort Dam. The data set for the Pongola River some distance upstream of the Pongolapoort Dam is given in the case studies in Chapter 14.

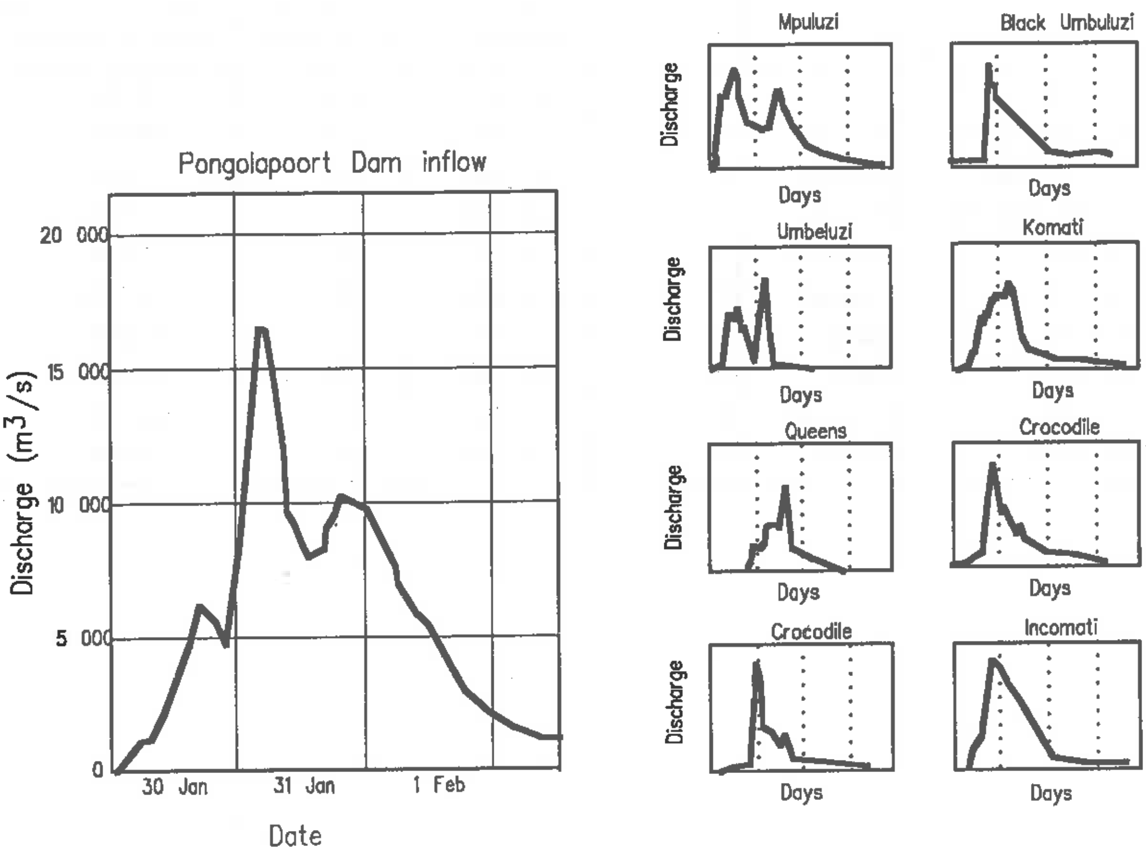

FIGURE 3D.4 Typical hydrographs produced by Domoina (from Kovâcs et al 1985).

Other tropical cyclones that have produced heavy rain over southern Africa.

Only 4% of the tropical cyclones observed in the south-western Indian ocean have caused heavy rainfall in South Africa. Of these Domoina covered the largest area, produced the greatest average depth and volume, and was the only cyclone centre to cross the South African coastline.

Table 3D.3 lists the main characteristics of the ten tropical cyclones that have been experienced in the vicinity of southern Africa since 1950.

Fig 3D.4 shows the rainfall patterns produced by nine of the tropical cyclones (details of the tenth cyclone Caroline are not shown).

TABLE 3D.3 Rainfalls produced by tropical cyclones over South Africa since 1950

Date	Tropical cyclone	Duration (days)	Storm rainfall within 100 mm isohyet		
			Area (km^2)	Depth (mm)	Volume ($10^6 m^3$)
Feb 1956	'A'	5	65 000	155	10 000
Dec '57/Jan '58	Astrid	6	82 000	230	19 000
Jan/Feb 1960	Brigette	4	49 000	155	7 600
Dec '65/Jan '66	Claude	7	24 000	175	4 200
Feb 1972	Caroline		10 200	170	1 700
Feb 1972	Eugenie	7	101 000	150	15 000
Jan 1976	Danae	5	66 000	175	12 000
Feb 1977	Emili	5	53 000	180	9 500
Jan 1984	Domoina	5	107 000	370	40 000
Feb 1984	Imboa	5	26 000	155	4 000

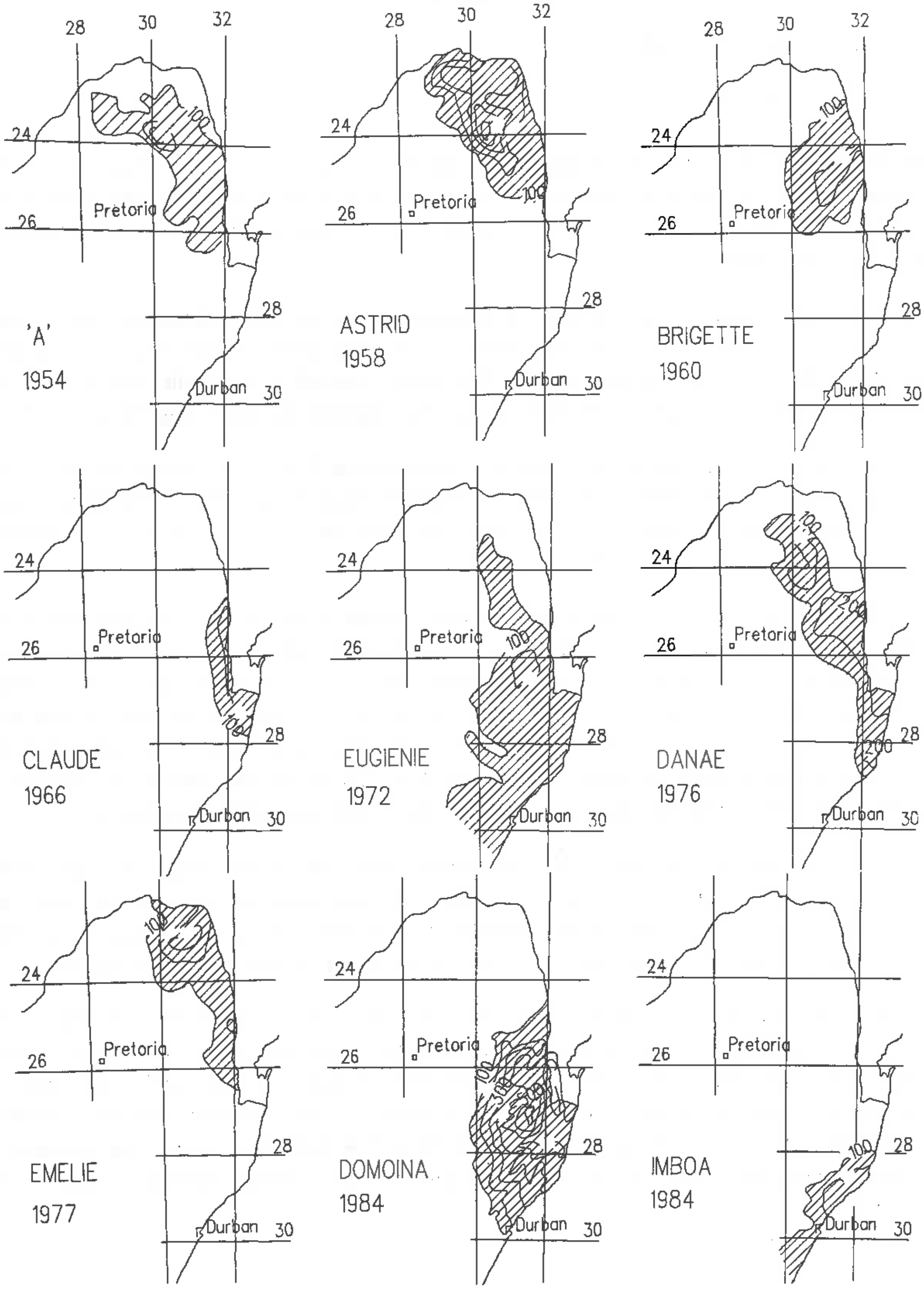

FIGURE 3D.5 Heavy rainfall produced by nine tropical cyclones that have occurred in the vicinity of southern Africa since 1950 (after Dunn, 1985).

3.18 APPENDIX 3E : LAINGSBURG STORM

23-25 JANUARY, 1981

This flood in the Buffels River at Laingsburg caused the heaviest loss of life yet recorded in an urban area in southern Africa. In the town 104 lives were lost; 205 of the total of 367 houses and business premises in the town were destroyed and another 95 damaged (Roberts and Alexander, 1982).

The storm which caused the flooding in Laingsburg was part of widespread rains over a large area of the south western Cape with two distinct storm centres where the rainfall exceeded 250 mm over a 3 day period. The storm covered a triangular area from Cape Town in the west to George in the east and Beaufort West in the north (Fig 3E.1).

The meteorological aspects of the storm are described in Estie, 1981 while the storm and resulting floods are described in detail in the Department of Water Affairs' Technical Report TR116 (Kovács 1981). Other aspects of the floods were the subject of papers by Alexander *et al* (1985), and Alexander and Kovács (1988).

The storm was caused by a strong high pressure system south of the continent which fed warm, moist air into a cold cut-off low which extended well into the upper troposphere. Rainfall data were collected from more than 300 points covering an area of about 75 000 km². The maximum recorded 3 day rainfalls were 375 mm in the storm centre near Robertson and 288 mm in the storm centre north of Laingsburg, which was higher than the mean annual precipitation in that area. Some 60% to 80% of the rain fell on the last day of the storm by which time the shallow soils of the catchment were close to saturation.

A similar storm occurred further inland in March, 1961. The storm isohyets are also shown in Fig 3E.1. The areas receiving more than 50 mm were 65 000 km² in 1961 and 45 000 km² in 1981. The mean storm rainfalls were 80 mm and 110 mm respectively. Both systems caused some of the most severe floods yet recorded in their respective regions.

Of particular interest in the 1981 floods were the observations that return periods of the floods were far in excess of those of the rainfall that generated them. The 3-day rainfall exceeded the calculated 3-day/200-year rainfall over an area of some 4 000 km², while the inferred return periods exceeded 10 000 years in several rivers with long records (Alexander and Kovács 1988). Two of these records (Buffels and Willem Nels Rivers) are included in the sample data set in the case studies in Chapter 14, and will be referred to later in this handbook.

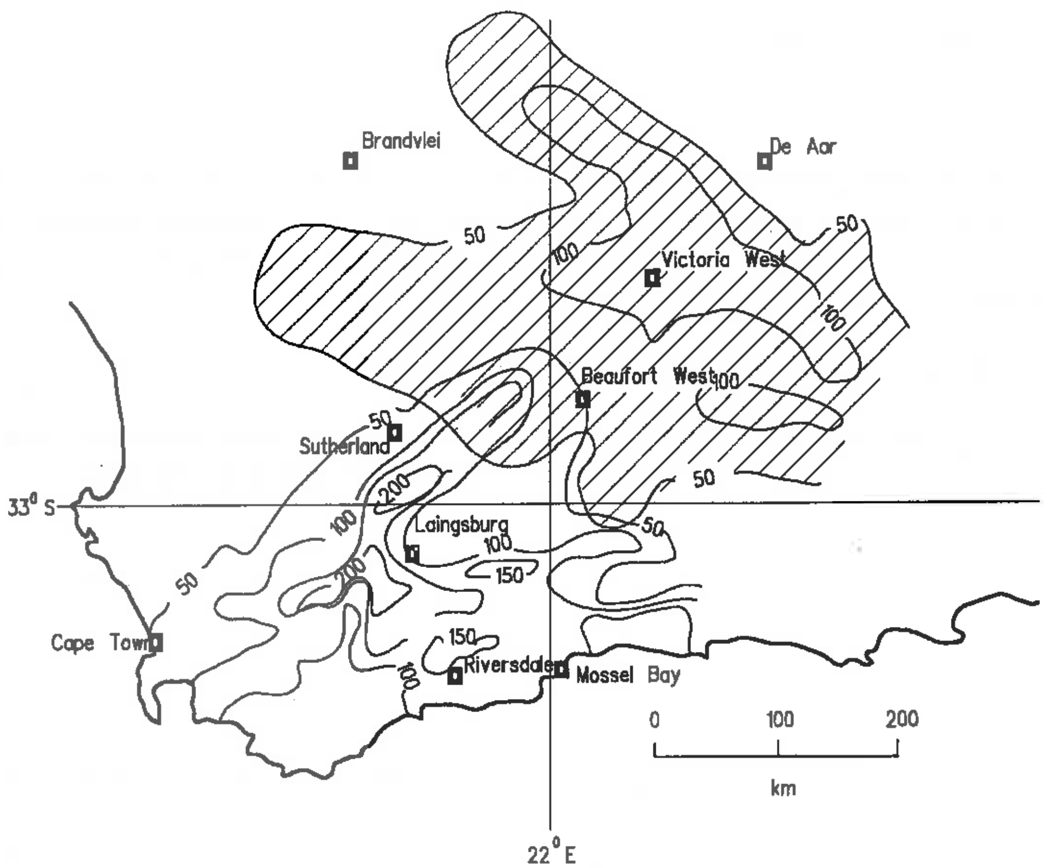

FIGURE 3E.1 Isohyets of the storms over the south-western Cape in January, 1981, and central Karoo in March,1961 (shaded).

3.19 APPENDIX 3F : VAAL DAM CATCHMENT

12-21 FEBRUARY, 1975

The floods which were generated in the Vaal Dam catchment during this nine day period were the consequence of three distinct storm events, each with different locations and isohyetal patterns. The flood hydrographs and the distribution of the storm rainfall are shown in Figures 3F.1 and 3F.2.

The implications are discussed in Chapter 10 on flood routing.

There is a long flow record for station C1M01 in the Vaal River near Standerton. This is included in the sample data set in Chapter 14 as is the flow record in the Vaal river at Vereeniging.

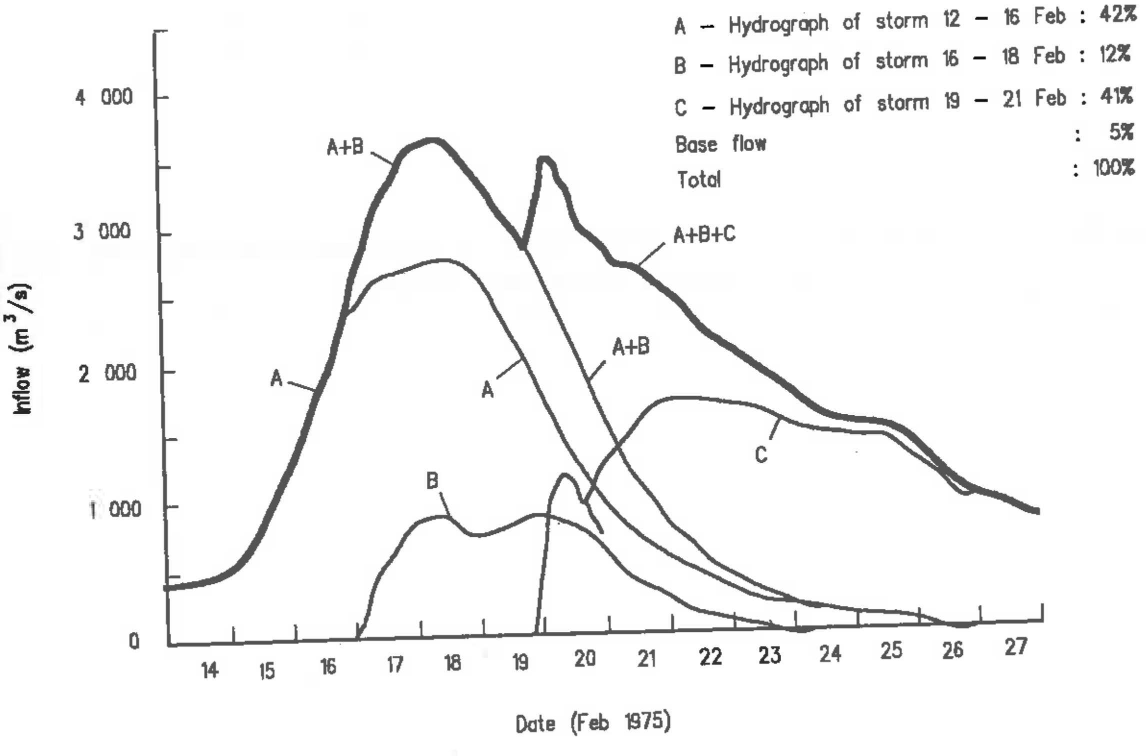

FIGURE 3F.1 Inflow hydrographs at Vaal Dam resulting from the three successive storms.

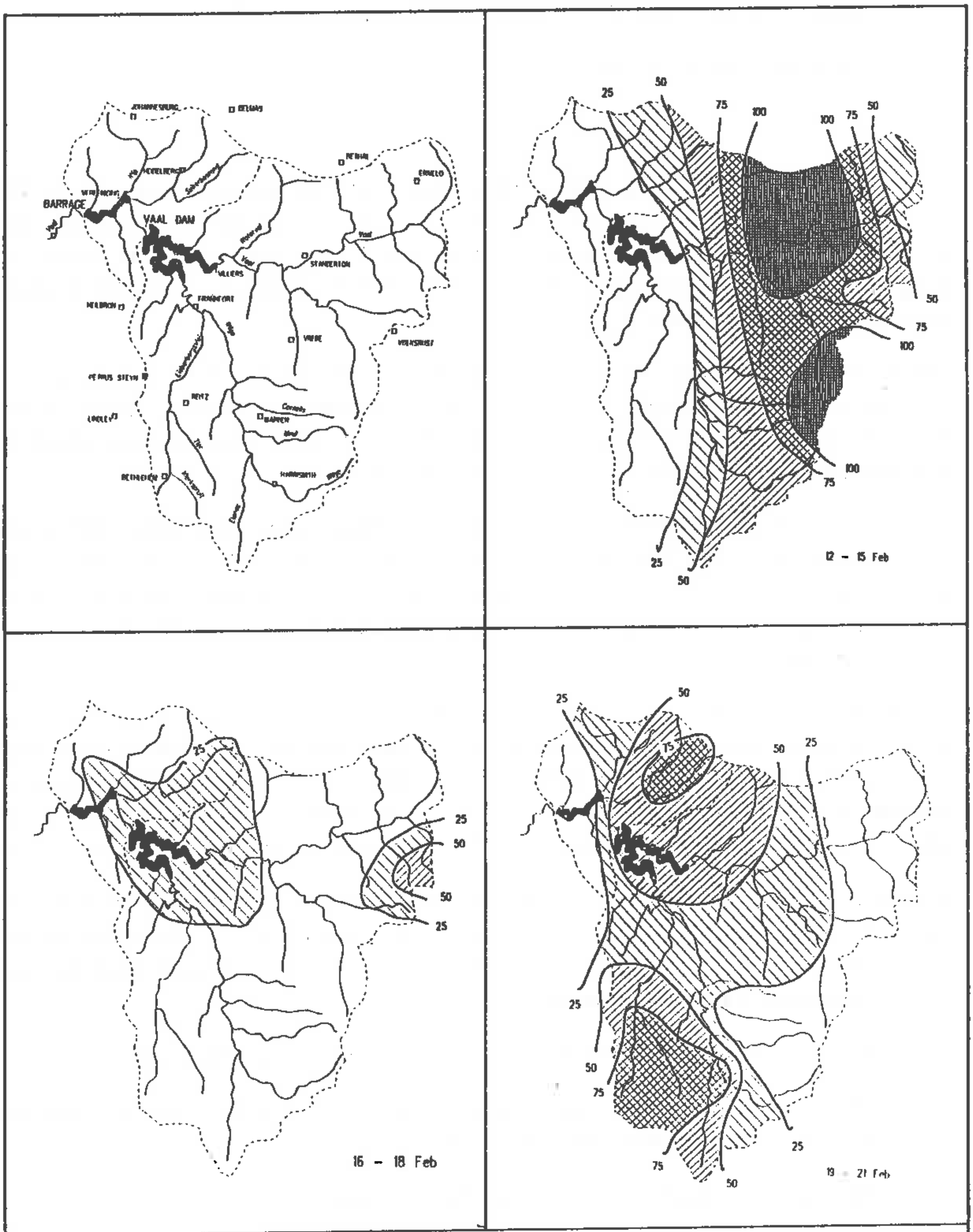

FIGURE 3F.2 Rainfall distribution pattern of three successive storms over the Vaal Dam catchment during February, 1975.

3.20 APPENDIX 3G : NATAL, OFS and N.CAPE SEASONAL RAIN JUNE 1987 TO MAY 1988

Very severe flooding occurred in Natal, the Orange Free State and the northern Cape during the 1987-88 summer season. Fig 3G.1 is a series of maps showing the actual monthly rainfall as a percentage of the median rainfall for each month from August, 1987 to March, 1988. Large areas of southern Africa experienced rainfall well in excess of the monthly median for each month other than January.

Similar widespread rains occurred during March and April 1974. The location and orientation of heavy rainfall along the axis of the typical low pressure troughs is clearly apparent in September, 1987, and February 1988 when severe country wide floods were experienced. The same situation occurred in March 1974.

A total of 287 flood related deaths were reported in Natal during September, 1987 of whom 214 drowned, and 101 persons were reported missing. The heavy loss of life was primarily due to steepness of the rivers with consequent high velocity, turbulent flow and little or no flood warning. The low lying business district of Ladysmith was flooded on six occasions during the season.

In the Orange Free State and northern Cape the prolonged, early season rains filled farm dams and natural pans, raised the water table, and increased the soil moisture status thereby creating ideal conditions for the development of floods later in the season. This is what occurred during February and March when two large dams and a large number of smaller dams were destroyed, communications were disrupted over a wide area on several occasions.

A characteristic of this situation was the repeated succession of floods which followed the major flood and disrupted repair work. There was a greater risk to the travelling public during the subsequent minor floods when repairs were being undertaken than during the severe flood which caused the damage.

Successive floods also create problems during flood routing through flood control dams.

These aspects are dealt with in later chapters. The 1987 Natal floods are included in the data used in the Natal regional case study in Chapter 14.

More details of these floods can be found in the following publications :

- The January, 1988 issue of *The Civil Engineer in South Africa* which was a special issue devoted to the September, 1987 floods in Natal.
- The Weather Bureau Technical Paper No 19 by Triegaardt, Terblanche, van Heerden and Laing *The Natal floods of September,1987*.

- Several papers in the proceedings of the conference on *Floods in perspective held in Pretoria* in October, 1988, organised by the Division of Water Engineering of the South African Institution of Civil Engineers.

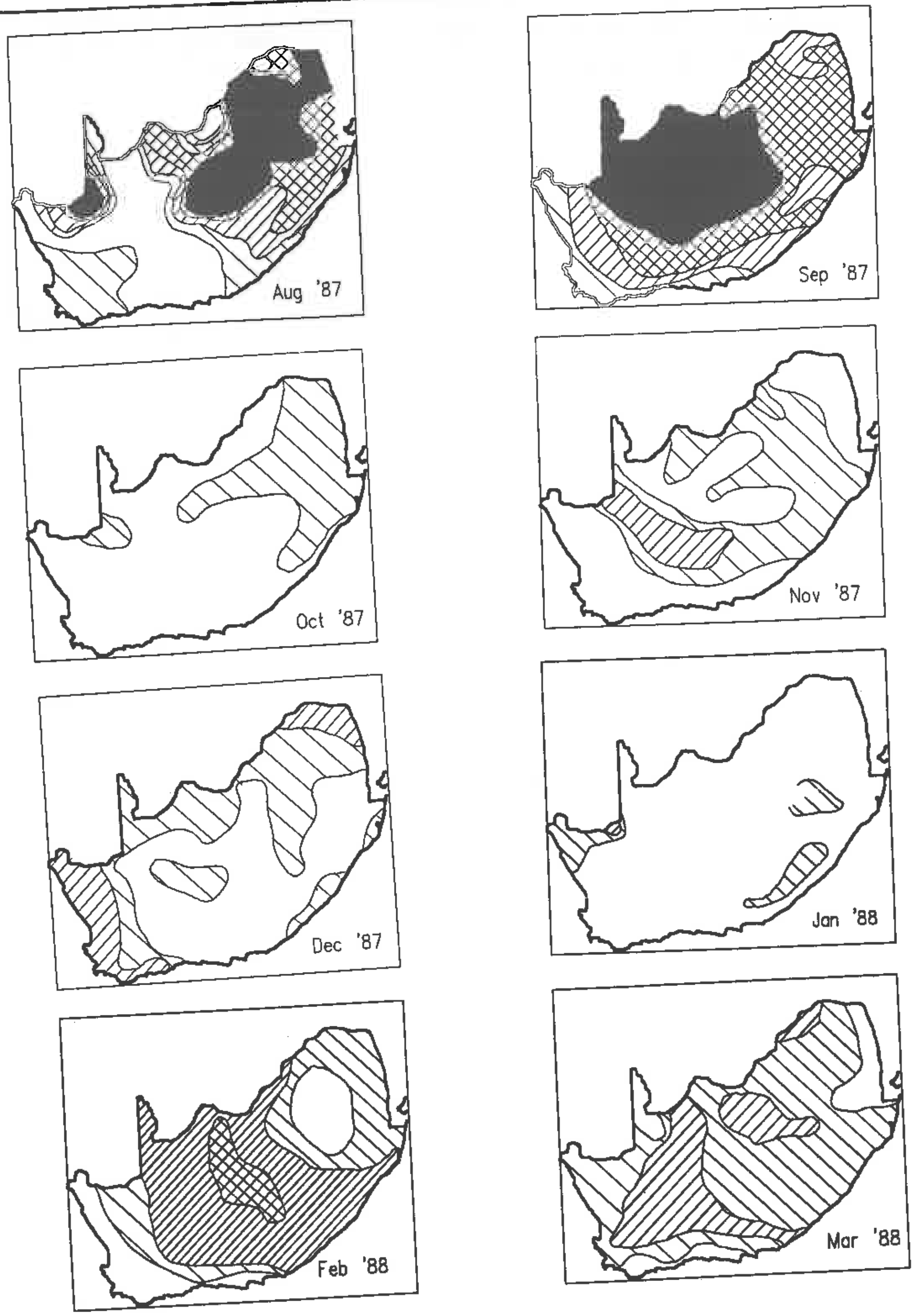

FIGURE 3G.1 Monthly rainfalls over southern Africa from August, 1987 to March, 1988 expressed as ratios of the average rainfall for the month. Less than the mean is unshaded. Thereafter the shading intensity is for factors 1 to 2, 2 to 5, 5 to 10, and greater than 10x the monthly mean.

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Chapter 4

CATCHMENT CHARACTERISTICS AND PROCESSES

Contents

Note:

The figures and tables in this chapter should not be used for calculation purposes. Those that are recommended for inclusion in calculation methods are repeated in the appendix to Chapter 13.

4.1 INTRODUCTION

4.1.1 A cautionary note

The cautionary note in the introduction to Chapter 3 is equally relevant to the effect of catchment processes on the flood magnitude-frequency relationship. Care must be taken to ensure that the assumptions made in the deterministic methods do not invalidate the assumption that the flood magnitude-frequency relationship is the same as the rainfall depth-frequency relationship when all other variables are held at their average or median values.

There is mounting evidence in South Africa that extreme floods occur more frequently than can be inferred from storm rainfall frequency studies. The most likely reason for this is the combination of intense rainfall occurring on a catchment with an abnormally high catchment moisture status due to antecedant rainfall. The annual exceedance probability of a flood is a function of the joint probability of abnormally high rainfall occurring on an abnormally wet catchment which is a very small probability if the causative storm and the antecedant conditions are mutually independent, but much greater when they are closely correlated. This will be discussed in detail towards the end of this chapter but should be borne in mind when reading through the earlier comments.

In humid climates the differences between mean annual rainfall and mean annual potential evaporation are small with the result that there is an almost continuous movement of water through the soil profile and the river systems. The result is that a high percentage of the rainfall is converted into runoff. (See Fig 4.1). The *variability* of the process is small and it does not have a meaningful effect on the assumption that the AEP of the flood is that of the storm rainfall. This is probably why the quantification of antecedant conditions has received so little attention in the literature when compared with the multitude of papers on the properties of storm rainfall.

With the exception of small areas along the southern and eastern mountain ranges the mean annual potential evaporation (MAPE) exceeds the mean annual precipitation (MAP) over the whole of southern Africa. In the arid areas of South Africa, Botswana and Namibia the MAPE exceeds the MAP by factors of 25 and more (Alexander, 1985). This is shown in Fig 4.2.

Consequently the antecedant catchment moisture status varies over a very wide range from storm to storm and has a pronounced effect on the variability of the rainfall-runoff process. The assumption that the AEP of the flood is the same as that of storm rainfall becomes more questionable with increase in aridity.

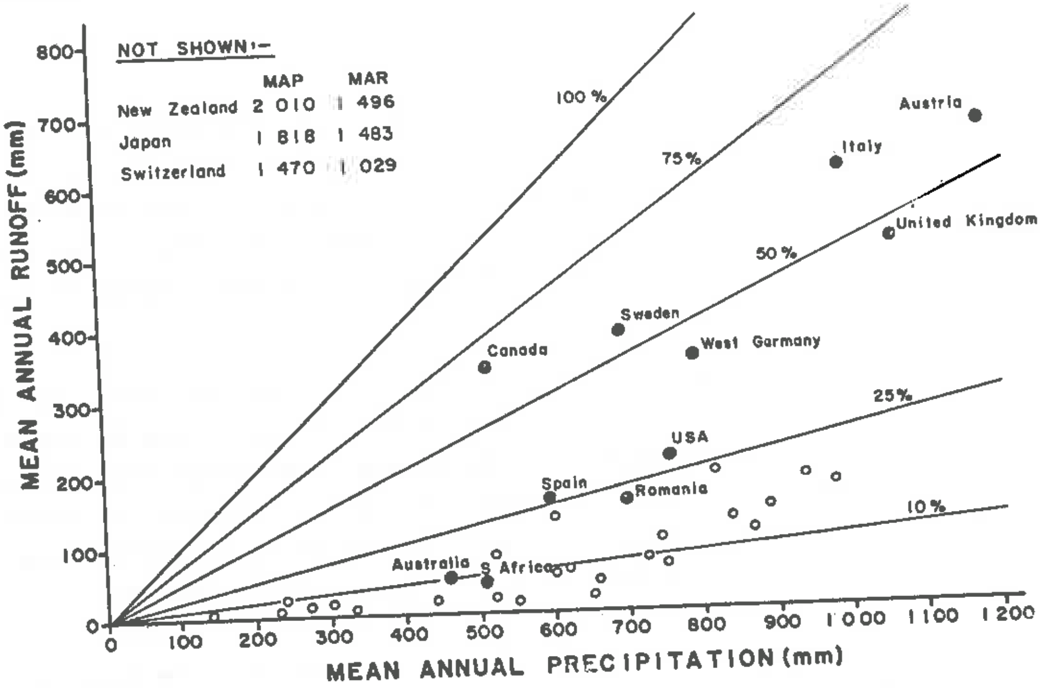


FIGURE 4.1 Relationship between mean annual precipitation (MAP) and mean annual runoff (MAR) for selected countries in the northern and southern hemisphere (named) as well as for major drainage regions in southern Africa (open circles).

FIGURE 4.2 Relationship between mean annual rainfall and mean annual potential evaporation in southern Africa.

Refer to Department of Water Affairs (1986) for more information on the rainfall, gross potential evaporation, river flow, climate and topography of southern Africa.

4.1.2 Overview

Before runoff can take place the rainfall intensity has to exceed the rate of infiltration into the soil and all depressions in the path of the resulting surface flow have to be filled. The rate of infiltration is a function of the properties of the soil and its moisture status.

All natural and man made features within the catchment will have an effect on the rainfall-runoff process. Some of these such as rock outcrops and paved areas are time invariant, while others such as vegetation change with the seasons and from year to year. The wetness of the catchment resulting from the combined effect of antecedent precipitation (moisture gain), and antecedant evapotranspiration (moisture loss), will have an important effect on the runoff from subsequent storms.

The magnitude of the flood peak is directly related to the rainfall intensity, which in turn is inversely related to the storm duration (see Chapter 3 on storm rainfall). The storm duration is usually assumed to be equal to the catchment response time (see Chapter 7 on deterministic methods). The three main independent variables which determine the catchment response time and consequently design storm duration are the area, shape, slope and the drainage channel density.

Several problems arise when attempting to determine the effects of the major catchment characteristics on the magnitude of a flood. Some of these such as the area of the catchment are easily measured and quantified while other major characteristics such as infiltration rates can not. A further hurdle is that many of the characteristics such as mean annual rainfall and vegetal cover are interrelated and should not both be used in the rainfall-runoff model if there is a high degree of correlation between them.

The Witwatersrand University Hydrological Research Unit's Report No 1/72 (HRU 1/72) (Midgley, 1972) makes use of zones of differing veld types (dominant vegetation types) to differentiate between regions of differing hydrological characteristics. The reasoning is that the dominant vegetation type depends on many characteristics such as mean annual precipitation, soil type, geology, elevation, aspect and topography, all of which together with the vegetation itself play major roles in runoff process. This information is shown on a single map.

In the Department of Water Affairs (DWA) implementation of the rational method, the main catchment characteristics are identified and have to be measured separately.

In several cases alternative methods are available for measuring a single characteristic - for example catchment shape. It is important that the method specified by the developers of a method be used when applying the method unless there are strong reasons for using an alternative procedure.

In other situations the "best" alternative may be sought, but the user must be sure that this will not violate the underlying assumption of the method itself. For example, if the method was calibrated against observed flows by adjusting parameter values, it would be incorrect to change one of these parameter values even if the change provided more realistic estimates of that characteristic.

There is a need for uniformity in the methods of measurement if comparisons are to be made between the results from several catchments, or the analyses by different persons.

Uniformity is essential where catchment characteristics are used to define hydrologically homogeneous catchments (Chapter 6) as well as in regional analyses (Chapter 7).

Finally, when selecting sets of characteristics to be included in a model it is essential that there should be little or no relationship between the selected characteristics ie they must be statistically independent, unless the model is designed with intercorrelation in mind.

The *Flood Studies Report* deals exhaustively with this subject. Multiple regression techniques were used to study the relationships between the mean annual flood and catchment characteristics.

4.2 MACRO SCALE CHARACTERISTICS

4.2.1 Area

The size of the catchment is the obvious dominant catchment characteristic. Within a hydrologically homogeneous region the flood peak for a given return period varies roughly with the square root of the area. This is discussed further in Chapter 7 on deterministic methods.

The standard maps recommended for flood analyses are the 1:50 000 scale topographical series published by the Government Printer. Where the catchment area is less than about 6 km^2 higher resolution maps should be used. If the catchment covers more than four 1:50 000 sheets the 1:250 000 topographical maps should be used, and if the catchment exceeds four 1:250 000 sheets then deterministic methods are no longer applicable.

If the problem site is located in a river basin where DWA gauging stations are located, the published DWA gross areas for the gauging stations may be accepted and only the differences in areas need be calculated.

The method of measurement may either be through the use of a digitizing tablet, planimeter, or by tracing the boundary on squared graph paper and counting the squares.

Internal drainage

Internal drainage areas consisting of the catchments of pans and lakes should be measured to the same scale as that used to determine the catchment area and subtracted from the gross catchment area. Note that the DWA published areas for gauging stations are the gross areas and have to be adjusted for internal drainage areas. Only internal drainage areas feeding pans which will not contribute to flood peaks in the system even when they are filled with water should be excluded.

Undefined boundaries
In very arid regions the catchment boundary may be difficult to define. A subjective estimate of its location will be adequate in view of all the other uncertainties in this situation.

4.2.2 Shape

A wide, fan-shaped catchment will have a shorter response time and consequently higher calculated peak than a long, narrow catchment. The catchment shape will also have an effect on the shape of the flood hydrograph.

There are several methods available for characterizing catchment shape. Fig 4.3 shows three methods :

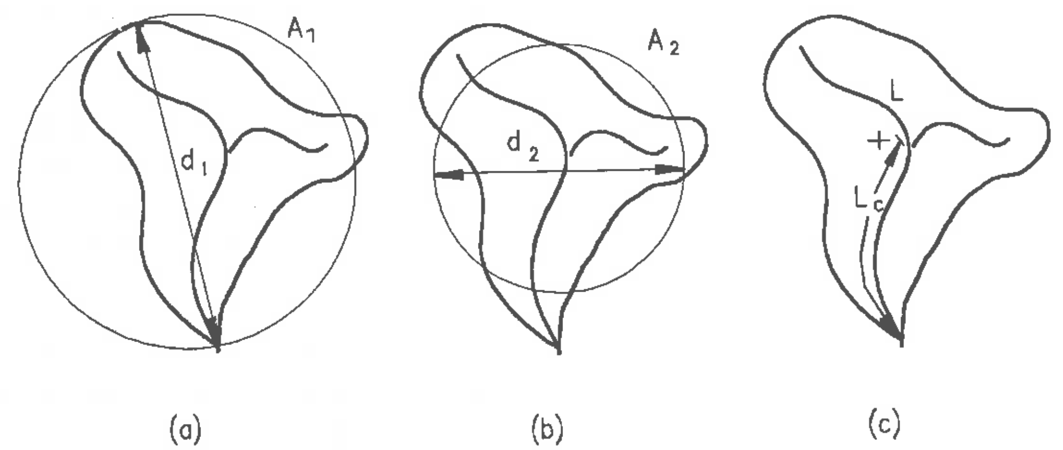


FIGURE 4.3 : Definition sketches for catchment shape factor

A	=	area of the catchment	
d_1	=	longest diagonal	
d_2	=	diameter of circle having area	= A
A_1	=	area of circle having diameter	$= d_1$
A_2	=	area of circle having diameter	$= d_2$
L	=	main stream length to watershed	
L_c	=	main stream length to a point opposite the centroid of the catchment	

The longest diagonal in (a) is sensitive to anomalous irregularities in the catchment boundary. Method (b) is based on the elongation ratio recommended by Schumm (1956) as quoted in Eagleson (1970). This is the ratio of the diameter of a circle having the same area as that of the catchment, to the length of the longest stream from the problem site to its intersection with the catchment boundary (to be described in more detail in the next section). According to Eagleson the elongation ratio seems to be strongly correlated with catchment relief and varies from near unity in regions of low relief to values in the range 0,6 to 0,8 in catchments with steep slopes and high relief.

Method (b) is preferred as it does not involve the measurement of additional variables, and secondly because in method (a) the longest diagonal d is not statistically independent of the length of the longest stream L.

Method (c) illustrates the derivation of the catchment shape variable used in HRU 1/72. It is the length from the mouth of the catchment along the main stream to a point opposite the centroid of the catchment. An eyeball estimate of the location of the centroid is adequate. The distance measurement method is the same as that used for the main channel length.

4.2.3 Main channel slope

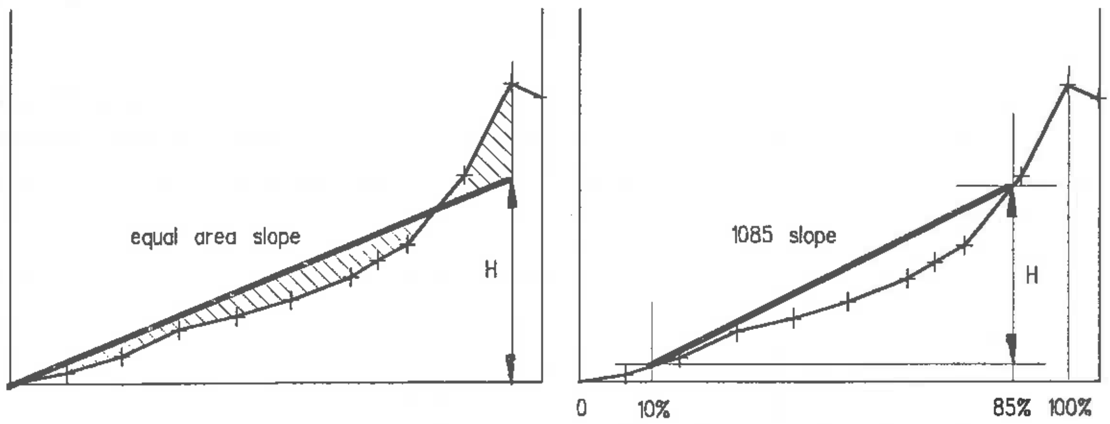
The two values that have to be measured to determine the channel slope are its length and the corresponding height difference. The slope unit is in metres height difference per metre of channel length.

The measured length of a stream from the site to the catchment boundary will depend on the map scale as well as on the method of measurement. To ensure uniformity of measurement which is a requirement for comparing catchment characteristics, it is recommended that the same maps be used as those used for calculating the area of the catchment.

Dividers should be used for measuring the main stream lengths. These should be set at 0,2 km for 1:50 000 maps and 1,0 km for 1:250 000 maps. When the latter maps are used the length should be multiplied by a factor of 1,2 to correct for loss of resolution.

The distances along the length of the stream where the contour lines are crossed should be used to plot the profile. Where waterfalls and rapids are clearly evident as discontinuities in the profile, the profile should be adjusted downwards to eliminate them.

Two principal methods have been used to determine main channel slope. These are the equal area method recommended in HRU 1/72, and the 1085 method recommended in the *Flood Studies Report*. These are illustrated in Figs 4.4(a) and (b).



(a) Equal area height method (b) 1085 height method

FIGURE 4.4 : Definition sketches for determining main channel slope

The sketches are self-explanatory. The *Flood Studies Report* found that the 1085 method developed by the United States Geological Survey was the most reliable indicator of channel slope. It has the additional advantage that it can be determined directly from the plotted profile, while the equal area method has to be determined indirectly. It is recommended in this handbook for all methods other than the HRU methods. The height difference is that between points located 10% and 85% of the distance along the main stream length, and the effective main stream length is 75% of the total length. For the equal area method a visual fit will be adequate.

If two or more streams of approximately equal length converge near the problem site, the main channel slopes should be determined separately and the average determined.

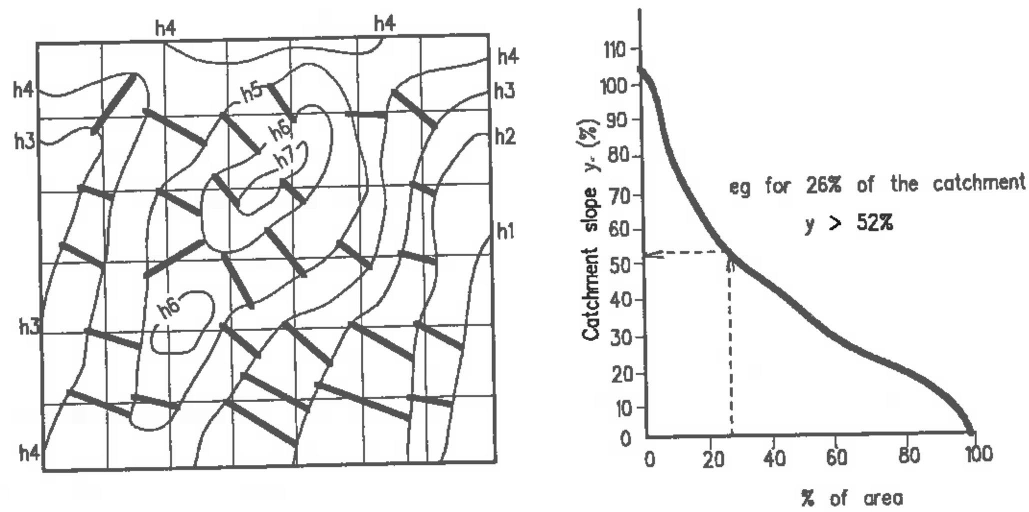
4.2.4 Drainage channel density

This is measured in units of junctions per square kilometer. The denser the drainage system the greater the proportion of the precipitation that will contribute to direct runoff and the shorter the catchment response time. The drainage density is determined by counting the average number of stream junctions per square kilometre. As this depends on the scale of the map it cannot be determined uniquely. The 1:50 000 Trigonometrical Survey series would be very suitable for South African conditions but as no drainage density counts have yet been made of South African catchments this information is not readily available. This variable is independent of the other variables which control catchment response time and was found to be significant in the *Flood Studies Report* studies it may be worthwhile to include it in future studies.

4.2.5 Average catchment slope

Average catchment slope is different from the slope of the main stream, although a degree of correlation may exist.

The recommended procedure is to superimpose a grid of squares over the catchment and at each grid intersection point determine the shortest distance between the two contours that straddle the grid point along a line which passes through the grid point. This is shown diagramatically in Fig 4.5.



(a) Determination of steepness (b) Steepness frequency curve

FIGURE 4.5 : Definition sketch of average catchment slope

The grid size should be such that not less than 50 grid points lie within the catchment. The map scale should be the same as that for determining the main channel slope.

The average catchment slope is the contour interval divided by the average of the distances between contours.

An advantage of the grid method is that the frequency distribution of the grid point slopes can be determined as shown in Fig 4.5(b). This is useful for comparing catchment characteristics.

4.3 MICRO SCALE CATCHMENT CHARACTERISTICS

Micro scale characteristics are those which individually occupy a small proportion of the catchment and may have a high degree of variability but which jointly play a significant role in the storm rainfall-runoff process. These include vegetal cover, soil type and land use.

4.3.1 Vegetal cover

Vegetal cover retards surface runoff and is also an indicator of other hydrological characteristics. The HRU methods make use of a map of veld type zones (see Chapter 7 on deterministic methods). The DWA categorization used in their rational method is as follows:

(a) Forest plantation (whether recently felled or not)
(b) Natural forest or dense bush
(c) Thin bush
(d) Cultivated land
(e) Grassland
(f) Bare surface

The 1:50 000 topographical maps show rock outcrops, cultivated lands, orchards and vineyards, scattered bush, and forest plantations, and can therefore be used for delineating areas of different vegetal cover. It is always good practice to inspect the catchment where the investigation is required for major or high risk structures. The topographic maps supplemented by a site visit should be adequate for this classification.

4.3.2 Soil type

Soil permeability controls the infiltration rate and consequently the balance of the rainfall that constitutes surface runoff and contributes to the flood peak.

The DWA methods use four categories of soil types based on their runoff potential :-

Soil group A : Low run-off potential (= very permeable)
. infiltration rate is high
. soils are deep
. texture is coarse (coarse sand, gravel)
(all dolomitic soils are included in this category).

Soil group B : Moderately low run-off potential (= permeable)
. moderate infiltration rate
. fine sand, loams.

Soil group C : Moderately high run-off potential (=semi permeable)

- low infiltration rate
- depth is shallow
- texture is fine (sandy loams., sandy clays)

Soil group D : High runoff potential (= impermeable)

- close to zero infiltration rate
- very shallow and/or expansive soils
- swelling clays, rocks

The Department of Agriculture is in the process of preparing 1:250 000 scale soil maps, some of which have already been published. These should be used where available. The maps show 28 broad soil patterns. The corresponding runoff classifications are shown in Table 4.1.

TABLE 4.1 Runoff classification of soils

Soil classification	Runoff classification	Soil classification	Runoff classification
Aa	A	Da	D
Ab	A	Db	D
Ac	A	Dc	D
Ad	A	Ea	C
Ae	A	Fa	C
Af	A	Fb	C
Ag	C	Fc	C
Ah	A	Ga	A
Ai	A	Gb	C
Ba	B	Ha	A
Bb	B	Hb	B
Bc	B	Ia	B
Bd	B	Ib	D
Ca	C	Ic	D

Where these maps are not available an inspection of the site will be required. This will be adequate for most applications.

4.3.3 Land use

This overlaps the vegetal cover classification and is therefore of doubtful validity as a separate classification. Urban areas are shown on the 1:50 000 maps. Further subdivision into zones of decreasing permeability in urban areas are lawns and parks, residential, industrial, and city centre areas.

4.3.4 SCS-based method

The SCS-based design runoff method (Schmidt *et al* 1987) is more dependent on catchment characteristics than other methods, and their publication contains useful information although the method itself is limited to agricultural catchments having areas of 8 km^2 or less.

4.4 CATCHMENT PROCESSES

4.4.1 Effect of vegetal cover

Earlier in this chapter the type of vegetal cover was identified as an important catchment characteristic. In humid areas the effect of vegetal cover does not change significantly during the high runoff season. However, in semiarid areas the vegetal cover may vary appreciably during the season as well as from season to season thereby introducing greater variability in the flood peak series.

4.4.2 Effect of evaporation

Surface runoff is directly related to rainfall intensity and inversely related to the rate of infiltration into the soil. The infiltration rate for a given soil decreases with increase in moisture content and may approach zero after prolonged heavy rainfall.

During periods between storms the soil moisture is depleted by transpiration from vegetation and evaporation from the soil surface. In arid areas this depletion of the soil moisture increases the initial infiltration rate as well as the depth of precipitation that is utilized to bring the soil back to a condition of near saturation. It therefore follows that the greater the evaporation potential (ie the greater the aridity of the area) the greater the influence of the time period between successive storms. This introduces additional variability in the rainfall-flood runoff relationship.

4.4.3 Effect of subsurface water

In high rainfall areas there is an almost continuous movement of water through the soil and rock profile towards the river channel thereby creating perennial river flow. With decrease in mean annual rainfall this movement becomes increasingly more sporadic, the base flow becomes seasonal, and eventually in the arid areas there is no base flow at all.

4.4.4 Contributing areas and interflow

The concepts of *contributing areas* and *interflow* have been used by authors on hydrology related to humid climates. These authors postulate that surface runoff is most likely to occur in the low portions of the catchment nearest the streams as the soil moisture content is highest in these areas. The contributing area expands with increase in rainfall.

Further from the stream channels interflow is presumed to provide a significant contribution to the flood hydrograph. This interflow takes place within the saturated zone and can be described by a kinematic wave approximation (Eagleson 1970). Alternatively interflow is considered to be the flow constituting the recession limb of the hydrograph and its source is water temporarily stored in the soil which continues to contribute to the flood hydrograph until the whole of the effective rainfall is depleted (Shaw 1983).

Neither of these concepts is likely to be useful in southern Africa where there is no published evidence to support the view that subsurface flow makes a meaningful contribution to flood hydrographs. Indeed, an inspection of excavations, road cuttings and unlined tunnels during heavy storms will provide evidence to the contrary.

4.5 QUANTIFICATION OF CATCHMENT PROCESSES

4.5.1 Accumulated rainfall vs accumulated runoff

Fig 4.6 is a diagrammatic representation of the relationship between accumulated rainfall and accumulated runoff both measured in equivalent depths of water over the catchment (mm), assuming a constant rainfall rate.

Runoff commences at time t_1 (Fig 4.6(a)). Prior to that time no surface runoff occurs because infiltration losses and pondage losses exceeding the accumulated precipitation. The infiltration rate decreases as the soil moisture content increases. At time t_1 the initial depression storage and retention demands have been satisfied and the infiltration rate is less than the precipitation rate. The rainfall in excess of these requirements becomes surface runoff.

The infiltration rate and incremental storage loss rate both decrease with increase in accumulated rainfall. By time t_2 both of these losses become insignificantly small and the incremental runoff becomes equal to the incremental rainfall.

The shape and position of the runoff curves (Fig 4.6(b)) will depend on the initial accumulated precipitation depth required to initiate runoff (p_3 and p_4) and the associated times t_3 and t_4, as well as the precipitation depths required to satisfy the soil moisture deficit and storage requirements (p_5 and p_6).

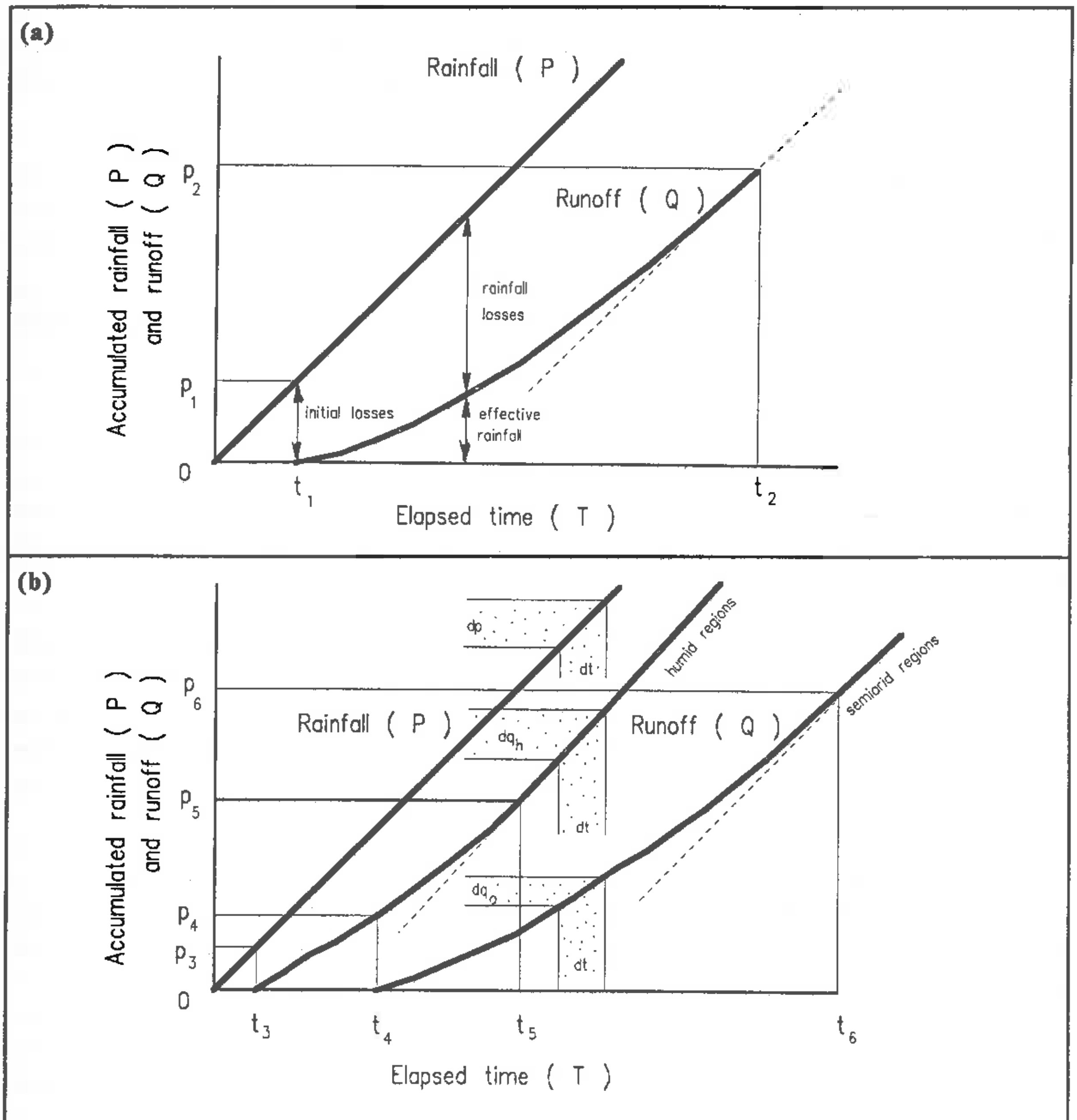


FIGURE 4.6 Relationship between accumulated rainfall and accumulated runoff.

The upper runoff curve in Fig 4.6(b) is typical of the average condition in humid regions which have a high soil moisture status and relatively low permeability. The lower curve is typical of the average condition in semiarid regions which have low soil moisture status due to the high and prolonged evaporation losses between storm events, and soils which have a lower clay content and are consequently more permeable.

For a given time increment *dt* the corresponding incremental rainfall depth is dp and the incremental runoff depths are dq_h for the humid region curve and dq_a for the arid region curve. In this example the incremental effective rainfall is appreciably higher in the humid region that the semiarid region.

Furthermore, the probability of an accumulated rainfall depth being experienced in a humid region is much greater than the probability of this depth being experienced in a semiarid region. Consequently *on average* the proportion of the rainfall that contributes to the flood peak is greater in a humid region. It is this observation that results in the allocation of higher runoff coefficients (or lower rainfall loss assumptions) in deterministic rainfall-flood runoff models.

4.5.2 The effect of antecedant rainfall

It must be appreciated that it is not the total storm rainfall that contributes to the flood peak, but the effective storm rainfall. The effective storm rainfall is dependent on the antecedant catchment moisture status which in turn is a function of antecedant rainfall.

Because time averaged storm rainfall intensity for a given return period decreases with increase in duration, most deterministic flood rainfall-runoff models assume that the critical storm precipitation depth is the depth derived from the depth-duration relationship for the required return period assuming a duration equal to the calculated catchment response time. This storm is assumed to commence at time zero in Fig 4.6(b) and extend to the assumed storm duration. The effective rainfall depth is then determined.

However, it is obvious from Fig 4.6(b) that a greater effective rainfall depth will result if the duration of the storm exceeds the catchment response time. This is equivalent to placing the catchment response time within but to the right of the total storm duration on the horizontal axis.

For a given annual exceedance probability the design storm rainfall intensity will decrease with design storm duration, but the effective rainfall expressed as a proportion of the total rainfall will *increase* with storm duration. Consequently the storm duration which produces the highest flood peak will have a duration greater than the catchment response time.

The shape and position of the runoff curves (Fig 4.6(b)) will depend on the initial accumulated precipitation depth required to initiate runoff (p_3 and p_4) and the associated times t_3 and t_4, as well as the precipitation depths required to satisfy the soil moisture deficit and storage requirements (p_5 and p_6).

A good example is the Port Elizabeth storm described in Annexure 3B to Chapter 3. Small urban catchments in Port Elizabeth with short catchment response times received immediately antecedant precipitation which not only satisfied the soil moisture deficit of the non-paved areas but also filled the streams and drainage systems. Rain which occurred towards the end of the storm fell on a catchment fully charged with water and with strongly flowing streams.

At the other extreme are large catchments in semiarid regions. Because these catchments have areas much larger than the area of rainfall produced by single storms, the average moisture status of the catchment can only be increased appreciably by multiple storm events prior to the storms which eventually cause the severe flooding.

The 1988 floods in the Vaal-Orange River system are an example (see Annexure 3G to Chapter 3). Fig 4.6(b) can also be used to visualize this situation using the same reasoning that was used in the Port Elizabeth example except that the time and precipitation scales are greatly enlarged. The total precipitation depth built up over a number of months before the occurrence of the storms which eventually produced the severe flooding. By this time the catchment moisture status was very high over a large area with the result that the effective precipitation from these storms was much higher than the long term 'average' for the region.

In high rainfall areas the catchment moisture status does not vary greatly from the average condition. However, in semiarid and arid regions the soils vary from very dry most of the time to a high moisture status after rare seasons of prolonged rains. The effect of the assumption of average soil moisture status is independent that the flood magnitude-frequency relationship will be discussed in the Chapter 7 on deterministic methods.

4.5.3 The method used in HRU 1/72 (Midgley, 1972)

Figs 4.7 and 4.8 from HRU 1/72 are the methods proposed in that report for determining the effective storm rainfall (storm runoff) as a percentage of the total storm rainfall. The mean values which are recommended for use when determining the flood magnitude-frequency relationship (Fig 4.7) are expressed as a function of veld type zone and catchment size, while the maximum envelope values are given as a function of catchment size only (Fig 4.8).

The authors of HRU 1/72 stressed that due to the paucity of available data, the relationships should be considered as rough guides only, and that they must be tempered with engineering judgment.

They noted that for catchments up to about 500 km² runoffs representing practically 100% of the input have been experienced, but that this declines to about half of the total storm rainfall for 100 000 km² catchments (Fig 4.8).

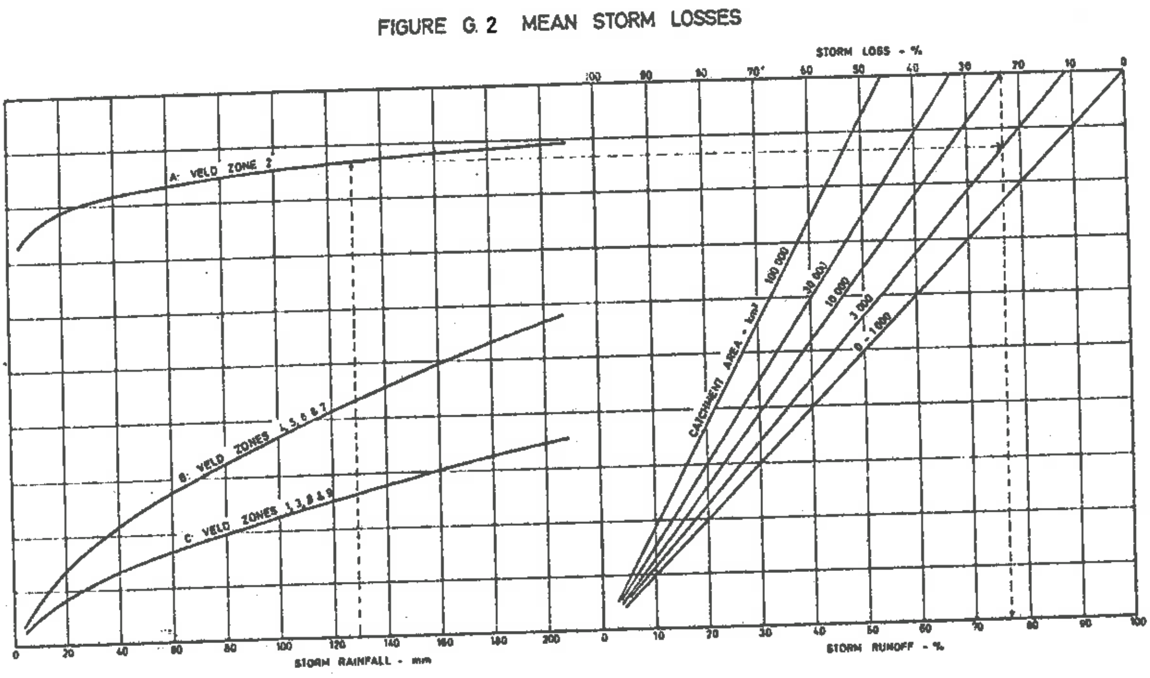


FIGURE 4.7 Mean storm losses. Fig G2 in HRU 1/72

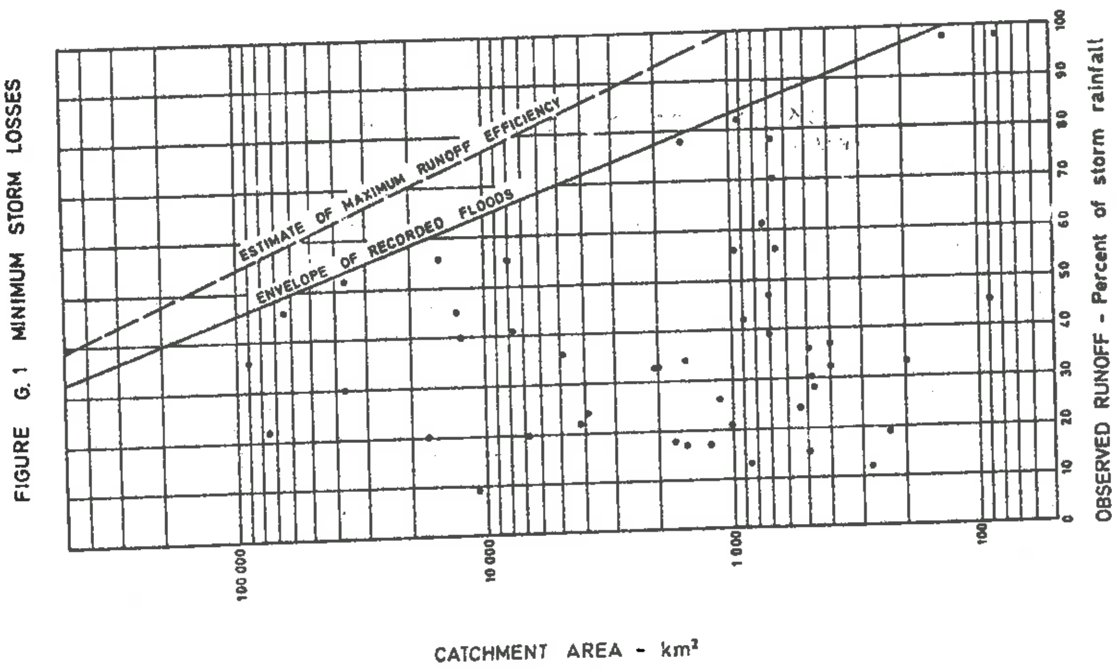


FIGURE 4.8 Minimum storm losses. Fig G1 in HRU 1/72

4.5.4 Department of Water Affairs' technical reports

The recent technical reports issued by the Department of Water Affairs on flood surveys undertaken after severe floods provide a valuable source of information that was not available to the authors of HRU 1/72. These are summarized in chronological sequence.

The Pretoria storm of January, 1978 (Kovács 1978)

(See also Appendix 3C to Chapter 3).

The effective rainfall varied within the range of 15% to 53% of total storm rainfall. The six highest values are given in Table 4.2.

TABLE 4.2 : Details of floods resulting from the Pretoria storm of January 1978

River or dam	Catchment area km^2	Flood peak m^3/s	Total catchment rainfall mm	Effective rainfall %
Pienaars River	1 028	1 105	94	53
Hartbeesspruit	161	520	145	47
Roodeplaat Dam	684	1 510	110	45
Apies River	676	678	122	37
Crocodile River	148	142	110	35
Klipvoor Dam	6 138	497	74	25

Parts of these catchments lie within the Pretoria municipal area. Forty percent of the Crocodile River catchment is dolomitic.

The January 1981 floods in the south western Cape (Kovács 1981)

(See Appendix 3E to Chapter 3).

In Table 4.3 the runoffs are expressed as percentages of the 3-day rainfall. These figures cannot be related to that portion of the 3-day rainfall that contributed directly to the flood hydrograph other than noting that the percentage effective rainfalls must have been higher than the quoted figures.

TABLE 4.3 Details of floods resulting from the January, 1981 storms in the south western Cape

River or Dam	Catchment area km²	Flood peak m³/s	Total catchment rainfall mm	Effective rainfall %
Wilgehout River	25	250	175	84
Stetynskloof Dam	55	137	110	81
Rooikloof River	11	30	85	72
Hartbees River	13	24	110	71
Korinte-Vet Dam	37	76	157	69
Poesjenels River	14	47	130	64
Houtbaais River	25	12	70	63
Keerom Dam	377	>50	215	60
Duivenhoks Dam	148	>194	170	60
Roode Elsberg Dam	139	93	110	57
Prins River Dam	757	>1 030	159	49
Molenaars River	113	111	70	46
Buffelsjagt Dam	601	>297	150	42
Keisies River	78	120	141	40
Theewaterskloof Dam	497	438	150	39
Floriskraal Dam	4 001	5 740	142	26

Floriskraal Dam's effective rainfall of 26% puts it further down the list. However, this is the 3-day rainfall and not the much shorter period rainfall that contributed to the flood peak. The effective rainfall percentages shown in the last column are underestimates of the percentage rainfall which contributed directly to the flood peaks in most of the catchments.

The dam inflow volumes are generally more accurate than estimated volumes derived from stage records at gauging stations. However, the values of the instantaneous flood peaks are less reliable, and only estimated minimum peaks can be determined for small capacity dams.

Unfortunately some of the largest floods could not be included because the gauging stations were destroyed. These included sites along the Buffels River downstream of Floriskraal Dam and along the Touws, Groot and Gouritz Rivers.

Kovàcs noted that the catchments in the region needed between 30 and 50 mm storm rainfall to produce runoff; and that the effect of antecedant conditions diminished with increasing rainfall depth and became insignificant for storm rainfalls exceeding 200 mm. He also commented that steep surfaces tend to minimize the importance of antecedant wetness on the runoff process.

The March-May 1981 floods in the south eastern Cape (du Plessis 1984)

Flooding in the southern Cape moved eastwards after the January floods in the south western Cape described in the previous section.

Fig 4.9 shows the direct runoff expressed as a percentage of storm rainfall for all 87 sites where flood hydrographs were available for the Jan-May 1981 storms (ie including the storms in the south western Cape described in the previous section). The direct runoff approached 100% of the storm rainfall at a number of stations where the total storm rainfall exceeded 150 mm. Du Plessis noted that a minimum of 5 mm of rain was needed to produce runoff and that this increased to 80 mm in small afforested catchments.

Du Plessis also compared the flood volumes with the mean annual runoff (MAR). The average for all 87 gauged hydrographs was 52% of the MAR. The runoff from the 24 largest floods was 86% of the MAR and the mean storm rainfall 25% of the mean annual rainfall.

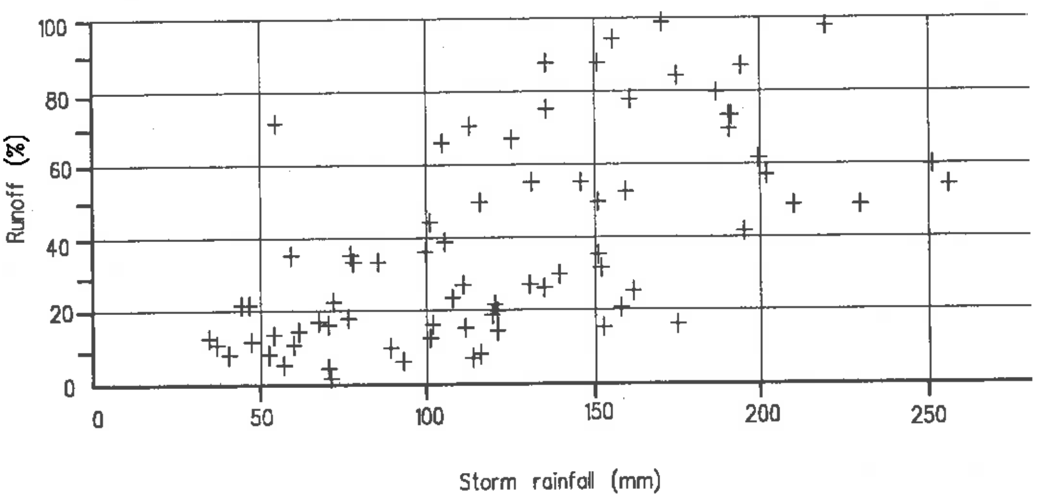


FIGURE 4.9 Direct runoff as a percentage of total storm rainfall at 87 stations in the south western and south eastern Cape from January to May 1981

The information in Table 4.4 has been abstracted from du Plessis' report to demonstrate the succession of severe floods that occurred at a number of sites during the three month period March to May of 1981. The first three examples are small, steep coastal catchments, whereas Gamkapoort Dam's catchment is located in the arid Great Karoo and the other two dams in the semiarid Little Karoo.

TABLE 4.4 Details of floods resulting from the March - May 1981 floods in the south eastern Cape.

River or dam	Catchment area km²	Month	Flood peak m³/s	Total catchment rainfall mm	Effective rainfall %
Groot Brak River	131	March April May	131 275 430	120 115 135	21 51 90
Karatara River	22	March April May	110 42 195	112 220 151	96 72 105*
Gouna River	91	March April May	200 130 310	150 154 170	51 34 119*
Gamkapoort Dam	17 076	Jan March	3 700 5 700	109 126	18 18
Stompdrif Dam	5 235	March April May	847 430 414	61 37 47	11 11 12
Kammanasie Dam	1 505	March April May	205 267 806	89 105 100	10 19 44

* These overestimates are probably due to errors in estimating catchment rainfall over the small catchments.

The 1984 Domoina floods (Kovács et al 1984) (See also Appendix 3D to Chapter 3)

In this report the authors computed the flood volume as the volume between the time of sudden rise of the hydrograph and a fixed time after the peak calculated from the equation

$$t = 0{,}8\ A^{0{,}2}\ \text{days}$$

where A is the area of the catchment in km².

Care was taken to exclude those parts of the hydrograph generated by post-Domoina rains.

The eleven highest direct runoff percentages are shown in Table 4.5.

TABLE 4.5 : Details of floods resulting from the tropical cyclone Domoina, January 1984

River or Dam	Catchment area km²	Flood peak m³/s	Total catchment rainfall mm	Effective rainfall %
Bloed River	543	1 200	385	75
Klipfontein Dam	340	1 090	450	67
Pongolapoort Dam	7 831	13 000	570	47
Slang River	676	400	250	43
Black Imbuluzi River	723	1 673	480	40
Mhlatuze River	1 136	2 400	480	39
Hluhluwe Dam	734	2 940	450	38
Westoe Dam	531	180	180	35
Jericho Dam	218	65	175	34
Goedertrouw Dam	1 273	1 900	360	33
Komati River	7 330	3 630	255	27

The catchments are appreciably larger than most of the catchments in the south western and south eastern Cape, and the total rainfall depths are also larger.

The total storm rainfalls are plotted against percentage direct runoff in Fig 4.10. The three curves drawn in the figure by the authors demonstrate the large influence of antecedant catchment wetness and vegetal cover on the percentage direct runoff. These three curves correspond approximately to the following conditions :-

Line I — The 14-day antecedent rainfall (AP_{14}) was generally 50 to 100 mm. This is more or less equivalent to average January conditions in the area. The characteristic vegetation cover was grassveld. The run-off percentages along the line are slightly higher than given by Fig 4.7.

Line II — AP_{14} was generally 20 to 50 mm. The vegetation was mainly bush and grassveld. This line seems to be fairly representative for most sites. Note that under the particular catchment wetness conditions approximately 50 mm storm rainfall was needed to start run-off.

Line III AP$_{14}$ was variable, but generally less than 50 mm. The predominant vegetation in catchments 66, 69, 70 and 71 which plot nearest to the line was forest plantations or orchards. The storm loss in these catchments was very high and apparently about 100 mm storm rainfall was needed to start run-off.

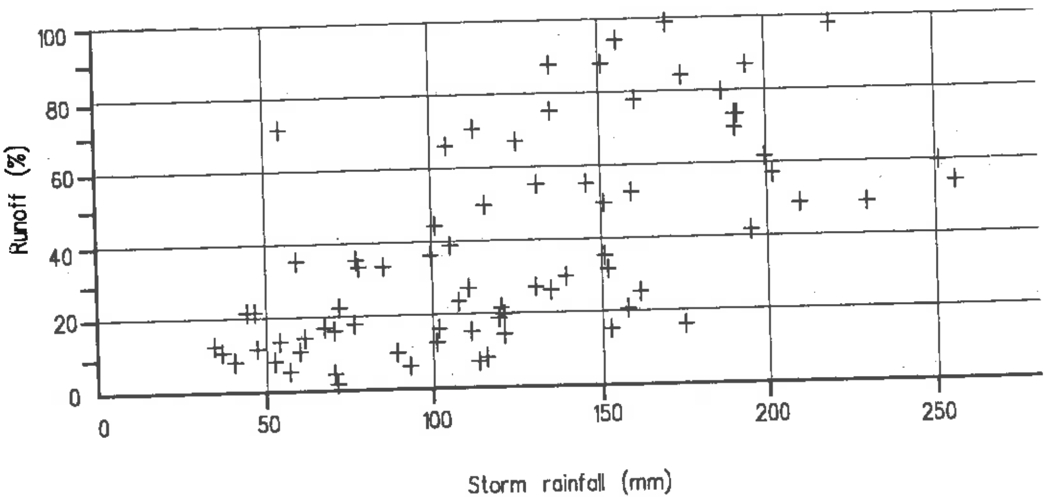


FIGURE 4.10 Percentage storm rainfall vs percentage direct runoff for the tropical cyclone Domoina.

Summary

This information from the Department of Water Affairs' flood documentation reports is valuable but cannot be used directly in flood magnitude-frequency estimation methods. This is because the proportions of the total storm rainfalls that generated the observed peak flows cannot be identified. All that can be assumed is that the effective rainfall percentages are underestimates of the actual values.

Secondly, the magnitude-probability relationship cannot be determined from this data alone, but would require studies to be made of the probabilities of occurrence and relationships between causative storm rainfall and effective storm rainfall for all the floods in a flow record.

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Chapter 5

DIRECT STATISTICAL ANALYSIS METHODS

Contents

5.1 INTRODUCTION

The three principal flood frequency estimation methods are direct statistical analysis of routinely gauged flood data (discussed in this chapter), deterministic methods where the statistical properties of the flood are assumed to be the same as the storm rainfall which is then used in a conceptual rainfall-runoff model (Chapter 7), and empirical methods which are based on historical maxima and have no statistical or theoretical basis (Chapter 8).

Of these, the direct statistical analysis of the flood data is obviously the preferred method because all of the factors which contributed to the magnitude of the flood are accounted for in the data. However, the application of the method is not without difficulty, particularly where records are short and there are anomalies in the data. The inclusion of historic information prior or subsequent to the gauged record can increase confidence in the results.

5.1.1 Statistical analysis

All statistical analyses require estimates to be made of the statistical properties of the recorded data set, and a decision on the most appropriate probability distribution function. Once anomalies in the data sets have been corrected the calculations are relatively simple and will seldom take longer than an hour for hand calculation methods making use of the tables in Chapter 13, or a few minutes on a microcomputer using the computer programs accompanying this handbook. The exceptions are the calculations for the Wakeby distribution and regional analyses neither of which are amenable to hand calculation.

The uncertainties related to statistical analysis arise from errors and gaps in the data, errors in the assumed statistical properties of the data set due to the short records from which they are derived, and the choice of the most suitable distribution function.

A still greater uncertainty arises where there are no records at the site and estimates of the distribution parameter values have to be based on a combination of multivariate analysis techniques and regional values.

5.2 HISTORIC INFORMATION, MISSING DATA, AND OUTLIERS

The general approach described below is a modification of that used in the Canadian guidelines (Pilon *et al*, 1985) which in turn were partially based on the USA guidelines (Interagency Advisory Committee on Water Data, 1982).

5.2.1 Inclusion of historic information

If information is available which indicates that any flood peaks that occurred before, during or after the systematic record are maximum values experienced over an extended period of time, or if it is known that peaks exceeding the highest gauged peak did not occur in a period prior to the establishment of the gauging station, then the period of analysis should be increased to incorporate this information. Before such data are used their reliability should be evaluated by checking against similar extremes in nearby catchments. A fairly large error in the estimation of the value of the peak can be tolerated. The decision that the user has to make is the extent to which possible measurement errors may reduce the value of this additional information. One way of assessing this is to place subjective upper and lower bounds on the estimated value and compare the effects that they have on the results.

Historical weighting makes use of historic information outside the period during which gauged records are available. The two components of the total period covered by the analysis are the gauged period and the historic period during which one or more years containing flood peaks of known magnitude occurred, or alternatively during which it is known that no peaks exceeded the maximum observed value during the gauged period.

The underlying assumption in the adjustment used to accommodate historic data is that the data from the systematic record are representative of the intervening period between the systematic and historic record lengths. As a rule, historic information should be included in the analysis unless there are strong grounds for its exclusion.

The USA guidelines use the historically weighted conventional moments method for incorporating historical data, while the Canadian publications use both conventional and maximum likelihood moment estimation methods. They recommend the latter, but conventional and probability weighted moments are recommended in this handbook due to difficulties encountered in maximum likelihood procedures.

5.2.2 Years with zero flow or low outliers

Gauged data may contain zero flows and/or low values which may be considered to be anomalous values due to the effects of upstream utilization such as storage dams and agricultural practices.

Zero flows cannot be included in analyses using log transformed data. Anomalous low values will reduce the calculated skewness of the data set and so result in an underestimation of the calculated flood peaks.

Zero flows as well as anomalous low flows can be accommodated in the statistical analysis by making use of conditional probability. A low threshold value is specified which can be zero or some higher value that will have the effect of excluding all undesirable low outliers. The observations exceeding this value are counted and expressed as a ratio of the total number of observations used in the analysis. This is the assumed probability that a future peak will exceed the specified low threshold. For a specified flood magnitude, the exceedance probability is the product of the probability that it will exceed the low threshold value, and the exceedance probability derived from the rest of the data set.

The exclusion of non-zero values creates theoretical difficulties when frequency distributions are based on the balance of the data set, and retrofitting procedures have to be used where these are available.

5.2.3 Derivation of parameters used in the equations

The determination of the values of the parameters used in the equations is best illustrated by way of an example.

Assume that a gauged record is available for the 40-year period from October, 1940 to September, 1980. During three years of the record there was no flow in the river, and during another two years there was flow in the river but the gauging station was not in operation.

Three severe floods occurred between 1924 and 1940 from which peak discharges can be determined. During the period 1980 to 1988 there were no peaks which exceeded the lowest of the three peaks between 1924 and 1940. During the gauged period two peaks were higher than the lowest of the three peaks during the ungauged period.

To complicate matters still further, four low outliers can be clearly identified when data are plotted on an appropriate log-probability scale graph.

This is a highly unlikely combination of circumstances, but it serves to illustrate the calculation method described below. All of the units are numbers of years during which the events occurred.

Total period (1924 - 1988) = 64 years

The high threshold is the *lowest* peak observed during the ungauged historic period, or the highest observed value where there are no historical maxima greater than this value. The assumption is made there were no unidentified flood peaks in the historic period whose values exceeded that of the high threshold.

The low threshold is zero in those data sets containing zero flows, or the lowest peak where there are no years with zero flows, or some higher value arbitrarily selected by the user.

The years with missing data within the gauged period are added to the number of years with missing data within the historic period. The assumption is that the statistical properties of all missing data are the same as those of the set of available observations.

The ranked observations from highest to lowest and the period during which they occurred are as follows :-

1 Prior to 1940
2 Prior to 1940

3 1940 - 1980
4 1940 - 1980
5 Prior to 1940 [This is the high threshold value]

\- -

58 Identified low outlier [This is the low threshold value]
59 Identified low outlier
60 Identified low outlier
61 Identified low outlier
62 Zero flow
63 Zero flow
64 Zero flow

Gauged record

Period 1940-1980	= 40
Missing data	= 2
Zero flows	= 3
Peaks greater than zero but less than the low threshold	= 4
Peaks equal to or greater than the high threshold	= 2
Peaks between threshold values including missing data [40 - 2 - 2]	= 36

Earlier and/or later periods containing historic peaks

Period 1924-1940 and 1980-1988	= 24
Peaks equal to or greater than the high threshold	= 3
Missing data [(16 - 3) + 8]	= 21

Parameters used in the equations

YT	= total time span (1924 to 1988)	= 64
NA	= floods equal to or above the high threshold	= 5
NB	= floods between high and low thresholds	= 36
NC	= missing data	= 23

[Note: YT = NA + NB + NC]

LW	= low outliers including zero flows	= 7
ZR	= zero flows	= 3
WT	= weight applied to data	

$$WT = (YT - NA)/NB \tag{5.1}$$

Calculation of historically weighted mean $\overline{Q}_h$, standard deviation s_h and skewness coefficient g_h :-

$$\overline{Q}_h = (WT \sum x_b + \sum x_a) / (YT - WT.LW) \tag{5.2}$$

$$s_h = [(WT \sum d_b^2 + \sum d_a^2) / (YT - WT.LW - 1)]^{0,5} \tag{5.3}$$

$$g_h = [(YT - WT.LW) (LW \sum d_b^3 + \sum d_a^3) / s^3] / [(YT.WK - 1)(YT - WT.LW - 2)] \tag{5.4}$$

where

x_a is the value of a peak equal to or above the high threshold

x_b is the value of a peak below the high threshold

d_a and d_b are deviations of $x_a + x_b$ from $\overline{Q}_h$

all values being the logarithms of the data.

These historically weighted values of the mean, standard deviation and skewness coefficient are then used in the equations for the LN/MM, LP3/MM, EV1/MM and GEV/MM distributions in the usual way.

5.2.4 The conditional probability adjustment

Assume that in a sample of N observations, L observations are less than the specified low threshold. Let n be the number of observations excluding the L observations. Assuming that the L observations are all zeros, the probability of exceeding x is given by

$$P(x) = [1 - F(x)] [(N - L)/N] \tag{5.5}$$

where $$F(x) = 1 - 1/T; \tag{5.6}$$

from which

$$P(x) = [1/T] [(N - L)/N)] \tag{5.7}$$

This equation can be substituted into the equations for obtaining Q_T for the various distributions, but this will give a probability distribution function which is not truly that of the assumed distribution. Further adjustments are therefore necessary.

At return periods greater than two years the conditional probability function can be simulated almost exactly by mathematically retrofitting the lognormal and GEV distributions, while the procedure used for the log Pearson 3 distribution is satisfactory for skewness coefficients within the range +2,5 to -2,0. No retrofitting algorithm for the Wakeby distribution has been found.

Details of the adjustments are described under the sections dealing with the different distributions below.

5.3 DIRECT STATISTICAL ANALYSIS

5.3.1 The *W-T* relationship

It is the hydrological extremes - flood peaks and flood hydrographs - that are of particular interest in hydrological analyses, and their values have to be determined by extrapolation using the properties of the historical data.

Linear relationships are the obvious choice for extrapolation, and most of the effort in hydrological analyses is expended in reducing the often awkward mathematical relationships into a linear form. Even where this is not practical, the extent of curvature is not determined directly, but as a measure of departure from a linear relationship. The two forms of the general equations are :

$$y = a + bx \tag{5.8}$$

and

$$y = a + bx \pm cx \tag{5.9}$$

These relationships are illustrated in Fig 5.1.

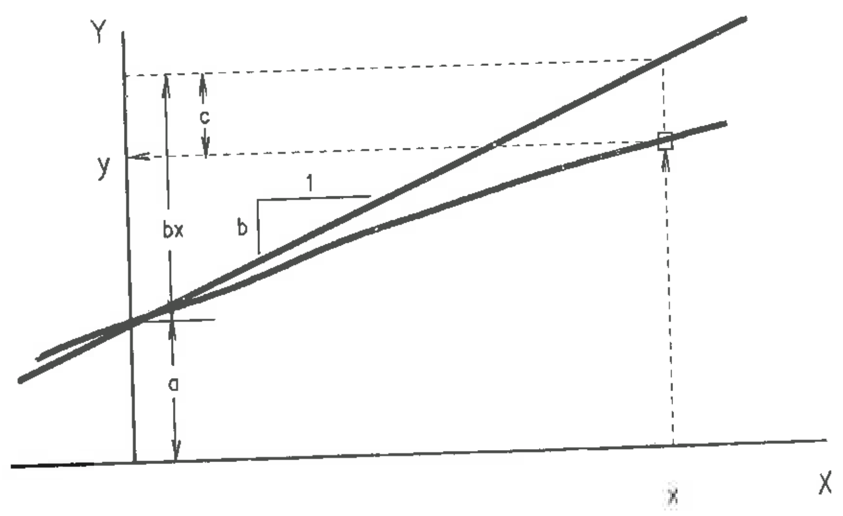

FIGURE 5.1 Graphical illustration of the method for determining the value of the dependent variable *y* from the independent variable *x*.

Where

a is a function of the location parameter of the distribution. This is the mean for the log normal (LN) and log Pearson Type 3 (LP3) distributions, whereas for the general extreme value (GEV) distribution it is the mode which in turn is a function of the mean.

b is the slope of the line and is a function of the scale parameter of the distribution. It is equal to the standard deviation for the LN and LP3 distributions, and a function of the standard deviation for the GEV distribution.

c is a function of the shape parameter of the distribution and can be positive or negative in the case of the skew distributions, or zero in the case of the LN distribution.

There are several methods which can be used for deriving the best estimates of the distribution parameters for a given set of data. The method of moments is the simplest to use. It is the recommended procedure for the LN and LP3 distributions and an alternative for the GEV family of distributions. Using this method, the sample mean, standard deviation and skewness coefficient can be derived from equations 2.2, 2.3 and 2.4 in Chapter 2. Other methods are detailed later.

5.3.2 Standardized variates

Standardized variates (called standardized variables, standardized deviates, or reduced variates in some publications) are measured in units of standard deviations, and are used to dimensionalise the estimated magnitude of the event being calculated. Alternatively the related frequency factor W_T can be used.

b) 13.52 - 13.54

Tables G1 and G2 in Chapter 13 handbook provide values of the frequency factor W_T used in the probability distributions of hydrological interest, while Table G3 provides additional parameter values required for the GEV distribution. Table G4 is an expanded table of the standardized variates for the normal and log normal distributions.

5.4 LOG NORMAL DISTRIBUTION

5.4.1 Equations

This distribution has zero skewness when used with log transformed data. The general form of the equation for estimating Q_T where all the variables are in log space is :

$$Q_T = \overline{Q} + s.W_T \tag{5.10}$$

where Q_T = the required value for return period T

$\overline{Q}$ = the mean of the sample

s = the standard deviation of the sample

W_T = the standardized variate from column 3 of Table G1 in Chapter 13.

5.4.2 Confidence bands

The confidence with which the values of the magnitude-risk relationships are estimated depends on the number of observations contained in the data set. The greater the number of observations the greater the degree of assurance in the results of the analyses, and the narrower the confidence band around them.

The confidence bands are a function of W_T as well as the length of the record N years. The general form of the equation for the log normal distribution is :

$$Q_{T,\alpha} = \overline{Q} + s.(W_T \pm W_\alpha) \tag{5.11}$$

where W_α = the displacement of the confidence band and is read from column 5 or 6 of Table G1 in Chapter 13.

5.4.3 Parameter estimation with historic information

When historic information is available the parameters of the distribution are estimated from equations 5.2 and 5.3 and these are substituted in equation 5.10.

5.4.4 Adjustments when low outliers have been removed from the data set

When zero and/or non-zero low values have been removed from the sample being analysed and the remaining observations are used to determine the distribution parameters via the statistics of the censored sample, the resulting non-exceedance probabilities will not be those of the full data set, and an adjustment has to be made. This is achieved by applying a correction factor (NA+NB) / (NA+NB+LW) to the specified exceedance probabilities derived from the reduced data set, using this to adjust the non-exceedance probability G(y), then proceeding with the calculations as before.

5.5 LOG PEARSON TYPE 3 DISTRIBUTION

5.5.1 Equations

This is a 3-parameter distribution, and the general form of the prediction equation is:-

$$Q_T = \overline{Q} + s.K_T \tag{5.12}$$

where K_T is a function of return period and skewness coefficient g, and is obtained from columns 7 to 17 on Table G1 in Chapter 13. If the skewness is zero, K_T is the same as that for the normal distribution. The frequency factor K_T is interchangeable with the factor W_T in the tables.

5.5.2 Adjustments when low outliers have been removed from the data set

When zero and/or non-zero low values have been removed from the sample being analyzed and the remaining observations used to determine the distribution parameters via the statistics of the censored sample, the resulting probability function will not be a true LP3/MM distribution.

A satisfactory approximation of the true LP3/MM distribution can be obtained by retrofitting a curve through the calculated Q_{100}, Q_{10} and Q_2 values derived from the conditional frequency curve. The following method is based on that given in the USA guidelines (Interagency Advisory Committee on Water Data, 1982).

Determine the Q_{100}, Q_{10} and Q_2 values using the sample statistics derived from the censored sample, then compute the synthetic distribution parameters using the following equations :

$$g_s = -2{,}50 + 3{,}12 \,[\log (Q_{100}/Q_{10})/\log (Q_{10}/Q_2)] \tag{5.13}$$

$$s_s = \log (Q_{100}/Q_{50})/(k_{100} - k_2) \tag{5.14}$$

$$x_s = \log (Q_2) - k_2 \, s_s \tag{5.15}$$

where g_s, s_s and x_s are the synthetic logarithmic skewness coefficient, standard deviation and mean, and k_{100} and k_2 are the LP3 deviates for the 100-year and 2-year return periods respectively, and skewness coefficient g_s. The equation for the estimation of g_s is satisfactory for skewness coefficients of the logarithms within the range of -2,0 to +2,5.

The synthetic parameters are then substituted for the conventionally derived parameters in the equations for this distribution.

5.6 EXTREME VALUE TYPE 1 DISTRIBUTION

5.6.1 Equations

This is a two-parameter distribution with a fixed skewness coefficient of 1,1396 with neither lower nor upper limits. The equations for the mode β, and the scale parameter α are :

$$\alpha = \frac{\sqrt{6}}{\pi} . s \qquad = 0,780 \ s \tag{5.16}$$

$$\beta = \overline{Q} - 0.5772 \ \alpha \qquad = \overline{Q} - 0.450.s \tag{5.17}$$

The general form of the equation for estimating Q_T is:

$$Q_T = \overline{Q} + s \ (0.780 \ W_{y,k} - 0.450) \tag{5.18}$$

where $W_{y,k}$ is the standardized variate from the central column of Table G2 in Chapter 13.

5.6.2 Parameter estimation with historic information

When historic information is available parameters are estimated from equations 5.16 and 5.17 using the historically weighted moments obtained from equations 5.2 and 5.3.

5.6.3 Adjustments when low outliers have been removed from the data set

(See general extreme value distribution below.)

5.7 GENERAL EXTREME VALUE DISTRIBUTION

5.7.1 Equations

This is a family of three-parameter distributions identified by the value of the skewness coefficient. If the skewness coefficient $g \simeq 1,14$ ($k \simeq 0$) then the EV1 distribution is appropriate (Section 5.6 above). Otherwise if $g > 1,14$ use the EV2 (Frechet) distribution, or if $g < 1,14$ use the EV3 (negative Weibull) distribution.

The forms of the equations used for hand calculations can be derived as follows, assuming that the method of moments is used to determine the mean, standard deviation and skewness coefficient. The alternative maximum likelihood (ML) method is not detailed in this handbook as it has been overtaken by the probability weighted moments (PWM) method which is dealt with later.

For the EV2 distribution :

$A = u + \alpha/k$ where $A < u$ is a location parameter at the lower limit,
$B = - \alpha/k$ where $B>0$ is a scale parameter, and
where $k<0$ is a shape parameter which is a function of g.

For the EV3 distribution :

$A = u + \alpha/k$ where $A > u$ is a location parameter at the upper limit
$B = + \alpha/k$ where $B>0$ is a scale parameter, and
where $k>0$ is a shape parameter which is a function of g.

The general form of the equation for estimating Q_T is :

$$Q_T = u + W(y,k) \quad (5.19)$$

where :

$$u = A + B \quad (5.20)$$
$$\alpha = -kB \quad (5.21)$$

and $W_{y,k}$ = standardized variate for the GEV distribution.

The general equation can be expanded by making the following substitutions :

$$A = \overline{Q} - B.E(y) \quad (5.22)$$
$$B = (s^2 / \mathrm{Var(y)})^{0,5} \quad (5.23)$$

The general equation for the EV2 distribution then becomes :

$$Q_T = \overline{Q} + (s^2 / \mathrm{Var(y)})^{0,5} (1 - \mathrm{E(y)} - \mathrm{k.W_{y,k}}) \quad (5.24)$$

and the general equation for the EV3 distribution becomes :

$$Q_T = \overline{Q} + (s^2 / \mathrm{Var(y)})^{0,5} (-1 - \mathrm{E(y)} + \mathrm{k.W_{y,k}}) \quad (5.25)$$

where $Var(y)$ and k are obtained from Table G3 as a function of the skewness coefficient g, and $W_{y,k}$ is the standardized variate from the right-hand side of Table G2 in Chapter 13.

5.7.2 Parameter estimation with historic information

When historic information is available parameters are estimated from the above equations using the historically weighted moments obtained from equations 5.2, 5.3 and 5.4.

5.7.3 Adjustments when low outliers have been removed from the data set

Note that it is not possible to estimate peaks for return periods T which are less than N/n, nor can the GEV distribution be used to accommodate zero flows in the data.

When historic information is available adjust the moment estimators for this distribution as follows :-

T-year flood peak estimation without low outliers:-

For EV1

$$Q_T = \mu + \alpha \{- \ln [- \ln (1 - 1/T)]\} \tag{5.26}$$

For EV2 and EV3

$$Q_T = \mu - (\alpha/k) \{ [- \ln (1 - 1/T)]^k - 1 \} \tag{5.27}$$

Conditional probability adjustment

From equation (5.7) $P(x) = [1/T] [(N - L)/N]$

for EV1

$$Q_T = \mu + \alpha \{- \ln [- \ln (1- (1/T)(N/(N-L))]\} \tag{5.28}$$

and EV2 and EV3

$$Q_T = \mu - (\alpha/k) \{ [- \ln (1 - (1/T)(N/(N-L)))]^k - 1 \} \tag{5.29}$$

The above equations will give a probability function which is not truly that of distribution, and further adjustments will have to be made.

The procedure is to retrofit synthetic EV1, EV2 and EV3 curves through specific points on the conditional probability curve.

Since the EV1 distribution has two parameters, only two points on the curve are necessary. These are :-

$$\alpha = (Q_{100} - Q_2) / 4{,}23364 \tag{5.30}$$

$$\mu = Q_{100} - 4{,}60015\ \alpha \tag{5.31}$$

For the EV2 and EV3 distributions three points on the curve necessary. To simplify the mathematics the calculated values for return periods T = 1,582, 10,483 and 100 years are used. Then the distribution parameters follow from :

$$\mu = Q_{1,582} \tag{5.32}$$

$$k = - 0{,}21738 \ln [1 + (Q_{1,582} - Q_{100})/Z_1] \tag{5.33}$$

$$d = k\ Z_1 \tag{5.34}$$

where $Z_1 = [(Q_{1,582}.Q_{100} - Q_{10,483}{}^{2}) / (Q_{1,582} + Q_{100} - 2.Q_{10,483})] - Q_{1,582}$ (5.35)

These adjustments are only applicable when all low outliers are non-zero, as the GEV distribution cannot be used to accommodate zero flows.

5.8 WAKEBY DISTRIBUTION

5.8.1 Equations

This distribution has five parameters and consequently requires a large data set for reliable estimates of the parameter values. The general form of the equation is

$$Q_T = m + a \{ 1 - (1 - F_T)^b \} - c \{ 1 - (1 - F_T)^{-d} \} \tag{5.36}$$

where $F_T = 1 - 1/T$

Once the values of the five parameters have been determined it is a simple matter to derive the value of Q_T for any return period T.

5.8.2 Parameter estimation with historic information

Pilon *et al* (1985) describe a least squares regression method for parameter estimation with historic information. Time has not permitted the inclusion of the method in the computer programs which accompany this handbook, and consequently the computer implementation of the Wakeby distribution cannot be used for historic information.

5.8.3 Adjustments when zero flows or low outliers have been removed

When zero flows or low outliers have been removed from the data set, equation 5.36 above has to be adjusted as follows :

$$Q_T = m + a \{ 1 - [nT/(n+L)]^{-b} \} - c \{ 1 - [nT/(n+L)]^{d} \} \tag{5.37}$$

where m,a,b,c and d are estimated from the reduced data set.

5.8.4 Application

The Wakeby distribution requires a more extensive data set for accurate determination of the parameter values, and the results have to be interpreted with caution. The main application is in the determination of regional parameter values, and this is discussed in more detail in the next chapter.

5.9 GRAPHICAL METHOD

It is always sound practice to plot the data as well as the results of statistical analyses on linear and log probability graph paper or the computer screen. In this way any anomalies - particularly station calibration errors and the effect of low or high outliers - will become immediately apparent.

Note that it is the results of the calculations that should be superimposed on the plotted points. Despite the recommendations in some guidelines *under no circumstances should an eyeball fitted curved line be drawn through the plotted data on the graph* as this will almost certainly violate the properties of the assumed distributions.

5.9.1 Introduction

The objective in graphical estimation is to reduce the cumulative distribution function to a linear relationship by adjusting the horizontal scale of the graph. The horizontal scale can be linearized by expressing the exceedance probability (and also the corresponding return period) in units of standard deviations.

The most useful graphical presentations are those with linear horizontal scales in units of standard deviations but calibrated in non-exceedance probabilities as well as return periods. The two distributions used for determining these probabilities are the normal and EV1 distributions. Vertical scales can be linear or logarithmic. The combinations included in the computer programs accompanying this handbook are :

Horizontal scale	Vertical scale
Normal probability	log
EV1 probability	log
EV1 probability	linear

5.9.2 Plotting positions

Rank the data starting with the largest value and give each a rank number starting from one ($m = 1$). Note that some plotting position algorithms start with the lowest value.

Determine the plotting position (return period) for each value from the plotting position formula:

$$T = \frac{N + a}{m - b} \tag{5.37}$$

where :

T	is the return period in years,
N	is the total number of observations,
m	is the rank number of the observed value, and
a and b	are constants

Some of the plotting positions recommended for use in hydrological analyses are given in Table 5.1 :

TABLE 5.1 Some commonly used plotting positions

TYPE	PLOTTING POSITION	DISTRIBUTION	N = 50 m = 1
Weibull (1939)	(N+1) / (m)	normal Pearson 3	51
Blom (1958)	(N+0,25) / (m-0,375)	normal	80,4
Gringorten (1963)	(N+0,12) / (m-0,44)	exponential, EV1, and GEV	89,5
Cunane (1978) = avr of above two	(N+0,2) / (m-0,4)	general purpose	83,7
Beard (1962)	(N+0,4) / (m-0,3)	Pearson 3	72
Greenwood (1979)	N / (m-0,35)	Wakeby, GEV	77

Having calculated the plotting position for each event (say annual flood peak) it is a simple matter to plot the flow versus plotting position.

If the objective of the probability plot is to determine whether a set of data conforms visually to a specified distribution then the appropriate plotting position should be used. If several distributions are plotted on a single graph then either the Weibull or (preferably) the compromise Cunane plotting position should be used.

5.9.3 Adjustments to plotting positions when historic information is used

When historic information is used, these values are included in the data set which now has NA+NB observations within a total range of YT years. Zero flows and low outliers are excluded from the plot.

The ranked positions are used directly for plotting all the values above and equal to the upper threshold. For example, if the Cunane plotting position is used the plotting position T is derived from

$$T = (YT + 0{,}2) / (m - 0{,}4) \tag{5.38}$$

where m is the rank number starting with the largest value, and YT is the total time span.

For the remaining observations the rank number is adjusted as follows :

$$MA = NA + (YT - NA)\,(m - NA) / NB \tag{5.39}$$

Then MA is substituted for m in equation 5.38.

The total number of observations plotted is NA+NB.

5.9.4 Adjustments to fitted frequency curve

It is not necessary to make any adjustments to the plotted position of a frequency curve to accommodate historic information, zero flows or low outliers as the equations used to derive the curves have already been adjusted.

5.9.5 Comparison of results

The fitted curves should also be plotted on probability paper together with the confidence bands for the LN/MM distribution if required.

5.9.6 Examples

Several examples in the case studies in Chapter 14 illustrate how graphical methods can be used to determine the appropriateness of alternative distributions for fitting data sets.

5.10 SOURCES OF ERROR

5.10.1 Introduction

The sources of error most often encountered in the data used for flood peak analyses are :-

Missing data.
Zero, or near zero flows.
Trends in the data due to changes in upstream conditions.
Measurement errors, particularly in the high flow range where the flows exceed the capacity of the gauging station.
Apparent outliers.

5.10.2 Missing data

If data for one or more whole years are missing then reduce the number of observations accordingly and proceed with the analyses as described above.

If data for one or more months in a year are missing use the maximum observed in the balance of the months if these are in the months during which the peak flow normally occurs or ignore the whole year if this is not the case.

The omission of data rests on the assumption that the missing data are distributed about the mean in the same manner as the remaining data. It would therefore be incorrect to omit data associated with zero flows or flows beyond the capacity of the gauging station, both of which require special treatment.

5.10.3 Zero flows

Zero flows for periods of more than a year may occur in arid regions. At other sites it may not have been possible to measure near zero flows due to the siltation of the gauge plates or gauging weir.

Zero values cannot be included in the data being analysed because this will create an error condition on the computer when the log of zero is calculated. If small values are substituted this will distort the analyses.

The difficulty can be resolved by making use of conditional probability theory as described in previous paragraphs.

5.10.4 Corrections for trends in the data

If there are trends in the data these have to be detected, the directions and magnitudes of the trends determined, and the data series adjusted.

The two most likely causes of trends in the flood peak series are the urbanization of small catchments where increases in flood peaks by up to a factor of nine have been reported, and decreases in flood peaks due to other forms of development within a catchment, particularly most agricultural practices and more significantly the construction of storage dams.

If there are significant trends in the data these will introduce positive curvature at the top end of the data (rising trend) or negative curvature at the lower end (decreasing trend), but not both. If one of these anomalies is apparent examine the ranked data to see whether the anomalous results tend to be grouped in recent years. An examination of the histogram of individual values may also provide evidence of a distinct trend (see Fig 2.1(a) and 2.1(d) in Chapter 2 where no trend is evident). As there are other possible causes of similar anomalies, particularly measurement errors, sampling errors (small samples), and mixed distributions, the presence of a trend should only be assumed if the visual tests provide unequivocal evidence of this. Conventional statistically based trend analyses are unlikely to be successful due to the small sample sizes and high degree of variability and skewness in typical annual flood peak series in southern Africa.

Decreasing trends

The construction of numerous small dams or one or more large dams in a catchment will have a major effect on downstream flood peaks in years with low flood peaks. The effect will increase with decrease in the magnitude of the flood peaks and consequently the result will be a decrease in the skewness coefficient of the data set. It is important that these anomalous low results be removed from the data set as they will introduce negative curvature into the analyses and so lead to an underestimation of the long return period peaks for which the analyses are intended.

The full data set should be plotted and the threshold below which all values should be excluded determined by visual inspection. The criterion should be the extent to which the plotted points depart from the straight line fit of the log normal plot. Not more than 25% of the values should be excluded from the analyses.

A worked example is given in the case studies for the gauging station in the Vaal River at Riverton near Kimberley.

Increasing trends
The only situation where increasing trends are likely are in small urbanized catchments. In this situation other methods appropriate to urban catchments should be used in preference to direct statistical analyses.

Apparent increasing trends in large catchments are more likely to be due to sampling or measurement errors, or mixed distributions.

5.10.5 Measurement errors

Many, if not most of the records of flow gauging stations in southern Africa contain years where the flood peaks exceeded the measuring capacity of the gauging weir. Once again the solution is to make use of conditional probability theory but this time G(y) has to be increased by multiplying it by the ratio of total number of years divided by the number of years in which flow was less than the gauging capacity. Thereafter the method is the same as that used for zero flows. However, high outliers should *not* be omitted from the data set unless there is clear evidence that the measurements were in error.

Omitting values at the top end of the range will lead to far greater uncertainties in the results than the omission of flows at the lower end of the data set. Every effort should therefore be made to establish the values of missing peaks. It must be appreciated that the natural variability of the annual flood peaks of South African rivers is very high, with the result that even relatively large measurement errors will not have a proportionally large effect on the results of the analyses. In many cases the inclusion of approximate flood peak values will produce more reliable results than the omission of these values from the data set.

For graphical solutions the value of N used in the plotting position is the total number of years while the value of m for the largest valid flow is the total number of years less the number of years where the flow exceeded the capacity of the weir. Subsequent values of m are decreased by one for each successive value plotted.

5.10.6 Apparent outliers

Outliers are values which plot at an unusual distance away from the trend of the plotted points when the full data set is plotted on probability graph. When this happens the accuracy of the flow measurements of the outliers should be re-assessed. If the accuracy of the flow measurements is within acceptable limits, a decision has to be made on whether or not to omit the apparently anomalous values or make an adjustment for them.

The USA guidelines stress that all procedures for treating outliers ultimately require judgment involving both mathematical and hydrological considerations. A normally distributed one-sided 10% level of significance test is used to identify high and low outliers. The equations derived from equation (5.4) are:-

$$Q_H = \overline{Q} + s.W_N \tag{5.40}$$

$$Q_L = \overline{Q} - s.W_N \tag{5.41}$$

for high and low outliers, where W_N is from tables, and all the other variables have units of base 10 logarithms (see Table G4 in Chapter 13).

Where high outliers are detected (observations with values greater than antilog Q_H) they should be compared with historic flood data in the vicinity. If information is available which indicates that the high outliers are the maximum observed over an extended period of time, then they should be treated as historic data. Otherwise they should be retained as part of the systematic record.

If low outliers are identified (values less than antilog Q_L) they should be removed from the data set and a conditional probability adjustment applied using the same procedure as in the case of zero flows. If an adjustment for historic flood data has previously been made, then the historically adjusted mean standard deviation and skewness coefficient should be substituted in the conditional probability equations.

The USA guidelines consider this procedure to be appropriate for use with the LP3/MM distribution over a range of skewness coefficients from -3 to +3.

There is very strong evidence that in South Africa most of these apparent high outliers are caused by rare, severe meteorological phenomena which result in the annual flood peak series being a mixture of two or more statistical populations, each with different sets of parameter values and consequently different flood peak-frequency relationships.

Because these outliers are few in number in an already statistically small sample, the effects of rare, severe meteorological phenomena are best assessed on a regional basis rather then on an individual station basis. Details are given in Chapter 6.

5.11 REFERENCES

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Chapter 6

REGIONAL STATISTICAL ANALYSIS METHODS

Contents

6.1 INTRODUCTION

The purpose of regional statistical analysis methods is to provide a basis for improving the estimates of the distribution parameters at gauged sites, particularly those with short records, as well as to provide a basis for estimating the flood peak-frequency at ungauged sites.

6.2 SUITABLE PROBABILITY DISTRIBUTIONS

6.2.1 Background

The following points must be stressed before discussing candidate distributions for regional statistical analyses.

(a) The flood-frequency relationship is a function of the joint probability of the following:

   (i) The depth, area, duration, and movement of the storm rainfall, which in turn are functions of the types of meteorological conditions which gave rise to them (orographic, convective, convergence, frontal, tropical cyclones, etc.)

   (ii) The conditions in the catchment immediately prior to the storm. These antecedent conditions include the soil moisture status, river flow, vegetal cover, and the content of storage dams in the catchment.

   According to ven te Chow (1964) this justifies the use of the log normal distribution which describes the product of a large number of independent variables, although in practice the number of variables is not sufficiently large, they carry different weights, and a degree of interdependence is present.

(b) The mathematical solution of a problem involving a number of independent variables requires the estimate of a minimum of three parameter values for each additional variable (the mean, standard deviation and relative weight in the case of the normal distribution). However, the reliability of the estimates of the parameter values derived from a data set decreases rapidly with increase in the number of parameters.

(c) In flood frequency analyses the interest is in the maximum annual value or, in the case of partial duration series, the peak values over a selected threshold. The annual maximum is therefore one of several peaks that occurred during the year. If the number of occurrences during the year is large and the distribution of these events is such that the tail of the distribution decays exponentially, then the family of extreme value distributions can be applied to the annual maxima series. Both of these conditions are only approximately valid in southern Africa. The first condition of a large number of within-year maxima is not valid in semiarid or arid regions where long periods of zero flow are experienced, and where these periods may be longer than a year in some cases.

(d) Parameter values have to be estimated before a distribution can be applied. These are the location, dispersion (scale), and shape parameters. In the case of the log normal and EV1 distributions only the first two are used, while three parameters are required for the LP3 and GEV distributions. Five parameters are used in the Wakeby distribution. Various methods are available for estimating these parameter values. The three methods most used in flood hydrology are the method of moments (MM) which is widely used, the maximum likelihood (ML) method which was recommended in the *Flood Studies Report* for the GEV distribution (the MM and ML methods give the same results for the log normal distribution), and the recently introduced probability weighted moments (PWM) method which has been recommended for use in the LP3, GEV, and Wakeby (WAK) distributions. Because of the different possible combinations of distributions and parameter estimation methods, both the distribution and the moment estimators should be used when describing the flood peak-frequency estimation model, eg LN/MM, LP3/MM, LP3/PWM, GEV/ML, GEV/PWM, WAK/PWM, etc.

(e) The accuracy with which these parameters can be estimated decreases with increase in the order of the parameters being estimated but increases with the length of the record. In general, the lengths of data sets available for flood hydrology are too short for acceptable estimates of the skewness coefficients and higher moments. However, if a number of stations in a region are used, the weighted mean of the station and regional skewness coefficients should provide a more reliable estimate than that from the single station alone. This is the basis for the regional estimation procedures adopted in the USA and UK guidelines, but the basic assumptions in this procedure must be considered. These are :

(i) That there is no spatial correlation between the stations used in the analysis. For example the flows recorded at two stations close together on a river will be highly correlated and it would be incorrect to include both in a regional analysis where all stations are given equal weight. This difficulty can be overcome to some extent by carrying out multivariate analyses.

(ii) That the region within which the stations are located is hydrologically homogeneous. This implies that it is subject to the same range of meteorological conditions and the physical characteristics of the catchments are similar.

(iii) Therefore, the larger the number of *uncorrelated* data sets from a *hydrologically homogeneous* region, the more reliable the estimates of the parameter values and consequently the more reliable the estimate of the flood peak-frequency relationship for return periods well beyond the lengths of the records. It should be noted however, that most of the severe floods in southern Africa are caused by widespread storms which are likely to cover most of the region, and consequently there will nearly always be some degree of correlation between the records from the stations within the region.

6.3 SITUATION IN SOUTHERN AFRICA

6.3.1 Flood frequency estimation at ungauged sites

Early empirical-probabilistic formulae had the form :

$$Q_T = C\,K_T\,A^m \tag{6.1}$$

where

Q_T is the flood for the desired return period T in years.
C is an independent catchment coefficient.
K_T is a constant derived from the assumed distribution function for the required return period T.
A is the area of the catchment.
m is a constant.

Three formulae of this type have been proposed for South African conditions.

6.3.2 Roberts Method

The method proposed by Roberts (1963 and 1965) was widely used for many years by the Department of Water Affairs and other organizations.

The general formula is :

$$Q_T = C\,K_T\,A^{0,5} \tag{6.2}$$

where :

Q_T is the T-year flood peak;
C is a catchment coefficient but is unrelated to any measurable catchment characteristic;
K_T is a coefficient derived from the Hazen frequency distribution function for the return period T.

The values for C vary between 0,1 and 2,4.

(Note that Roberts used the symbol K for the coefficient C. These have been transposed here to conform with conventional usage).

The major objection to the Roberts method is that the factor C (his coefficient K) shows very wide variations from stream to stream and cannot be related to any regional or measured variables. Another weakness is the assumption of single values for the coefficient of variation and skewness coefficient for annual flood peaks for all South African rivers inherent in the Hazen distribution. The average record length of the data used by Roberts has more than doubled since he carried out his analyses and it is no longer necessary to assume these parameter values to be a constant value for the whole of South Africa. The Hazen method is an early version of log normal curve fitting in combination with empirically derived coefficients for fitting skewed distributions (Benson 1968).

This method has given way to the use of other distributions which have a less empirical basis.

6.3.3 Pitman and Midgley method

Pitman and Midgley (1967) used the log EV1 (log Gumbel) distribution to derive K_T, retained the factor $A^{0.5}$, and regionalized the coefficient C. The results were presented in the form of a coaxial diagram in HRU 1/72.

Although the log EV1 distribution was reported to have a sounder theoretical basis, American studies showed it to be less satisfactory than the Hazen, log normal and log Pearson Type 3 distributions (Benson 1968). Roberts (1965) found it to be less suitable than the Hazen method for South African conditions. Adamson (1978 and 1980) also found the log EV1 distribution to be less satisfactory than most other distributions, particularly for long return periods.

As in the case of the Roberts method, the Pitman and Midgley method assumed that the annual flood peak distributions for all South African rivers have the same coefficients of variation and skewness.

6.3.4 Herbst method

Herbst (1968) went one step further by including mean annual precipitation in the formula, but still used the Hazen distribution. His equation had the form:

$$Q_T = C\,A^m\,P^r\,\{\,(\,1 + K_T\,C_v)\,/\,100\,\} \qquad (6.3)$$

where :

C, m and r are constants;
A is the area of the catchment;
P is the MAP,
K_T was derived from the Hazen distribution for return period T;
C_v the coefficient of variation where short records were available.

This formula is much more flexible than those of Roberts or Pitman and Midgley.

6.3.5 Subsequent developments in South Africa

Unfortunately, this method was not developed further in South Africa probably due to the availability of the more tractable deterministic methods.

6.4 USA GUIDELINES

6.4.1 Introduction

In 1982 the Hydrology Committee of the US Interagency Committee on Water Data published the revised Bulletin 17B on *Guidelines for determining flood flow frequency* (referred to as the USA guidelines below). The original guidelines published in 1967 recommended the use of the log Pearson Type 3 (LP3) distribution using regionally derived skewness parameters as the standard method for flood frequency estimation by all Federal agencies in the USA.

The study on which the recommendation was based included :

(a) A review of the literature and current practice to select candidate methods and procedures for testing.

(b) Selection of long record station data of natural streamflows in the USA and development of data management and analysis computer programs for testing alternate procedures.

(c) Testing eight basic statistical methods for frequency analysis including alternative distributions and fitting techniques.

(d) Testing of alternative criteria for managing outliers.

(e) Testing of procedures for treating stations with zero-flow years.

(f) Testing relationships between annual maximum and partial duration series.

(g) Testing of expected probability adjustment.

(h) Testing to determine if flood data exhibit consistent long-term trends.

(i) Recommendations with regard to each procedure tested and development of background material for guidelines.

In all, 300 stations having essentially unregulated flows were used in the testing. Record length exceeded 30 years with most stations having records longer than 40 years. The stations were selected to give the best feasible coverage of catchment size and geographic location and to include a substantial number of stations with no flow for an entire year.

The following frequency distributions were tested.

• Log normal	(2 parameters)
• Extreme value type 1	(2 parameters)
• Log extreme value type 1	(2 parameters)
• Best linear invariant extreme value	(2 parameters)
• Two-parameter gamma	(2 parameters)

- Three parameter gamma (3 parameters)
- Log Pearson type 3 (3 parameters)
- Regional log Pearson type 3 (3 parameters)

Notable exclusions in the light of subsequent developments are :

- The general extreme value family of distributions. (3 parameters)
- Two-component extreme value distribution. (4 parameters)
- Wakeby distribution. (5 parameters)

The conclusions were :

1. Only the single station log normal and regional log Pearson are not greatly biased in estimating future frequencies.

2. The regional log Pearson gives somewhat more consistent results than the single station log normal.

3. For these two methods retaining high outliers in the record is more accurate than methods which give less weight to outliers.

4. For these two methods discarding zero flows and adjusting computed frequencies is slightly superior to the alternative technique.

5. Streamflows as represented by the 300 stations selected for the study are not substantially autocorrelated, thus, records need not be continuous for use in frequency analysis.

The Bulletin was updated and extended in 1976. Revised procedures for weighting station moments were incorporated in Bulletin 17B published in 1981, but the working group that prepared this revision did not address the suitability of the originally recommended distributions.

6.4.2 Regional estimates of the distribution parameters

The standard errors of the estimates (SEE) of the mean, standard deviation and skewness coefficient are functions of the length of the record N and are given by :-

$$\text{SEE}\,(x_0) = s\,/\,N^{0,5} \tag{6.4}$$

$$\text{SEE}\,(s) = s\,/\,(2N)^{0,5} \tag{6.5}$$

$$\text{SEE}\,(g) = [\,\{\,6N(N-1)\,\}\,/\,\{\,(N-2)(N+1)(N+3)\,\}\,]^{0,5} \tag{6.6}$$

There is an appreciable degree of uncertainty related to the estimate of the skewness coefficient which in turn is very sensitive to extreme events at both ends of the data set. The high magnitude peaks are very important as they define the part of the curve which is the base from which the long return period estimates are extrapolated.

The USA guidelines recommend that the skewness coefficients be estimated from at least 40 stations, or all stations within a 160 km radius of the site. The stations should have at least 25 years of record. The skewness coefficients should be plotted on a map at the centroids of the catchment areas. If a pattern is evident isolines should be drawn and the standard deviation of the differences between the observed and plotted values determined. If no pattern is evident then an averaging technique should be used.

In both of these situations the standard error of the estimate of the regional skewness coefficient should be determined. To do this the difference between the calculated skewness coefficient at each site and the generalized skewness coefficient at the site determined from the isoline plot or regional average is obtained, and the standard deviation of this set of differences is determined.

When undertaking a flood frequency analysis at a site the standard error of the estimate of the skewness coefficient of the data set is determined. If this is greater than that of the regional value at the one sided 5% level of significance then the regional value should be used in preference to that from the data set.

An alternative weighting method was recommended in the earlier guidelines. For records between 25 and 100 years in length the station skew is given a weight of (N-25)/75 and the regional skew a weight of 1-(N-25)/75. For record lengths of less than 25 years attempts should be made to extend the record by correlation with nearby stations. Failing this, the regional skew should be used and the results accepted with caution.

In Bulletin 17B the 25/75 method was omitted in favour of a skew prediction equation based on various catchment parameters. It is doubtful whether this approach will be successful in South Africa due to our greater spatial variability of flood causative conditions which preclude the identification and quantification of these parameters. The 25/75 weighting procedure has therefore been retained as the method recommended in this handbook, and is used in the accompanying computer program.

In the map of regional skew coefficients (logs) supplied with the Bulletin, these coefficients varied from +0,7 to -0,4 with about a quarter of the continent having skewness coefficients between +0,1 and -0,1.

6.4.3 Subsequent criticism of the USA guidelines

Subsequent criticism of the principal recommendation of the USA guidelines that the LP3 be used by all Federal agencies has come primarily from the research community (Matalas *et al* 1975, Wallis *et al* 1977, Landwehr *et al* 1978, Wallis and Wood 1985 and others). In the abstract of their 1985 paper *Relative accuracy of log Pearson III procedures* Wallis and Wood stated that :

> *The U.S. Water Resources Council (WRC) has suggested that the log-Pearson III distribution, fitted by the method of moments, should be used in flood frequency analysis. A Monte Carlo simulation assessment of the WRC procedures shows that the flood quantile estimates obtainable by these procedures are poorer than those obtainable by using an index flood type approach with either a generalized extreme value distribution or a Wakeby distribution fitted by probability weighted moments. It is suggested that the justification for using the WRC Bulletin 17B guidelines is in need of re-evaluation.*

The authors ended with the comment :

> *The Water Resources Council recognized that the U.S. national guidelines would need to be periodically re-evaluated and updated in light of future research findings. Given the results presented herein and related research from other studies -----, we suggest that the current national guidelines now be reassessed. In the interim, it is suggested that the design engineers could avail themselves of the permissible, and legal, option of substituting a more accurate and consistent methodology. <u>In fact, given the weight of current evidence failure to do so might well be considered by a court as construing professional negligence.</u>*

The underlining in the previous paragraph is mine. This is a very serious statement from authors of standing in the research community and highlights the gap between research and practice, as well as the dilemma faced by those responsible for preparing guidelines where the objective when recommending a method is to achieve a balance between accuracy, consistency and cost of carrying out alternative procedures. This is an issue that will be raised again in Chapter 7 on the design flood.

Most of the criticism of the LP3 procedures has been based on the analysis of very large synthetic data sets all of which rest on the fundamental assumption that the real world data sets constituted samples from a single parent population ie that each observation is independent of other observations and that all observations are identically distributed. There is ample theoretical and observational evidence in South Africa that this is not so, and consequently the basis for much of the criticism is unfounded.

The general extreme value and Wakeby distributions recommended by the authors listed above as well as methods for generating synthetic data sets are detailed in this handbook. Users are in a position to carry out their own studies along these lines should they wish to do so.

6.5 UK FLOOD STUDIES REPORT (1975)

6.5.1 Introduction

The *Flood Studies Report* (National Environmental Research Council, 1975) is the most comprehensive study of flood frequency analysis methods undertaken to date. The major objective was to develop a method which could be used to determine the flood peak-frequency relationship at any site in the UK. The first phase was the development of a method for the estimation of the mean annual flood through the application of multivariate analyses. The second was the development of a method for determining the flood peak-frequency relationship. The GEV distribution was selected for the study. The data sets were scaled with respect to the mean (= 1,0) and plotted on EV1-linear scale. Best fits were determined, and regionalized flood peak-frequency growth curves for each region obtained. For any site the recommended procedure was to determine the mean annual flood peak using the correlation model and then use this value to dimensionalise the flood peak-frequency relationship.

6.5.2 Statistical correlation models

The conversion of precipitation into runoff is a complex process which involves a large number of variables. Those having a major influence are catchment size and slope which are easily quantified. Others such as vegetal cover are not readily measurable.

The choice of catchment characteristics is dealt with exhaustively in the *Flood Studies Report*, where the contributions that the selected variables make to explaining the variance of the mean annual flood were evaluated by using principal component analysis and multiple regression techniques.

The final set of independent variables used was :

AREA	Catchment area in km^2.
STMFRQ	The number of stream junctions as shown on the 1:25 000 map, divided by the catchment area (this is a measure of the drainage channel density of the catchment).
S1085	The stream channel slope measured between two points 10% and 85% of the main stream length from the catchment outlet to the watershed expressed in metres per kilometre.
SOIL	A soil index with values of the range 0,15 to 0,50.
RSMD	A net 1-day rainfall of 5 year return period (1-day rainfall less weighted mean soil-moisture deficit).
LAKE	The proportion of the catchment draining through lakes or reservoirs.
URBAN	Urban proportion of the catchment.

A typical equation had the form :

$$\overline{Q} = 0{,}0213\ AREA^{0,94}\ \mathrm{STMFRQ}^{0,27}\ \mathrm{S1085}^{0,16}\ \mathrm{SOIL}^{1,23}\ \mathrm{RSMD}^{1,03}\ (1+LAKE)^{-0,85} \quad (6.7)$$

where $\overline{Q}$ is the mean annual flood, from which Q_T, flood peak of return period T, is derived.

This is one of the methods recommended in the *Flood Studies Report*. The methodology and some of the parameters may be applicable to South African conditions but this still has to be tested.

This short summary cannot do justice to the study. The *Flood Studies Report* is strongly recommended for further reading on this subject.

6.5.3 Subsequent developments

Hosking *et al* 1985 recommended that probability weighted moments be substituted for maximum likelihood and quantile methods for estimating the parameters of the general extreme value distribution.

> *Currently favoured methods of estimation of the parameters and quantiles of the distribution are Jenkinson's method of sextiles and the method of maximum likelihood ---. Neither method is completely satisfactory. The justification of the maximum likelihood approach is based on large sample theory, and there has been little assessment of the performance of the method when applied to small or moderate samples; whereas the sextile method involves an inherent arbitrariness ----- , requires interpolation in a table of values of a function in order to estimate the shape parameter of the distribution, and has statistical properties that are not known even for large samples.*

The authors concluded :

> *Estimators of parameters and quantiles of the GEV distribution have been derived using the method of probability weighted moments. These estimators have several advantages over existing methods of estimation. They are fast and straightforward to compute and always yield feasible values for the estimated parameters. The biases of the estimators are small. except when estimating quantiles in the extreme tails of the GEV distribution, and they decrease rapidly as the sample size increases. The standard deviations of the PWM estimators are comparable with those of the maximum likelihood estimators for moderate sample sizes (n = 50,100) and are often substantially less than those of the maximum likelihood estimators for small samples (n = 15,25).*

In this handbook conventional as well as probability weighted moment estimators are detailed and included in the computer programs in preference to maximum likelihood methods.

6.6 BASIS FOR REGIONAL ANALYSIS

6.6.1 Model selection

The choices for the flood peak-frequency estimation models relate to the selection of :

(a) The distribution (LN, LP3, EV1, GEV or WAK).
(b) The parameter estimation method (MM, ML or PWM).
(c) Untransformed or log-transformed data.
(d) Plotting positions.

All of these have received attention in the various published guidelines and research papers.

6.6.2 Evaluation methods

Both the USA and UK methods were based on an examination of historical records. As the true population distributions or mixtures of distributions within a region are unknown and the parameter estimates have wide error bands, the absolute accuracy of candidate models could not be determined.

Recent developments have concentrated on the generation of large data sets of synthetic records from selected distributions and consequently known *Q-T* relationships. Estimation models were then tested using data samples equivalent in length to those found in practice.

If a model performed consistently better than other models the assumption was made that it will also perform better when applied to observed data. This conclusion does not necessarily follow as it does not resolve the question of the true nature of the parent population, and an unavoidable measure of uncertainty remains.

6.6.3 Evaluation criteria

Hosking, Wallis and Wood (1985) and Wallis and Wood (1985) calculated the flood peak-frequency relationship for T=20, 50, 100, 500 and 1000 for each sample set at each site, for a number of repetitions representative of USA and UK conditions respectively. All generated data and parameter values were scaled with respect to the mean (=1,0) for each data set.

Four statistical indices were calculated for each return period for each site. These were the average bias (arithmetic difference), average root mean square error, and the 90% confidence band (5% on each tail).

The "accuracy" of a model is not the accuracy with which it fits past events, but the accuracy of the prediction of the flood peak-frequency relationship for future events. Obviously, the greater the number of parameters in a model the better the fit to historical data. Equally, the greater the number of parameters the less reliable the estimate of their values and consequently the more they are likely to change with increasing length of record at one site or from site to site for the same period of record.

One desirable property of a model is that the parameter values should be stable and not subject to fluctuations over a period of time.

Houghton (1978) made the following observations when recommending the use of the Wakeby distribution :

> *The Wakeby distribution has five parameters, a significant increase from the two or three in standard distributions. There must be good reason for introducing a new distribution, particularly if it absorbs more degrees of freedom than distributions currently in use. The instability of higher moments and of their functions, such as the coefficient of skew, is well known. They often add more noise than signal to estimation procedures for conventional distributions. Although the Wakeby distribution has five parameters, neither the higher sampling moments nor even the sampling variance is used to estimate those parameters. Hydrologists and engineers in past years have occasionally felt the need to go beyond three parameters, but it was recognized that the use of moments higher than the third would introduce too much error into the estimation process. The estimation procedure developed for the Wakeby distribution circumvents this problem.*
>
> *In traditional estimation procedures the smallest observations can have a substantial effect on the right-hand side (large observations) of the distribution. But the left-hand side (small observations) does not necessarily add information to an estimate of quantile on the right-hand side. Indeed, since floods are not known to follow any particular distribution, it seems intuitively better to divorce the left-hand side from the right. It will be shown that the Wakeby does exactly that.*

6.6.4 Conclusions

The principal recommendations and conclusions of the researchers who generated and analyzed large data sets during the decade following the publication of the USA and UK guidelines are given below (Landwehr *et al*, 1978, Hosking and Wallis, 1985, Wallis and Wood, 1985).

(a) That the regional WAK/PWM and GEV/ML models be recommended in the USA and UK guidelines respectively.

(b) The relative performances of six models reported by Wallis and Wood (1985) in order of preference were:-

   (i) Regional WAK/PWM. Near zero bias and a narrow confidence band over the whole range of return periods and record lengths.

   (ii) Regional GEV/PWM. Near zero bias for $T<100$ but increasing thereafter. Narrow confidence band over the whole range of T.

   (iii) LP3/MM with regionally weighted skew and LP3/MM with regional skew only. Both have confidence bands which increase with increasing return periods, and both are markedly poorer than (i) and (ii).

   (iv) Single site LP3/MM and GEV/PWM models were both poor performers with very wide confidence bands for $T>50$ expanding approximately exponentially with T.

(c) Another contender not yet tested on a large scale is the four-parameter, two component extreme value (TCEV) distribution using L-moments.

It should be noted that other investigators have not yet contested these conclusions, nor have the two organizations which published the guidelines amended their procedures. The investigations were supported by the UK Institute of Hydrology which was closely involved in the production of the *Flood Studies Report.*

6.7 RECOMMENDED REGIONAL STATISTICAL ANALYSIS METHOD

6.7.1 General

No comprehensive studies of regional statistical analysis methods have been undertaken in South Africa over the past twenty years and no reliable estimates of regional values are available. Therefore a fair amount of time and effort will be required for this analysis although the computer programs accompanying this handbook are designed for this purpose. Situations where regional direct statistical analysis methods are justified are detailed in Chapter 9 on the design flood, and examples are given in Chapter 14.

The following procedure is recommended.

1. Identify the boundaries of the region where reasonably homogeneous conditions can be expected.

2. Identify suitable flow gauging stations within this area which have record lengths of at least 10 years from the Division of Hydrology Publication No 12 (list of hydrological gauging stations).

3. Abstract annual flood peak maxima from the records obtained from the Division of Hydrology and request comments on the accuracy of the measurements at high stages, and the possibility of patching gaps in the record.

4. Dams with continuous records of water storage levels are another source of data, but the time and effort required to derive the annual peak series from this data will be appreciable.

5. If one or more complete years of data are missing then reduce the number of observations accordingly and proceed with the analysis.

6. If one or more months in a year are missing accept the maximum observation if the remaining months cover the period when the annual peak usually occurs otherwise reject the whole year.

7. Load the data into a suitable data file that can be accessed when running the computer program.

8. Develop a computer program that will perform the following analyses on the data from each site within a region and calculate the regional parameters.

9. Correct for zero flows, low and high outliers, and accommodate historical data.

10. Rank the data and calculate the mean, standard deviation and skewness coefficients of logarithms (base 10).

11. Calculate the three MM and five PWM parameters from each data set. The ML parameter values can also be calculated if required (not included in the accompanying program).

12. Plot the data using log *vs* normal probability scales and the Cunane plotting position.

13. Draw the calculated fitted lines for the various distributions on the graph.

14. Examine the plot for anomalies. If upstream dams command a large proportion of the catchment, this would be a valid reason for excluding obvious low outliers. High outliers should be retained. A data plot that exhibits a noticeable double curvature ('S'-shape), flat sections, or discontinuities would indicate possible rating curve errors which should be corrected, or if this is not possible the station should be rejected.

15. While analytical methods for determining the grouping of stations within hydrologically homogeneous regions have been developed overseas, these require a denser network of stations than is available over most of South Africa. The USA guidelines recommend plotting the skewness coefficients at the centroids of the catchments on a map of the region and determining if obvious spatial trends are present, in which case isolines of the skewness coefficient can be drawn to improve the estimate of regional skewness coefficients. However, single high outliers will have an appreciable effect on the

calculated skewness coefficient and this should be taken into account in the mapping. An alternative method for grouping stations is to follow the procedure in the *Flood Studies Report* of using scaled *Q-T* growth curves which can be plotted on a single graph. The *Q-T* relationships can be compared visually as well as analytically by quantifying the dispersion (standard deviation) about the mean *Q* for each *T*. The analytical version of this procedure has been followed by several recent investigators to test the validity of the various flood frequency models. In practical applications it has the further advantage that it can be used to determine confidence bands about the regional *Q-T* relationship. This method can be applied to any *Q-T* estimation model.

16. Reject any stations which have growth curves which are clearly inconsistent with the rest and recalculate the regional parameter values using the mean of the station values weighted linearly according to their length.

6.7.2 Determination of the flood peak-frequency relationship at a gauged site

Three options are available :

(a) Use the parameter values derived directly from the data set for the site.

(b) Use the dimensionalised regional parameter values only.

(c) Use a weighted average of the station and dimensionalised regional values. The following weighting method is recommended. For records between 25 and 100 years in length the station parameter values are given a weight of (N-25)/75 and the regional parameters a weight of 1-(N-25)/75. For record lengths of less than 25 years attempts could be made to extend the record by correlation with nearby stations or by the inclusion of historical data. Both methods may introduce further uncertainties and should be used with caution. Failing this the regional parameter values should be used. Once the parameter values have been derived the Q-T relationship can be determined using the single station procedures described in Chapter 5 or by using the computer programs.

6.7.3 Determination of the Q-T relationship at an ungauged site.

The mean annual peak flow at the site has to be estimated and then used to dimensionalise the regional parameter values. The variables used in the UK guidelines for determining the mean annual peak at an ungauged site were :

(a) The area of the catchment in km^2. This is the dominant variable and is easily measured. Internal drainage areas should be excluded.

(b) The number of stream junctions as shown on 1:50 000 maps divided by the area of the catchment. This can be determined but will be time consuming.

(c) The slope of the main stream measured between two points 10% and 85% of the main stream length measured from the site to the watershed expressed in metres per kilometre. This is readily determined.

(d) A soil type index. This is not available in the same form in South Africa, but alternative permeability indices could be used.

(e) One-day, five-year return period rainfall less weighted mean soil moisture deficit. This is not available in South Africa but the one-day, two-year return period rainfall has been found useful in estimates of storm rainfall and is the recommended alternative. It can be obtained from the Department of Water Affairs Technical Report TR 102.

(f) The proportion of the catchment occupied by lakes. This is not relevant in South Africa.

(g) The proportion of the catchment occupied by urban areas. This can be ignored for large catchments. Direct statistical analysis methods should not be attempted in urbanized catchments.

The five relevant variables for this analysis can be determined for South African conditions. Other possible contenders are listed in the Department of Water Affairs' Flood Studies Note No 6 (Petras and du Plessis, 1987).

Conventional multiple linear regression analysis using the logarithms of values should provide an acceptable equation for the determination of the mean annual flood at any gauged site within the region being investigated, after which the Q-T relationship can be determined using the single station procedures described in Chapter 5 or the computer programs.

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Chapter 7

DETERMINISTIC METHODS

Contents

7.1 INTRODUCTION

Deterministic methods used for flood frequency estimation are those where the flood magnitude is derived from an estimate of the catchment rainfall for the required annual exceedance probability (AEP). Catchment characteristics including the shape, slope, vegetal cover, as well as the antecedant moisture status of the catchment, and antecedant river flow all play a role in the conversion of rainfall into runoff. Chapters 3 and 4 describe the properties of storm rainfall and catchment processes respectively. These are summarized briefly below.

7.1.1 Storm rainfall

The properties of storm rainfall of interest to the hydrologist are the areal extent of the precipitation, the depth of the rainfall over this area (from which the volume can be derived), the duration of the rainfall (from which the intensity can be derived), and the frequency of occurrence of the precipitation (or more strictly the probability that a specified storm property will be exceeded during a specified length of time - conventionally one year).

These properties cannot be determined from a knowledge of the physical process within a storm, and have to be inferred from records at rain gauges at fixed points on the ground or from weather radar observations.

Several storm types are illustrated in the annexures to Chapter 3 where it is shown that these vary from short duration, high intensity storms, through tropical cyclones to prolonged seasonal rainfall.

7.1.2 Hydrograph shape

The shape of the flood hydrograph depends on the physical characteristics of the catchment, and the time and space distribution of the storm rainfall. The greater the size of the catchment, the greater the variety of time and space distribution properties of storm rainfall that will produce the annual flood maxima, and consequently the greater the variability of hydrograph shapes for a specific catchment.

Storms with durations longer than the catchment response time will produce hydrographs with longer time bases, and consequently greater volumes than short duration storms.

Multiple successive storms will produce compound hydrograph shapes such as those illustrated in the annexures to Chapter 3.

Prolonged seasonal rainfall will result in antecedant river flow and will also increase the groundwater contribution to river flow. A series of two or more large floods may occur successively as was the case with the Orange River floods during the 1987-88 summer season.

7.1.3 Antecedant conditions

The state of wetness of the catchment immediately prior to the onset of the rainfall which produces the flood plays an important role in determining the proportion of the storm rainfall that contributes directly to the flood hydrograph, and consequently to the magnitude of the flood. For a specific storm, a wet, impervious catchment will obviously produce a larger flood than a dry, pervious catchment. However, if the rainfall prior to the storm is such that the moisture content of the catchment surface is raised to the point where infiltration and storage losses are minimal, the difference between the effects of the two catchments will be considerably less.

The flood magnitude is therefore a function of the properties of the storm rainfall and as well as the antecedant moisture status of catchment.

In the case of the Nelspruit, Port Elizabeth and Pretoria storms described in Chapter 3 the moisture status of their catchments immediately prior to the storms was unrelated to the meteorological conditions which caused the storms. However, the moisture status of small catchments with catchment response times appreciably less than the storm duration would have increased appreciably during the early part of the storm and approached near saturation towards the end of the storm.

The larger the area of a catchment, and the greater the aridity of the climate, the greater the effect of antecedant conditions on the flood magnitude. For large catchments in the dry interior of Southern Africa, several months of above average rainfall are necessary to raise the moisture status to a condition where subsequent storms result in major floods. This was the situation during the 1987-88 summer season.

The antecedant catchment moisture is a result of the combined effect of antecedant rainfall and antecedant evaporation losses. While rainfall and potential evaporation can be measured, actual evaporation losses over a catchment can neither be measured nor calculated.

7.1.4 Point vs areal rainfall

The claimed advantage of the deterministic methods is that the rainfall records on which they are based are far more numerous, longer, and are considerably less prone to measurement error than runoff records. However, the interest is in rainfall over the catchment and consequently the statistics of *storm* rainfall as different from *point* rainfall. Whereas point rainfall only has two dimensions - depth and duration - storm rainfall has the additional dimension of area. There is a measure of validity in the assumption that estimates of the depth - duration - frequency (DDF) relationship of point rainfall are more reliable than flood peak - frequency estimates for the reasons given above. However, it does not follow that the full depth - area - duration - frequency (DADF) relationship of storm rainfall has the same degree of reliability. It is clear from the discussions in Chapter 3 on storm rainfall and a study of the storms described in the annexures to the chapter that while there may be a usable *average* relationship between point and areal rainfall, there is a very wide band variability around this relationship. As there are no available studies of the depth - area - duration relationships of storm rainfall based on the statistical analysis of observations of *a large number of annual storm maxima*, these properties have to be derived from point rainfall and extrapolated to catchment rainfall. Consequently there are a number of uncertainties in the derivation of the area and time distribution of rainfall over a catchment in addition to the uncertainties related to the rainfall - runoff conversion processes which greatly reduce the validity of the initial assumption.

7.1.5 Rainfall-runoff model structure

In essence, rainfall-runoff model structure consists of modules which determine the catchment response time, the catchment rainfall intensity that corresponds with the response time, and the proportion of the catchment rainfall that contributes directly to the result.

The art of hydrological modelling lies in the choice of model parameters and their calibration. The parameters can be purely empirical and calibrated on the basis of experience alone, such as in the simplest version of the rational method. At the other end of the scale are models with parameters that are based on the physical processes in the system, and where mathematical procedures are used for model calibration.

As will be seen, it is the simplest, experience based models which are overwhelmingly used in practice, while complex models are notably absent. It is important that reasons be sought for this situation before discussing available models and possible improvements to them.

From the user's point of view a rainfall-runoff model must meet the following requirements.

1. It must provide accurate estimates.
2. It must provide consistent results ie different users should get the same answers when applying the method to the same problem.
3. It must be applicable to a wide variety of situations likely to be encountered in practice, and not produce anomalous results in unusual situations.
4. It must have wide usage indicating general acceptance in practice.

Whereas simple models will often provide satisfactory results when the user has to rely on his own experience when deciding on parameter values to be used in the method, the larger the number of parameters the greater the need for mathematical calibration of the model.

Despite the use of the term *deterministic* to describe these models, the applications are probabilistic in the sense that the answer is the most probable result for a given set of conditions. For a given storm rainfall, the resulting flood magnitude will vary over a wide range depending on the status of a large number of unquantifiable variables at the onset of that portion of the storm rainfall that results in the flood. Consequently it is not possible to calibrate the model using a few observed storm rainfalls and the resulting floods. Successful calibration requires a record with a large number of annual maximum floods from which the average parameter values can be determined. Conversely the model cannot be tested by comparing the theoretical and observed flood magnitudes produced by a known storm rainfall. For any one storm the calculated parameter values may differ significantly from the model parameter values, but for a large number of storms the average of the observed parameter values should be the same as those used in the model.

The relatively small number of observations of annual flood maxima at any one site inhibits the use of a model with a large number of parameters where the values are determined by calibration. This is because the accuracy of the model parameters decreases rapidly with the increase in the number of the parameters. Furthermore, the greater the number of parameters the greater the possibility that the model will not be applicable to situations outside those used for model calibration, and the greater the risk of anomalous results. This has not deterred model developers from incorporating a number of variables in their models and ignoring the need for calibration of the model, or testing its application under a wide range of conditions. This is a dangerous practice.

7.1.6 South African deterministic models

The rational method was developed in Ireland by Mulvaney in 1855 although in the USA Kuichling (1889) is usually given credit for its development. It is still the most commonly used method in the world from Japan in the east, to the USA in the west and South Africa and Australia in the south. It is the preferred method used in the Division of Hydrology of the Department of Water Affairs. It is often erroneously stated that the rational method is limited to small catchments. There are no theoretical or practical grounds to support this contention. The confusion arises from the assumption that the model is based on runoffs generated by observed storms, whereas the calibration, if undertaken at all, is based on a statistical analysis of floods without attempting to relate the model parameters to the storm rainfall which generated the floods (for example Schaake, 1967 and Pilgrim, 1987).

The 1959 floods in southern Natal resulted in the establishment of the University of the Witwatersrand's Hydrological Research Unit (HRU) under the leadership of Professor D.C. Midgley. After a major research programme the HRU produced Report No 4/69 on design flood determination in South Africa. The metricated version Report No 1/72 (Midgley, 1972) was published in 1972 and minor adjustments were made in 1978. The principal method proposed by the HRU is the unit hydrograph method.

The unit hydrograph method was first proposed by Sherman in the USA in 1932. There are a number of modified versions in use in various parts of the world. The HRU version was based on the South African data then available (prior to 1970), and is widely used in practice in South Africa.

These two deterministic methods have much in common as the calculation procedure for each method listed below shows :-

Rational method for a desired annual exceedance probability (return period) :

1. Calculate the time of concentration from catchment characteristics.

2. Determine the point rainfall depth for the desired AEP and a duration equal to the time of concentration.

3. Apply an areal reduction factor to obtain the equivalent average rainfall depth over the catchment.

4. Determine the runoff coefficient, which is that proportion of the storm rainfall that contributes directly to the flood peak.

5. A simple calculation provides the flood peak.

Unit hydrograph method for a desired annual exceedance probability (return period):

1. Calculate the catchment lag time from catchment characteristics. This is required for the determination of the shape of the unit hydrograph.

2. Assume a series of storm durations, and for each duration determine the point rainfall depth for the desired risk and the selected duration.

3. Apply an areal reduction factor to obtain the equivalent rainfall depth over the catchment.

4. Determine the effective rainfall, which is that proportion of the rainfall that contributes to the flood peak.

5. Use an iterative procedure to determine the storm duration which results in the largest flood peak, which is then the required peak for the desired AEP.

7.2 RATIONAL METHOD

7.2.1 General

The rational method is the oldest, and despite some criticism, it is still the most useful and readily applicable method for a wide range of catchments.

Pilgrim (1987) produced some illuminating information on the rational method :

> *"However, 86% of the (Australian) respondents reported use of the rational method, indicating that it is still the dominant design method used in practice, despite what may be implied by research literature. Also, it is still used by the great majority of organizations that employ computer models".*

He quotes similar figures for other countries, including South Africa, Canada, the UK and the USA, and showed that it is also the preferred method for urban and agricultural catchments. He concludes :

> *... it is evident that the rational method is used for the majority of design in economic terms ... (also) ... much of this design is carried out in practice by designers who do not possess great expertise in hydrology".*

The rational method has the simple formula

$$Q_T = 0{,}278\ C_T\ I_T\ A \qquad (7.1)$$

where :

C_T is the runoff coefficient for the specified AEP,
I_T is the precipitation intensity for the specified area, duration and AEP, and
A is the area of the catchment.

The method is theoretically sound for small, impervious catchments such as roofed or paved areas where C has a value of 1,0 and point rainfall values can be assumed. As the size and permeability of the catchment increase, the values for C become more probabilistic than deterministic in their derivation.

Most of the shortcomings of the rational method also apply to a greater or lesser extent to the other deterministic methods such as the unit hydrograph method, which require a great deal more computational effort.

The first step in the rational method is the calculation of the catchment response time, which is the time it takes for the discharge from the catchment to reach a peak value for a constant rate of precipitation. This is called the *time of concentration* and is measured from the beginning of precipitation to peak runoff. The time concentration used in this method should not be confused with the catchment lag used in the unit hydrograph method.

The empirical Bransby-Williams (Williams, 1922) formula is the most widely used formula for calculating the time of concentration (Equation 7.2).

$$t_c = \left[\frac{0{,}87L^2}{1000s} \right]^{\cancel{0{,}36}\ 0{,}385} \qquad (7.2)$$

Flow is assumed to be in the form of sheet flow until the water reaches the collecting channels. In the case of impervious catchments such as paved areas conventional hydraulic flow equations can be used to calculate the time of concentration, but it is usual to use one of the several available empirical equations.

The most effective method for the measurement of the channel slope used in determining the time of concentration has been the subject of investigation. Several methods were discussed in the Flood Studies Report (NERC,1975) and the conclusion was reached that the US Geological Survey method of defining the main stream slope as that between the 10 and 85 percentiles of main stream length was the simplest to determine and gave the best prediction results, despite the theoretically more correct Taylor-Schwarz method which derives an index equivalent to the slope of a uniform channel having the same length as the longest watercourse and an equal time of travel.

Having derived the time of concentration the next step is to determine the maximum precipitation over the catchment for this duration and the required return period. Methods for this determination are given in Chapter 3.

Finally, the rational method requires an estimation of the coefficient *C* which is a measure of the proportion of the precipitation which contributes to the flood peak. Many versions for estimating the value of this coefficient have been published. The procedure used in the Department of Water Affairs version is detailed in Chapter 13.

7.2.2 Experience based methods for estimating C

Experienced users of the rational method often reconstruct the value of *C* by subdividing it into sub-components as in the Department of Water Affairs method described in Chapter 13. This militates against effective calibration of the model but may nevertheless provide satisfactory answers.

The evaporation and transpiration losses are low during a storm, but a proportion of the total rainfall does not reach the river channel due to infiltration into the soil or by damming up behind vegetation, in natural ponds or in artificial dams.

Steep slopes must consist of rocks or cohesive soils to exist as such and can be assumed to be relatively more impervious than flat areas. Similarly the natural pondage will decrease significantly with increase in slope. The slope of the catchment area must therefore have a major influence on the percentage runoff during the time of concentration.

A well developed drainage system will also influence the time of concentration and can either be taken into account when assessing this or corrected in the coefficient of runoff.

An increase in the runoff coefficient with increase in AEP is necessary to accommodate the variation of known effects which also increase with rainfall intensity but are not accounted for in the calculations. These include :

(i) Shortened time of concentration.
(ii) Higher percentage runoff
(iii) Greater possibility of a saturated catchment prior to the storm.
(iv) Validity of the basic assumptions and calculation method.

Also when using the longer return periods, a greater degree of safety in flood estimation is required. The effects of breaches of farm dams and conservation works by widespread floods have also to be considered (cascade effect), as well as possible future changes in the nature of the catchment (eg denudation with consequent increase in runoff, or the construction of storage works, or increase in urban development). Urban development may dramatically increase flood peak values within a small catchment.

7.2.3 Probabilistic methods for estimating C_T

The value of C can also be determined from gauged flood peak records. Direct statistical analysis methods will provide better estimates of the mean annual flood peak than any other method. Given Q_2 and the estimated value of I_2 the corresponding value of C_2 can be determined from equation (7.1). The procedure can be repeated for other annual exceedance probabilities (AEP) and a relationship sought that expresses C_T as a function of the AEP within a hydrologically homogeneous region. Additional variables can be used to improve the estimates of C provided that the data sets are large enough. The advantage of this approach is that the values of C_T are calibrated against calculated Q-T relationships.

7.2.4 Summary

The choice of the value of the coefficient C, which is related to catchment characteristics is largely subjective, but can be improved by comparing it with the values obtained by direct statistical analysis. The values given in section B of Table E1 of HRU 1/72 are far too low. Sokolov *et al* (1976) give a wide range of C values used by different countries of the world.

The Australian guidelines also have graphs of this coefficient.

The peak flow is finally obtained as a product of the runoff coefficient C, the precipitation intensity I, and the area of the catchment A in the appropriate units.

7.3 UNIT HYDROGRAPH METHOD

7.3.1 General

This is another deterministic method and is described in detail in HRU 3/69 and 1/72. It is the method recommended by the HRU for catchments from 10 to 5 000 km² and can be extended to larger catchments by subdividing the catchment into subcatchments each with areas less than 5 000 km², and routing the resultant hydrographs to the site being investigated.

The calculation of the storm rainfall is much the same as that used for the rational method described above and has the same shortcomings.

The basic assumptions in the unit hydrograph method are that provided the effective (excess) rain is substantial - of the order of 10 mm or more -, the principle of superposition or linearity can be assumed to hold and that a unit of effective precipitation, which is that proportion of the precipitation which results in direct runoff, will result in a uniquely shaped unit hydrograph for that catchment.

The method used for deriving standard hydrographs for South Africa is described in HRU 3/69.

Briefly, the steps used were as follows :

1. Some 600 flood events at 96 gauging stations were examined. For each hydrograph the base flow was deducted and the total volume of direct runoff was divided by the area of the catchment to convert it to the equivalent depth of water over the catchment. The hydrograph ordinates were reduced proportionately to produce a hydrograph having unit volume (then one inch, now one millimetre).

2. The durations of the precipitation events producing the floods could not be determined due to the lack of adequate precipitation records. In each case, a storm duration equal to the time to peak was assumed and the corresponding S-curve was derived. The tail of this curve usually hunted about the mean value. The duration was then decreased until this hunting was reduced to within 5% of the mean, and this duration was assumed to be the basin lag.

3. An average basin lag for each station was determined in this way and then expressed as a proportion of the catchment index given by :

$$\left[\frac{L.L_c}{\sqrt{S}} \right]^{0,36} \qquad \Leftarrow\in\leftrightarrows\perp\updownarrow$$

where :

L = Length of longest watercourse (kilometres)

L_c = Distance along the main watercourse to a point opposite to the centroid of the catchment (kilometres).

S = The average channel slope which is the slope of a straight line drawn on the profile of the longest watercourse, which cuts off equal areas above and below it.

The basin lag is an index representing the weighted area of the catchment, by taking both shape and slope into account, and has the dimensions of area.

This relationship was regionalised and a general expression for basin lag was derived.

$$t_l = C_t \left[\frac{L.L_c}{\sqrt{S}} \right]^{0,36} \qquad (7.5)$$

where : t_l = Basin lag in hours
C_t = A regional coefficient which has the dimensions of time per unit area.

4. Once the basin lag was determined, hydrographs of unit volume C_t were converted to corresponding unit hydrographs of unit basin lag t_l by reversing the S-curve procedure.

5. These unit hydrographs were then rendered dimensionless by dividing the time ordinates by the basin lag t_l and the flow ordinates by the hydrograph peak q_p.

6. The mean relationship between q/q_p and t/t_l was determined for each region. This relationship was tabulated and represents a *regionalised, dimensionless one-hour hydrograph of unit effective precipitation.*

7. The area beneath the dimensionless unit hydrograph represents unit volume. The ordinate values must therefore be multiplied by a factor K_u which will depend on the shape of the dimensionless unit hydrograph for the particular region. These factors were also determined and tabulated.

7.3.2 Derivation of the S-curve

The dimensionless unit hydrograph can be dimensionalised by multiplying the time ordinates by the basin lag t_l and the discharge ordinates by the unitgraph peak flow q_p. The result is the *standard unit hydrograph* which represents the direct runoff in cubic metres per second resulting from one millimetre of effective precipitation falling uniformly over the catchment for a period of one hour, and is illustrated in Fig 7.1.

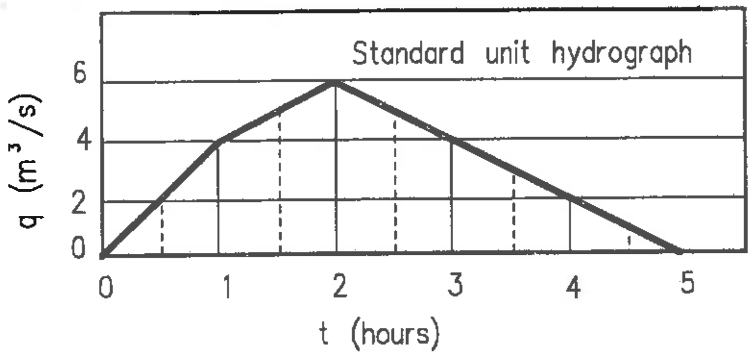

FIGURE 7.1 Standard unit hydrograph

The next step is the derivation of the *standard S-curve* which is defined as the cumulative sum of the hourly ordinates of the standard unit hydrograph, and is a standardized approximation of the hydrograph that would result from a constant effective precipitation rate of one millimetre per hour.

The method is illustrated graphically in Fig 7.2.

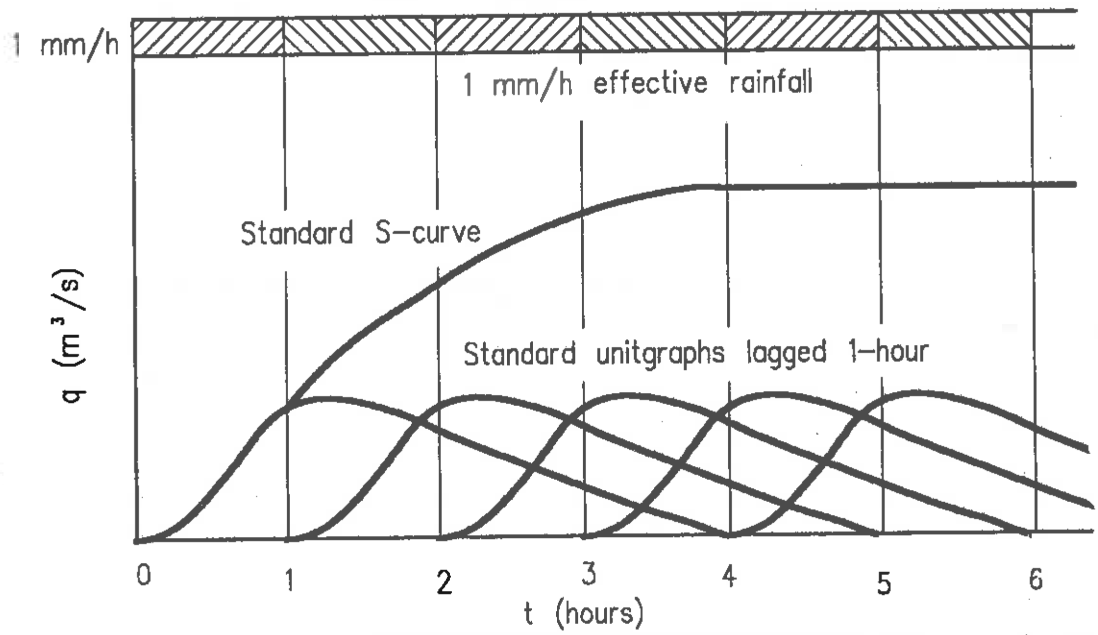

FIGURE 7.2 Graphical derivation of the standard S-curve

The tabulation of this calculation is shown in Table 7.4.

TABLE 7.4 Calculated derivation of the S-curve																	
t =	0		1		2		3		4		5		6		7		8
Unitgraph + 1 h + 1 h + 1 h + 1 h + 1 h etc.	0	2	4 0	5 2	6 4 0	5 5 2	4 6 4 0	3 5 5 2	2 4 6 4 0	1 3 5 5 2	0 2 4 6 4 0	1 3 5 5 2	0 2 4 6 4 0	1 3 5 5 +	0 2 4 6 +	1 3 5 +	0 2 4 +
1 mm/h S-curve	0	2	4	7	10	12	14	15	16	16	16	16	16	16	16	16	16

Alternatively, the standard S-curve can be derived by cumulatively summing the *hourly spaced* ordinates of the standard unit hydrograph, (cumulatively summing every second value horizontally in Table 7.5).

TABLE 7.5 Alternative derivation of the S-curve														
t	=	0	*0,5*	1,0	*1,5*	2,0	*2,5*	3,0	*3,5*	4,0	*4,5*	5,0	*5,5*	6,0
uh	=	0	*2*	4	*5*	6	*5*	4	*3*	2	*1*	0	*0*	0
S	=	0	*2*	4	*7*	10	*12*	14	*15*	16	*16*	16	*16*	16

Where the figures in italics are summed separately ie 0,5 1,0 1,5 etc. Both of the above methods give identical values of the S-curve.

It is important to note that when summing the unit hydrograph ordinates, this must be done in *hourly spaced* sequences. The following methods are *not* correct :

TABLE 7.6 Incorrect derivation of S-curve using half-hour intervals											
t	= 0	0,5	1,0	1,5	2,0	2,5	3,0	3,5	4,0	4,5	5,0
uh	= 0	2	4	5	6	5	4	3	2	1	0
Σ	= 0	2	6	11	17	22	26	29	31	32	32
S	= 0	1,0	3,0	5,5	8,5	11,0	13,0	14,5	15,5	16,0	16,0

TABLE 7.7 Incorrect derivation of S-curve using two-hour intervals						
t	= 0	2	4	6	8	10
uh	= 0	6	2	0	0	0
Σ	= 0	6	8	8	8	8
S	= 0	12	16	16	16	16

Once the S-curve has been derived, unit hydrographs for other durations can be determined by lagging the S-curve with respect to itself, calculating the difference in the corresponding ordinate values and dividing this difference by the lag time. The resulting unit hydrograph represents one millimetre of effective precipitation uniformly distributed over a period equal to the assumed duration. This derivation is shown diagrammatically in Fig 7.3 and in tabular form in Table 7.8 :

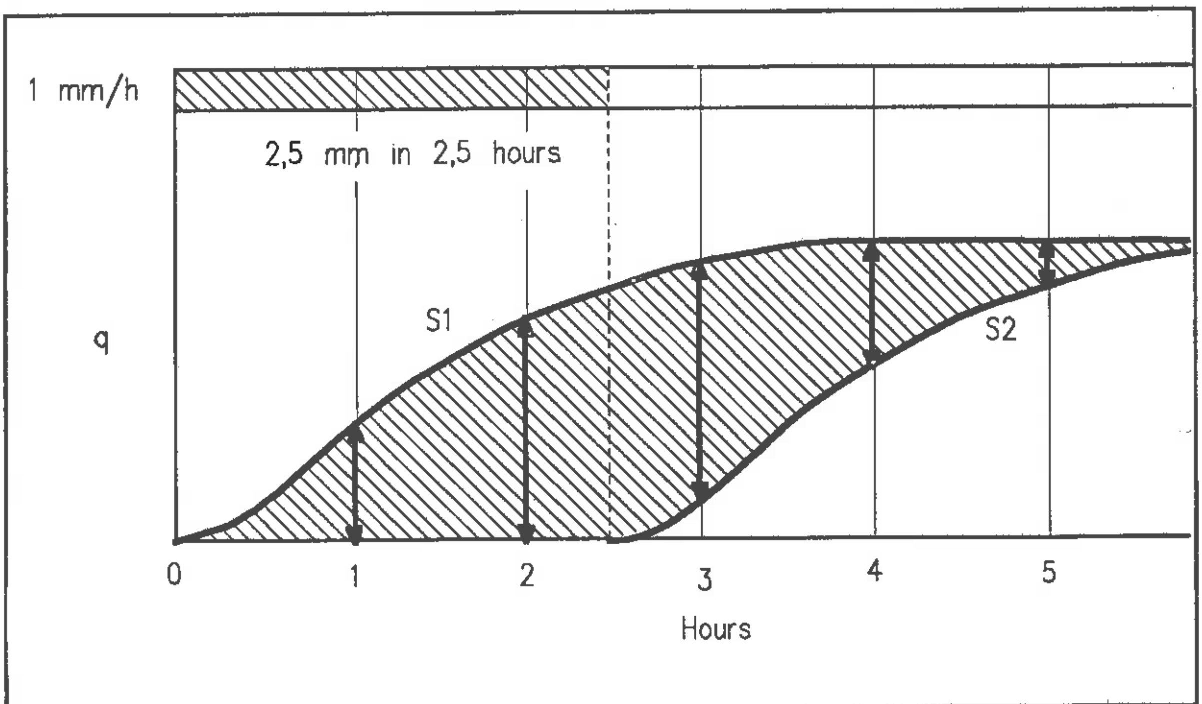

FIGURE 7.3 Derivation of 1mm hydrograph of the required duration

Curve S_1 is the hydrograph which will result from continuous effective precipitation of one millimetre per hour starting at time zero, and S_2 is the identical hydrograph starting 2,5 hours later. The difference represents the hydrograph of one millimetre per hour for 2,5 hours, ie 2,5 millimetres in total. If the ordinates are divided by 2,5, the resulting hydrograph will be that of 1 mm precipitation falling in 2,5 hours.

The tabulation of this calculation is as follows (all hydrograph values in m^3/s) :

TABLE 7.8 Derivation of hydrograph of 1mm in 2,5 hours.												
		1mm/30min			1mm/60min		1mm/90min			1mm/120min		
t	S	0,5S	Δ	2Δ	S	Δ	1,5S	Δ	.67Δ	2S	Δ	.5Δ
						##						
0	0		0	0		0		0	0		0	0
0,5	2	0	2	4		2		2	1,3		2	1
1,0	4	2	2	4	0	4		4	2,7		4	2
1,5	7	4	3	6	2	5	0	7	4,7		7	3
2,0	10	7	3	6	4	6	2	8	5,3	0	10	5
2,5	12	10	2	4	7	5	4	8	5,3	2	10	5
3,0	14	12	2	4	10	4	7	7	4,7	4	10	5
3,5	15	14	1	2	12	3	10	5	3,3	7	8	4
4,0	16	15	1	2	14	2	12	4	2,7	10	6	3
4,5	16	16	0	0	15	1	14	2	1,3	12	4	2
5,0	16	16	0	0	16	0	15	1	0,7	14	2	1
5,5	16	16	0	0	16	##	16	0	0	15	1	0,5
6,0	16	16	0	0	16		16			16	0	0

Note: Column ## is the same as the original dimensionalised unit hydrograph

Having previously calculated the relationship between depth and duration of precipitation for a given area and return period, the estimated precipitation losses can be subtracted from these figures, and the resulting depths of effective precipitation for various durations determined.

Using the same example :

TABLE 7.9 Determination of the design hydrograph peak						
		Duration t				
		0,5 h	1,0 h	1,5 h	2,0 h	
If effective	$P^t{}_T$ =	15	19	21	22	mm
then	Q_P =	15x6	19x6	21x5	22x5	m³/s
	=	90	*114*	111	110	m³/s

Therefore the 1-hour storm produces the highest peak for the given return period. The resulting hydrograph can be obtained by multiplying all the values in the 1mm/1-hour column by 19.

7.3.3 Unit hydrograph calculation procedure

1. The first step is to determine the values of the dimensionalising factors, basin lag and unitgraph peak, by using the following formulae :

$$t_l = C_t \left[\frac{L.L_c}{\sqrt{s}} \right]^{0,36} \tag{7.6}$$

$$q_p = K_u \cdot \frac{A}{t_l} \tag{7.7}$$

When these have been calculated, the relationship between discharge and time can be determined from the tabulated values of t/t_l *vs* q/q_p.

The resulting hydrograph is the standard unit hydrograph which represents the direct runoff in cubic metres per second resulting from 1 mm of effective precipitation falling uniformly for a period of one hour over the catchment.

2. The design hydrographs can be derived more readily from the S-curve than from the unit hydrograph itself. The *S-hydrograph* is defined as the hydrograph that would result from effective precipitation falling continuously at a constant rate over the catchment. The *standard S-curve* is defined as the cumulative sum of the *hourly* ordinates of the standard unit hydrograph, and is a standardized approximation of the hydrograph that would result from constant effective precipitation at 1 mm per hour.

3. Once the S-curve has been derived, the unit hydrographs for other durations can be calculated by lagging the S-curve with respect to itself, calculating the difference in the corresponding ordinates values and dividing the difference by the lagged time.

4. Having previously calculated the relationship between depth and duration of precipitation for a given area and return period, the estimated precipitation losses can be subtracted from these figures, and the resulting depths of effective precipitation for various durations determined.

5. The unit hydrograph ordinates for the various durations are multiplied by the effective precipitation depth for each duration, giving a series of direct runoff hydrographs for the catchment for the given return period. The most unfavourable hydrograph (usually the one with the highest peak) is then used for the design of the structure.

The authors recommend this method for direct use in medium sized catchments from 100 to 5 000 km².

For large catchments, the authors of the method suggested dividing the catchment into smaller segments and then routing the hydrographs from these sub-catchments to the point of interest. This method depends very heavily on the assumed time and areal distribution of the storm rainfall which in turn is based on a series of severe large area storms which are detailed in Appendix D of HRU 1/72.

7.3.4 Probable maximum flood

The procedure is the same as that set out above except that the precipitation depth for the selected range of durations and the effective precipitation are read from the relevant figures from HRU 1/72 which are reproduced in Chapter 13.

7.3.5 Conclusion

Refer to HRU 1/72 for a more complete description of the method, as well as its application to large catchments which require routing procedures.

7.4 LAG-ROUTED HYDROGRAPH METHOD

7.4.1 General

In HRU 1/74, Bauer and Midgley (1974) developed a method where the runoff hydrograph is obtained directly by routing uniformly distributed precipitation through assumed reservoir-type storage using Muskingum routing as modified by Nash. The precipitation input is the same as that derived for the unit hydrograph method, but the resulting hydrograph can be determined with less calculation. The authors feel that this method can be used with caution for catchments of up to 10 000 km^2 provided the catchment shape is not too unusual.

The basic assumption in this method is that the catchment modifies the effective precipitation in two ways when converting the input hyetograph into the output hydrograph. Firstly there is a time lag which is the time taken for the runoff to reach the point of interest, and secondly, the catchment acts as a storage reservoir in that runoff is temporarily stored in the the system and released at a rate which is less than that of the precipitation input. These two properties have the effect of delaying the flood hydrograph as it moves downstream (time lag) and progressively changing the shape of the hydrograph by flattening its peak and lengthening its duration (storage attenuation).

The authors of this method related these two factors to catchment characteristics. They found that the Muskingum routing coefficient *K* which relates to the size of storage was primarily dependent upon the area of catchment and vegetal cover but was relatively independent of catchment shape provided the shape was not too unusual.

The effective precipitation ie that which contributes directly to the flood hydrograph is delayed and its peak is attenuated as it moves through the catchment. This effect can be simulated by routing the effective rainfall through a hypothetical storage system. The storage in the system at any one time is given by

$$S = K\,[x \,.\, inflow + (1 - x) \,.\, outflow\,] \tag{7.8}$$

where x is zero (reservoir type storage was assumed)

and $K = a\,A^{b}$ (7.9)

where $b = 0{,}318$

and a is obtained from Table 3 of HRU 1/74

The volume detained in storage therefore becomes:

$$S = a\;A^{0{,}318} \,.\, \text{outflow} \,. \tag{7.10}$$

where A = the area of the catchment.

If S, the storage, is a function of outflow only, (eg routing through a reservoir) then $x = 0$, while when routing through a uniform channel which has the same inflow and outflow characteristics, $x = 0{,}5$.

The authors assumed that catchment storage was of the reservoir type only, that effective precipitation was the inflow, and the runoff was the outflow. They found that reservoir storage could be represented satisfactorily by a single reservoir and that the Muskingum constants K and x could be determined by analysing the standard unit hydrographs developed in HRU 3/69 (Pullen 1969).

These K-values were then regionalised using the same veld type zones used in the unit hydrograph method, and tabulated in the form of coefficients a and b which have to be applied to the area of the catchment in order to calculate the K-value.

$$K = a\,A^{b} \tag{7.11}$$

This K factor has the dimension of hours. It was found that a constant value for $b = 0{,}318$ could be assumed.

7.4.2 Calculation procedure

The required flood hydrograph is the outflow from this hypothetical storage system and is therefore a function of the effective precipitation and the volume temporarily detained in the storage system. The effective precipitation is divided into time segments and each segment is sequentially routed through the system. The output from the system is a function of the current precipitation segment and the preceding segment as follows :

The Nash-Muskingum routing equation for outflow Q in m^3/s for increments in effective precipitation P (also in m^3/s) as at time m is given by the equation:

$$Q_m = c_0 \cdot P_m + c_1 \cdot P_m + c_2 \cdot Q_{m-1} \tag{7.12}$$

where
$$c_0 = - \frac{K}{\Delta t} (1 - c_2) + 1 \tag{7.12}$$

$$c_1 = \frac{K}{\Delta t} (1 - c_2) - c_2 \tag{7.13}$$

$$c_2 = \frac{1}{e^{\Delta t/k}} \tag{7.14}$$

Given the size of the catchment and the veld type zone, K can be obtained from equation (7.12). If hourly time steps are assumed then $t = 1$ and the values of c_0, c_1 and c_2 can be determined from equations (7.14), (7.15) and (7.16).

For the first hour $m = 1$ in equation (7.13). The direct runoff for time interval $m = 1$ is zero, so the runoff is a function of the precipitation for the first time increment only. Thereafter it is a function of both the effective precipitation for the particular time increment as well as the precipitation and runoff from the preceding time increment. After precipitation has ceased, it is a function of preceding runoff only.

Note that precipitation P over the catchment measured in millimetres can be expressed in m^3/s by applying the following factor :

$$\frac{P}{\Delta t} \cdot 0{,}278\ A \tag{7.15}$$

where t is measured in hours.

7.4.3 Application

While the calculation procedure is simple, the authors caution the users that the decisions regarding the K values to be adopted should be tempered with engineering judgment, particularly with catchments of unusual shape.

The justification given by the authors for this method is that it reduces the time consuming process of unit hydrograph compilation, S-curve derivation and hydrograph superposition.

However, the simplification results in a loss of accuracy in the time resolution (ie hydrograph shape).

If the standard S-curve procedure is used in the Unit Hydrograph method, the calculation should not take more than an hour or two. While the routed hydrograph method may take less than an hour it should not be used in preference to the unit hydrograph method if the shape of the hydrograph is important. *It does have the advantage of giving a quick, reasonable estimate of the flood peak for catchments up to 10 000 km²* when hand calculations are used.

The method used by the authors for determining the *K* value was based on the standard unit hydrographs obtained in the earlier study, so the unit hydrograph and routed hydrograph are not independent methods. This means that the routed hydrograph cannot be used as an independent check of the more time consuming unit hydrograph calculations.

7.5 SIMULATION MODELS

Although Flemming and Franz (1971) strongly recommend simulation models for small catchments, no simulation models which are generally applicable for flood determination for South African catchments have yet been published.

7.6 MODEL COMPONENTS REQUIRING IMPROVEMENT

Many South African practitioners feel uncomfortable when applying the deterministic models because of the lack of calibration procedures used in their development, the high degree of subjectivity involved when applying them, and the absence of large scale studies involving comparisons of the performance of these methods. The computer programs issued with this handbook allow users to compare the results obtained when using these methods with direct statistical analysis methods based on regional analyses of long, reliable records. Users may wish to explore possible improvements to these popular deterministic methods. The obvious model components of these methods where improved algorithms can be sought are those for determining :-

1. Time of concentration or catchment lag.
2. Point precipitation.
3. Area reduction factor.
4. Runoff coefficient or effective rainfall (storm loss allowances).

No recent research has provided the means for deriving better estimates of the time of concentration or basin lag.

Technical Report TR102 (Adamson,1980) published by the Department of Water Affairs provides statistical properties of daily rainfall at some 2 000 rainfall stations in South Africa. This, together with the modified Hershfield equation for short duration rainfall, provides a much sounder basis for determining point precipitation depths than the coaxial plot used in HRU report 1/72 (amended in 1978).

The area reduction factor recommended in this handbook is based on extensive studies in the UK and USA but modified at the short duration end. It provides somewhat higher ARF values than those used in the HRU reports. Kovåcs (1988) found that this was more appropriate than that used by the HRU.

That leaves the runoff coefficient of the rational method and the equivalent effective rainfall (storm losses) of the unit hydrograph method. It is becoming increasingly apparent in South Africa that a major contributor to extreme floods is the wet state of the catchment due to rainfall preceding that which was the direct cause of the flood. The unit hydrograph method makes no provision for the increase in catchment wetness, and consequently decrease in storm losses with increase in AEP.

The authors of HRU 1/72 pointed out that the storm loss algorithms had to be based on rather meagre data. In the alternative algorithms accompanying this handbook, the storm losses are considered to be a function of the AEP which is shown to be a realistic procedure for model calibration.

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Chapter 8

EMPIRICAL METHODS

Contents

8.1 INTRODUCTION

Now that the limitations of both the direct statistical analysis methods as well as the deterministic methods for very long return period estimations (200 years up to the probable maximum flood) are becoming apparent, there is a tendency to have another look at empirical methods based on maximum recorded floods on a world-wide basis.

The two empirical methods for flood magnitude estimation that have been used in South Africa are the Creager method and the regional maximum flood method.

8.2 CREAGER METHOD

Tables of unusually high flood discharges in the USA and other countries were compiled by Jarvis in 1926 and updated by Creager in 1941 (Creager, Justin and Hinds, 1945). The table included only those floods which were necessary to define the upper envelope curve in each region. These authors made the following important observation : *In making use of records of maximum recorded floods on rivers in a given district to estimate the expected peak discharge at a given place, it must be remembered that what has occurred in the past must surely be exceeded in the future.* Consequently it must be recognized that any relationship describing the upper envelope of observed flood maxima will have to be revised from time to time as more data become available.

The Creager formula was used in HRU 1/72 to draw envelope curves through the plots of the maximum observed floods in South Africa against the area of the catchment. (Appendix A of HRU 1/72). The curves were intended to facilitate comparison between computed values and historical maximum flood peaks experienced in catchments of similar size.

The Creager formula is :

$$q = 46\ C\ A\ (\exp(0{,}894A - \exp(-0{,}048)\ -1) \qquad (8.1)$$

where :

- q is the runoff in cusecs per square mile,
- C a coefficient and
- A the area of the catchment in square miles.

The Creager formula has been superseded by the regional maximum flood method and is *not recommended* as a means for determining the design flood for catchments of any size.

8.3 FRANCOU - RODIER METHOD

Francou and Rodier (1967) of the Hydrological Services of the Electricité de France examined some 1 200 maximum recorded floods representing most regions of the world. They noted that when these values were plotted against the catchment areas on a log-log scale, the envelope curves for floods recorded within homogeneous regions could be approximated by straight lines which converged towards a single point at $A = 10^8 km^2$, $Q = 10^6 m^3/s$ where A is the size of the catchment in km^2 and Q is the peak flow in m^3/s.

This family of curves is described by the formula

$$\frac{Q}{Q_0} = \left[\frac{A}{A_0} \right]^{1 - 0,1.K} \tag{8.2}$$

where :

Q is the required peak value in m^3/s

Q_0 has a value of $10^6 m^3/s$ which is the approximate total mean annual runoff of all the rivers of world expressed in m^3/s

A is the area of the catchment in km^2

A_0 has a value of $10^8.km^2$ which is roughly the total drainage area of our globe (excluding deserts and polar regions!)

K is a regional characteristic.

Sokolov, Rantz and Roche (1976) quoted the following values for K :

6,0	Typhoon areas of the Pacific (Korea, Japan,Philippines, Taiwan and southern Texas).
5,6 to 5,5	India, Australia, Central America and Mexico.
5,5 to 5,4	New Zealand, VietNam, Mediterranean basin.
5,2 to 5,0	China, India and Madagascar.
5,0 to 4,8	North Africa and Andes.
4,8 to 4,5	Brazil and Uruguay.
3,5 to 2,0	Europe, with the higher values associated with Alpine streams.
3,0 to 1,0	Tropical Africa.

8.4 WORLD CATALOGUE OF MAXIMUM OBSERVED FLOODS

8.4.1 Catalogue

The International Association of Hydrological Sciences published a world catalogue of maximum observed floods including South Africa (Rodier and Roche, 1984) where the authors confirmed the applicability of the Francou-Rodier equation. The catalogue lists 1 400 floods recorded in 102 countries. Only six of the countries contacted failed to submit data. These were Burma, Angola, Chile, Afghanistan, Zambia and Tanzania.

The catalogue provides additional information including the location of the site, period of record, catchment slope, soil permeability, vegetal cover, climatic regime, mean annual rainfall, mean annual discharge, precision of the measurement and the depth and duration of the antecedant rainfall.

Table 8.1 gives details of 41 of the floods listed in the catalogue, including the 36 which have the highest *K*-values.

The first two values are not considered to be representative; the Amazon because of its unique features, and the Qualême because of doubts related to the method of estimation.

The last five peaks have been included because of their special interest at the two extreme catchment sizes, which confirms the consistency of the method when applied to all likely sizes of catchments and all likely regions of the world.

TABLE 8.1 World maximum floods ranked in order of decreasing K-values from Rodier and Roche, 1984.

Rank	K-value	Country	River	Area km^2	Discharge m^3/s	Year
1	6,76	Brasil	Amazon	4 640 000	370 000	1953
2	6,39	N.Caledonia	Qualéme	330	10 400	1981
3	6,29	Japan	Shingu Oga	2 350	19 025	1959
4	6,22	Taiwan	Cho Sui	259	7 780	1979
5	6,21	India	Narmada	88 000	69 400	1970
6	6,20	Taiwan	Tam Shui	2 110	16 700	1963
7	6,16	Mexico	Cithuatlan	1 370	13 500	1959
8	6,16	Texas	W. Nueces	1 800	15 600	1959
9	6,11	Japan	Nyodo Ino	1 560	13 510	1963
10	6,11	Texas	Pecos	9 100	26 800	1954
11	6,06	India	Macchu	1 900	14 000	1979
12	6,06	N. Korea	Toedong Gang	12 175	29 000	1967
13	6,05	S. Korea	Han Koan	23 880	37 000	1925
14	6,01	Taiwan	Hualien	1 500	11 900	1973
15	5,98	Philippines	Cagayan Echague	4 244	17 550	1959
16	5,92	California	Eel Scotia	8 060	21 300	1964
17	5,91	Japan	Kiso	1 680	11 150	1961
18	5,87	Texas	Pedernales	2 450	12 500	1952
19	5,87	Japan	Tone	5 110	16 900	1947
20	5,87	Texas	Nueces Uvalde	5 504	17 400	1935
21	5,87	China	Hanjiang	41 400	40 000	1583
22	5,86	Mexico	San Bartolo	81	3 000	1976
23	5,84	N.Caledonia	Quinne	143	4 000	1975
24	5,84	Japan	Yoshino Iwasu	3 750	14 470	1974
25	5,84	Australia	Pioneer	1 490	9 840	1918
26	5,83	N. Korea	Daeryong Gang	3 020	13 500	1975
27	5,83	USA	Little Nemaha	549	6 370	1950
28	5,82	Hawaii	Wailua	58	2 470	1963
29	5,81	N.Caledonia	Yaté	435	5 700	1981
30	5,78	Madagascar	Betsiboka	11 800	22 000	1927
31	5,77	California	M.F. American	1 360	8 780	1964
32	5,76	New Zealand	Haast	1 020	7 690	1979
33	5,74	Pakistan	Jhelum	29 000	31 100	1929
34	5,70	Madagascar	Mangoky	50 000	38 000	1933
35	5,65	Tahiti	Papenoo	78	2 200	1983
36	5,62	Cuba	Buey	73	2 060	1963
37	5,52	USSR	Lena	2 430 000	189 000	1967
38	5,49	Hawaii	Halawa	12	762	1965
39	5,23	California	San Gorgonio	4,5	311	1969
40	5,20	China	Chang Jiang	1 010 000	110 000	1870
41	5,20	California	San Rafael	3,2	250	1973

8.4.2 Comments by the authors

The authors of the catalogue made the following comments:

- The precision of flood measurements vary, with about half of the 41 highest floods listed having accuracies of within 15%.

- It would be incorrect to assume that the high *K*-values are the result of overestimates of the river flow. The realism of the estimates of the highest floods is confirmed by the calculated associated velocities which are in the range of 8 to 9 m/s. Accurate velocities in excess of 7 m/s and up to 8 m/s have been measured in the past. Some of the 41 values were checked against well established procedures such as precipitation records and change in reservoir storage.

- The estimated return periods associated with the flood peaks in the catalogue (not the above table) range from about 10 years to more than 2 000 years based on record lengths and isotope dating procedures.

- An upper envelope *K*-value of 6,0 encompasses all but 14 of the 1 400 world-wide maxima, with an absolute upper value of 6,3 if the two anomalous values are excluded.

- For catchment areas less than 100 km^2 the envelope *K*-value decreases.

- Despite the larger number of countries which participated in the latest survey and the normal enlargement of the networks in all countries, the additional information received over the last 20 years has not necessitated a change in the location of the enveloping lines.

8.5 REGIONAL MAXIMUM FLOOD METHOD

8.5.1 Introduction

Kovâcs (Department of Water Affairs TR 105, 1980) undertook a similar analysis based on the earlier work of Francou and Rodier for 237 South African maximum recorded flood peaks. He recommended Francou-Rodier *K*-values for South Africa as follows :

5,25 Coastal areas from Mossel Bay through to the Mozambique border, (Max 10 000 km^2)

5,0 Inland sections of coastal rivers, northern and eastern Transvaal interior, south-western Cape coastal areas. (Max 10 000 km^2).

4,6 Inland areas other than the Karoo region. (Max 40 000 km^2).

Kovâcs also tentatively postulated the following :

(i) In large catchments with flat relief and extensive flood plains the flood waterways (river channel plus submerged flood plain) have a major flood peak attenuation effect, and the precipitation intensity/flood peak relationship assumed in the deterministic methods are no longer valid. Kovács goes further and maintains that in these areas the flood peak for a given return period could even *decrease* with increase in size of catchment, particularly in the more arid regions of South Africa.

(ii) The regular increase of the observed maximum floods with catchment size when compared with the somewhat irregular relationship in calculated values (statistical as well as deterministic methods) and lack of obvious outliers in the observed maxima seem to indicate that maximum floods have a ceiling with a return period within the range 200 to 500 years.

Several conclusions were drawn from the RMF analysis.

1. The Francou - Rodier equation satisfactorily describes the upper boundary of maximum floods in South Africa. The fact that it has been found to be reliable in most other parts of the globe strengthens confidence in the method.

2. For $K = 5$ the Francou-Rodier equation reduces to :

   $$Q_{max} = 100\ A^{0,5} \tag{8.3}$$

   This confirms the earlier equations by Roberts (1963, 1965) and by Pitman and Midgley (1967). These authors found that in South Africa the flood peaks are approximately proportional to the areas of their catchments raised to the power 0,5. (See Chapter 6).

   This confirmation adds confidence to the use of the areas of catchments for extrapolation from gauged to ungauged catchments for flood peak estimation.

8.5.2 Stability of the relationship

Table 8.2 details the twenty sites in South Africa which experienced floods with the highest K-values up to 1978. Five of the six top sites are dam spillways and are therefore accurate measurements. The instantaneous peaks may have been somewhat higher due to the peak attenuation effect of the dams.

Note the narrow range of K-values within which these peaks lie despite the wide range of catchment sizes; the varied geographical locations; and the 70-year time span.

Table 8.3 details the top twenty sites up to 1985, and Table 8.4 gives details of flood peaks exceeding the twentieth ranked flood in Table 8.3 during the subsequent period ending 1988.

TABLE 8.2 : Twenty sites in South Africa experiencing flood peaks with the highest K-values up to 1978. (Data from Kovács 1980)

Rank	K-value	River	Site	Area km^2	Year	Method of measurement
1	5,10	Hluhluwe	Hluhluwe Dam	734	1963	Spillway
2	5,03	Nahoon	Nahoon Dam	473	1970	Spillway
3	5,02	Loerie	Loerie Dam	147	1977	Spillway
4	5,02	Blyde	Pearston	130	1922	Unknown
5	4,95	Buffalo	Bridle Drift Dam	1 176	1970	Spillway
6	4,94	Buffalo	Laing Dam	913	1970	Spillway
7	4,94	Mkomanzi	Mkomanzi Drift	4 375	1959	Gauging site
8	4,92	Pauls	Coutzenberg	873	1974	Gauging site
9	4,90	Baakens	Frame's Drift	67	1908	Unknown
10	4,87	Mkuze	Rietboklaagte	1 467	1963	Gauging site
11	4,86	Tugela	Bond's Drift	28 490	1925	Unknown
12	4,86	Mtamvuma	Mtamvuma	715	1959	Gauging site
13	4,84	Kougha	-	2 538	1932	Unknown
14	4,83	Pienaars	Pretoria	243	1978	Road bridge
15	4,83	Heuningklip	Campherspoort	2 957	1950	Gauging site
16	4,81	Bloemspruit	Bloemfontein	36	?	Unknown
17	4,81	Berg	Wellington	2 126	1954	Gauging site
18	4,80	Mtamvuma	Port Edward	1 557	1959	Unknown
19	4,80	Shark	Port Elizabeth	9	1968	Unknown
20	4,80	Cougha	Paul Sauer site	5 099	1932	Unknown

Comment

Three of the maxima occurred during the 1959 floods in southern Natal (nrs 7,12 and 18), while another three were experienced at dams in the East London vicinity in 1978 (nrs 2,5 and 6).

TABLE 8.3 : Twenty sites in South Africa experiencing flood peaks with the highest K-values up to 1984. (Data from Kovács 1985)						
Rank	K-value	River	Site	Area km²	Year	Method of measurement
1	5,56	Black Mfolozi	Res 12 Zululand	1 635	1984	Slope-area
2	5,52	Mfolozi	Mtubatuba	9 218	1984	Slope-area
3	5,41	Pongolo	Pongolapoort Dam	7 831	1984	Slope-area
4	5,31	Mozane	Tobolsk	426	1984	Spillway
5	5,27	Loerie	Loerie Dam	147	1981	Slope-area
6	5,18	Van Stadens	Van Stadens Dam	74	1981	Slope-area
7	5,10	Hluhluwe	Hluhluwe Dam	734	1963	Slope-area
8	5,06	Mkuze	Morgenstond	2 647	1984	Slope-area
9	5,04	Mgwavuma	Mbuzini	1 660	1984	Spillway
10	5,03	Nahoon	Nahoon Dam	473	1970	Slope-area
11	5,03	Vink	Noree	194	1981	Slope-area
12	5,03	Willem Nels	Robertson	32	1981	Unknown
13	5,02	Blyde	Pearston	130	1922	Bridge
14	5,02	Buffels	Laingsburg	3 070	1981	Slope-area
15	4,98	Groot	Conradie	12 466	1981	Spillway
16	4,96	Bulk	Port Elizabeth	34	1981	Spillway
17	4,95	Buffels	Bridle Drift Dam	1 176	1970	Slope-area
18	4,94	Mkomanzi	Mkomanzi Drift	4 373	1959	Slope-area
19	4,92	Pauls	Coertzenburg	873	1974	Slope-area
20	4,92	Papenkuils	Port Elizabeth	39	1981	Slope-area
21	4,92	Elands	Longhill	400	1981	

Comment

The 1984 tropical cyclone Domoina was the cause of the extreme floods experienced at five of the sites (nrs 1,2,3,8 and 9), while nine of the floods were caused by the floods in the southern and south-western Cape in 1981.

TABLE 8.4 : Sites experiencing peaks with K-values higher than 4,92 during the period 1985 to 1988 from Kovács,1988a						
Rank	K-value	River	Site	Area km²	Year	Method of measurement
1	5,21	Matigulu	Dunn's Res.	583	1987	
2	5,13	Mkomanzi	Delos Estate	4 349	1987	
3	5,08	Mhlatuze	Goedertrouw Dam	1 270	1987	spillway
4	5,00	Mvoti	Well Vale	2 473	1987	
5	4,95	Mdloti	Hazelmere Dam	380	1987	spillway

Comment

With the single exception of the Mkomanzi River all of the flood peaks were less than previously recommended *K*-values for the regions in which they are located.

8.5.3 Comment from Kovács (1988b)

The 1988 study by Kovács (1988b) was based on 519 peak discharges recorded in southern Africa since 1856. This included information from Botswana, Lesotho, Swaziland, Namibia, Zimbabwe and Mozambique. Of the 374 peaks selected for analysis the approximate mean representative period was 50 years and the probable error in peak discharge in 53 cases was less than 10%, in another 173 cases it was less than 30%, and it was unknown in the remaining 148 cases.

Regional boundaries of K-values were drawn for southern Africa and are reproduced in Fig 8.1. In defining the boundaries consideration was given to the individual K-values, the number and accuracy of data in a particular area, the maximum recorded 3-day rainfall depths, topography, catchment orientation with respect to the dominant storm producing weather systems and general soil permeability.

The envelope values were established by plotting the peaks against the effective catchment area in each region and tracing the K = constant straight line close to the highest observed flood but not exceeding the maximum observed K-value by more than K = 0,1 to 0,3.

The RMF is expected to be most reliable in medium sized catchments having areas in the range of about 300 km² to 10 000 km². There are inherent shortcomings in all regionally based methods when applied to small and large catchments. The smaller the catchment the greater the possibility that its characteristics will differ from the typical regional value, while large catchments may lie within more than one homogeneous region.

The recommended equations for estimating the RMF values in the eight defined regions were provided and are reproduced in Table 8.5.

TABLE 8.5 RMF equations for the eight maximum flood peak regions in southern Africa from Kovács (1988b)

Region	Number of floods	Transition zone		Flood zone	
		Area range km²	RMF m³/s	Area range km²	RMF m³/s
5,6	25	1-100	$100\ A^{0,68}$	100-10 000	$302\ A^{0,44}$
5,4	34	1-100	$100\ A^{0,62}$	100-20 000	$209\ A^{0,46}$
5,2	61	1-100	$100\ A^{0,56}$	100-30 000	$145\ A^{0,48}$
5,0	155	1-100	$100\ A^{0,50}$	100-100 000	$100\ A^{0,50}$
4,6	55	1-100	$100\ A^{0,38}$	100-100 000	$47,9\ A^{0,54}$
4,0	26	1-300	$70\ A^{0,34}$	300-300 000	$15,9\ A^{0,60}$
3,4	12	1-300	$50\ A^{0,265}$	300-500 000	$5,25\ A^{0,66}$
2,8	6	1-500	$30\ A^{0,262}$	500-100 000	$1,74\ A^{0,72}$

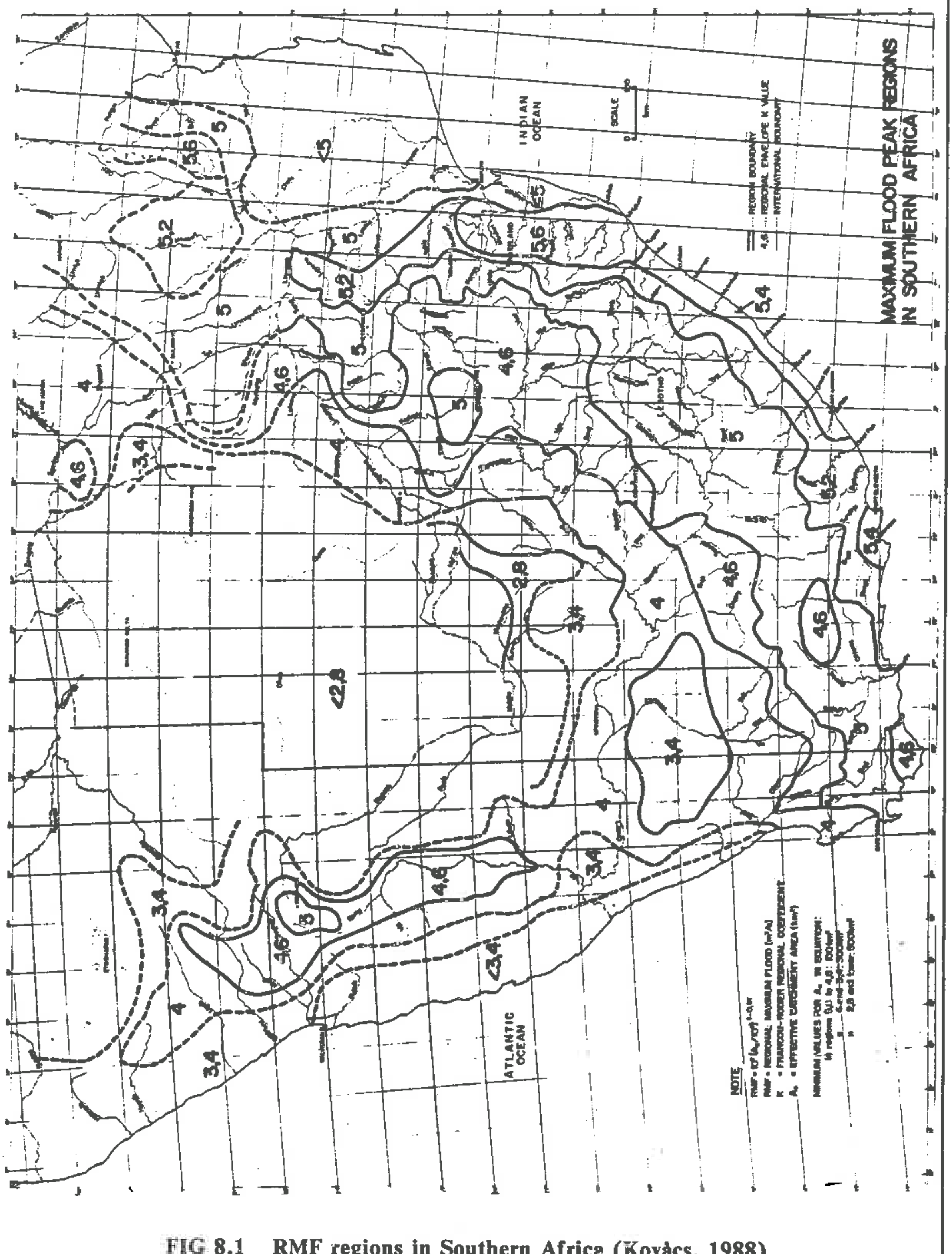

FIG 8.1 RMF regions in Southern Africa (Kovàcs, 1988)

Kovács made the following points:

- He confirmed the observation by Rodier and Roche (1984) that the RMF *K*-values are only valid for catchment areas greater than 100 km² and have to be changed for smaller catchments.
- The envelope of southern African floods has moved up from 5,2 to 5,6 during the past 28 years (1960-1988).
- During the eight years that have elapsed since the publication of TR 105 no less than five extraordinary, large area storms have been experienced over southern Africa which have resulted in the highest floods in memory in many places. The regional *K*-values given in TR 105 were exceeded at 18 sites. The excess was less than 0,2 in most of these cases with a maximum of 0,31 which was due to one of the floods caused by the tropical cyclone Domoina.

8.6 GENERAL COMMENTS

Several valuable lessons can be learnt from the use of the RMF *K*-values to rank floods:

1 Most of the extreme floods recorded in southern Africa were the result of storms which covered a wide area and caused severe floods of approximately the same severity (*K*-value) at a number of different sites. This raises serious doubts relating to the validity of the basic assumption used in the deterministic methods that the properties of the design storm are a function of the size of the catchment and the catchment response time. It would seem that the critical storm rainfall for a required annual exceedance probability (AEP) is unrelated to catchment characteristics, and is a function of the statistical properties of storm rainfall alone. It is therefore necessary to examine the joint probabilities of the depth-area-duration characteristics of storm rainfall in order to determine which combination produces the maximum flood hydrograph for the required AEP.

2 The regional maximum flood must not be confused with the probable maximum flood. The RMF is an upper envelope value of flood peaks that have been observed in a region, whereas the PMF is an estimate of the maximum floods that could be experienced in the region.

3 The validity of Creager's comment quoted earlier that ".... *it must be remembered that what has occurred in the past must surely be exceeded in the future.*" has been demonstrated. The question is whether or not the upper limit is being approached.

8.7 PALEOFLOODS

8.7.1 Introduction

A new development in flood hydrology is research on the occurrence of past or ancient floods (eg Baker,1987). As described in Chapter 11, flood plains typically slope away from the river banks towards the flanks of the flood plain. The reason for this is that the vegetation along the river banks reduces the velocity of sediment laden water as it leaves the channel during floods. As the sediment carrying capacity of flowing water decreases rapidly with decrease in velocity, the coarser sediment is deposited on the river banks, while only the finer material is deposited in the low lying slackwater areas away from the river channel. This sediment contains organic matter which can be dated using radiocarbon dating techniques. During exceptionally large floods the deposition will occur on the flanks of the valley and may remain undisturbed by subsequent lesser floods. If the river channel and flood plain geometry has not changed significantly since the extreme flood, and the elevation of the deposit can be traced along the valley sides for a long enough distance, it will be possible to estimate the magnitude of the flood and the date of occurrence.

If gauged data is available at or near the site, the paleodata can be included in the data set in the same way as historical data. This is described in Chapter 5. Where a succession of extreme floods can be identified from paleodeposits and their magnitudes and dates of occurrence determined, the series can be submitted to a statistical analysis.

8.7.2 Alternative application in southern Africa

An alternative method is the determination of the magnitude of the single highest paleofloods at a number of sites within a region, and then to use the RMF method to determine the K-values associated with these floods. This would provide a valuable indication of the asymptotic value of the RMF K-value for the region. Dating the deposit is not necessary for the application of the method.

Southwood, S.R. (personal communication) noted that the sedimentary sequence evident in many of the rivers of southern Africa, but especially in Natal, Swaziland and the eastern Transvaal lowveld follows an altitudinally distinct pattern, indicated by four distinct levels at increasing elevations.

1. *River bed* being either the parent rock or coarse alluvium, except near the coast where river sand may be encountered to depths of 20 metres or more.

2. *Stratified alluvium* (Dundee soil form terrace) of recent origin, comprising sands and silts with very little evidence of mature soil formation.

3. *Consolidated alluvium* (Oakleaf soil form terrace) which, although of sedimentary nature, comprises developed soils indicative of medium age.

4 *Weathered alluvium* (Hutton soil form terrace) of considerable age and soil development, but originally of alluvial origin.

The structure of the higher horizons depends on climate, parent rock, aspect, slope and other factors but is generally independent of recent alluvial activity, although ancient alluvial terraces associated with higher valley floor and river channel elevations may be evident.

Southwood noted that during the floods caused by the tropical cyclone Domoina in 1984 described in Chapter 3, the Dundee form terraces were either submerged or else radically altered, while flood levels were still below the Oakleaf form terrace level.

Further research on this potentially valuable method for determining the upper limit floods experienced within a region is urgently necessary.

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Chapter 9

THE DESIGN FLOOD

Contents

9.1 INTRODUCTION

> *Judgment is important not only as regards the physical phenomena involved and the interpretations of the results but also, and perhaps more importantly, the applicability of the methods envisaged to the problems addressed.*
> (Bouvard, 1988)

The quotation from Bouvard is the essence of flood risk analysis. The physical phenomena and the available methods have been dealt with in previous chapters. In this chapter specific procedures and methods will be recommended. The applicability of these methods to the problem at hand and the interpretation of the results require the application of the user's experience and judgment. Comment on these aspects is also included in this chapter.

9.1.1 Policy decisions

There are a number of situations where the client for whom the flood risk analyses are undertaken has to achieve a balance between accuracy, consistency, design costs, project capital costs, project operation and maintenance costs where the latter include damage risks to the structure, and third party inconvenience, financial loss and possibly loss of life.

Consistency
A good example is the determination of flood lines where these are used to control development on flood plains (to be discussed in more detail in Chapter 11). Different analytical methods will produce different flood line elevations which would obviously be undesirable within urban areas, or where insurance premiums are based on flood risks. In this situation consistency is of greater importance than the incremental accuracy, and simple, robust methods such as the log normal distribution and rational method using a specific implementation may be the preferred methods.

Minimum design costs
A roads authority may specify that all culverts up to a certain size be designed to pass (say) the 20-year flood peak. As there will be little difference in the results produced by the different methods, and the consequences of under design are likely to be minimal (otherwise a smaller AEP would have been required), the method which requires minimum design costs could be specified.

Minimum project costs
A number of structures will fall into this category. Provided the failure of the structure is unlikely to result in loss of life the client may specify an empirical method which experience has shown to produce satisfactory results. For example a roads authority may specify that all bridges across rivers along a particular route be designed so that the

minimum clearance below the bridge decks is one metre (to allow for wave action and floating debris) during the passage of a flood peak equal to 50% of the RMF; that the approach embankments be designed to withstand 75% of the RMF; and that the structure itself must be able to withstand the full RMF.

Full hydro-economic design
Continuing with the last example, in the case of a national highway of strategic importance the client may prefer to instruct the designer to derive the best practically possible estimates of the flood peak-frequency relationship, and carry out a full hydro-economic optimisation study including the risks of structural damage to bridge elements and costs to road users during the repair period as well as replacement costs of the bridge elements. This investigation which will have to include Monte Carlo simulation analyses may well take several hundreds of man-hours to complete.

Loss of life
This aspect is discussed in more detail in Chapter 12 on dam safety.

9.1.2 Sound and desirable practice

The purpose of this chapter is to present methods which represent sound and desirable engineering practice in South Africa subject to the proviso that the results are interpreted with care. The eight recommended methods cover the full range of flood risk problems likely to be encountered in South Africa, other than specialized problems such as the design of urban drainage systems, although much of the material in the handbook is relevant to these problems as well.

The use of methods other than those described in this chapter places an additional onus on the user to satisfy himself, and possibly others, of the validity of the alternative method adopted when applied to the problem at hand.

It is anticipated that the users of these methods will range from farmers with no hydrological knowledge through to experienced staff in large organizations. The problems will vary from a small farm structure whose failure will result in nothing more than inconvenience and small repair costs, through to the design of large storage dams where under-design may cause loss of life, and over-design may result in the project not being economically viable.

A concern often expressed by experienced analysts in South Africa as well as overseas is that if "cookbook" methods are provided where the user simply follows instructions, these may be counterproductive in that users are not forced to understand the problem (Pilgrim 1986, 1987).

The other alternative must be considered. Many users of this handbook will not be as proficient in hydrological analyses as one would like. Indeed, the main purpose of the handbook is to increase the proficiency of the readers. This is better achieved, and at less risk of the users making calculation errors and misinterpreting the results, if standard procedures and computer programs are provided than if they are not.

It is the responsibility of the users of the methods detailed in this chapter to assess whether or not they are competent to carry out the analyses. Neither this handbook nor the computer programs have been structured so as to force the user to apply his mind to the problem. That is the user's responsibility. Our objective is to provide the user with information and methods that will allow him to carry out the analyses with consistent accuracy and efficiency.

The original publications describing the methods referred to in this handbook should be studied, particularly in the case of important or difficult problems, or when recalibration studies have to be undertaken.

9.1.3 Computer programs

The computer programs were developed over a period of some five years, and more than 200 program disks are in circulation at the time of writing. During this period the programs have been improved and known program errors corrected. The purpose of the programs is to reduce the calculation burden and in so doing provide the user with greater insight into hydrological analyses by allowing him to explore alternative solutions and build up a library of solved problems using consistent calculation methods. In time the user will gain experience in the relative applicability of the methods in solving a range of problems.

It is impossible to test the programs exhaustively under all possible combinations of circumstances, and no warranty is expressed or implied as to the performance of the programs. There may well be as yet undiscovered errors in the program algorithms.

As a first step users should apply the programs to previous problems and compare the results. The answers will not be exactly the same as curve fitting procedures were used to determine the mathematical formulations of the graphical relationships in some of the original methods. This applies particularly to the HRU 1/72 methods. Reasons for major discrepancies should be sought.

9.1.4 Departure from the recommended methods

If methods other than those described in this chapter are used, it nevertheless remains sound practice to apply those of the recommended methods that are appropriate to the problem, compare the results, and motivate reasons for the departure from the recommended methods.

The recommended methods have all been developed and tested against large South African data sets.

The following criteria on their own, are not sufficient reasons for assuming that another method is superior to a recommended method.

(a) That it has a sounder theoretical basis. This is a fallacious argument and arises from a lack of understanding of the problems of small samples, scale and dimensionality in hydrological modelling.

(b) That it can be shown to perform better when tested against actual storm rainfall and the resulting flood peaks. In the case of deterministic models where the flood magnitude is derived from storm rainfall, the relative merits of alternative models can *not* be evaluated by comparing their ability to replicate individual floods from actual storms. This assumption arises from a misunderstanding of the basic probabilistic nature of flood frequency estimation models, their applications, and parameter estimation procedures.

(c) That it can be shown to perform better when tested against large synthetic data sets. Mixed populations of storm rainfall generating mechanisms are known to be present over most of southern Africa. Antecedent catchment moisture status is also a significant variable. Our present knowledge is such that it is not possible to generate realistic synthetic flood peak data sets that can replicate these factors at a level of accuracy that would allow meaningful comparative performance testing of alternative combinations of probability distributions and moment estimators. The results may be illuminating but cannot be conclusive, particularly in southern African situations.

9.1.5 Consistency

Another requirement for the application of the recommended methods is that the methods used for deriving the input data should be consistent and if possible, the same as those used in developing the methods. It is incorrect to assume that a more accurate measurement will provide a more accurate result. An example is the determination of the length of the main stream which is required for the determination of the main channel slope. The length is clearly a function of the scale of the map and the method of measurement. The use of a larger scale map and more refined method of measurement will result in a longer stream length, flatter slope and reduced estimate of the flood peak. However, the assumption that this would result in a greater accuracy in the answer would be incorrect as a change in method would require a recalibration of the model. This includes experience-based models.

The same applies to other components of the models, including the methods for estimating the rainfall data used as input to the models.

9.2 RECOMMENDED METHODS

The following methods are recommended for flood risk analysis in southern Africa. The application of the methods is described in Chapter 13 and they are included in the computer programs provided with this handbook. No significance should be attached to the order in which the methods are listed.

1. **Unit hydrograph : original algorithms**
   Source : University of the Witwatersrand Hydrological Research Unit Report No 1/72. This report is referred to as HRU 1/72 below.

   A revised depth-duration-frequency diagram for point rainfall was published in HRU 2/78. The two associated publications for Namibia are HRU 2/80 on large area storms and HRU 14/81 on design flood determination.

   This method can be applied manually using the graphs reproduced in the appendix to Chapter 13 of this handbook, or by using the computer programs. The results will not be identical as curve fitting procedures had to be used to convert some of the graphs into computer algorithms. These differences are not meaningful.

2. **Unit hydrograph : alternative algorithms**
   Source : This handbook.

   The basic methodology is the same as that in HRU 1/72, except that alternative algorithms are used for the rainfall depth - area - duration - frequency (DADF) relationship, and effective rainfall.

   This method can be applied manually but the computer method is preferred due to the interpolation required for determining short duration point rainfall.

3. **Probable maximum flood : original algorithms**
   Source : HRU 1/72

   This method is substantially the same as method 1 except for changes in some of the relationships. Both the manual and the computer versions follow the HRU 1/72 methodology except that the computer program does not make provision for subdivision of large catchments, and uses the single probable maximum precipitation (PMP) relationship for the whole of southern Africa, and not the regional figures.

4. **Rational method : original algorithms**
   Source : Department of Water Affairs (DWA) calculation sheet which is reproduced in Chapter 13.

This method is flexible in that the user can use his own DADF relationships, but the method for determining the *C*-coefficient is fixed.

Only the manual method for the full DWA implementation is provided in this handbook.

5. Rational method : alternative algorithms
Source : This handbook.

This is a simplified version of the DWA method. Fewer criteria are used for determining the *C*-coefficient, but this is adjusted to accommodate the effect of antecedent conditions which in turn are assumed to be a function of the return period.

Two computer versions are provided :-

(a) DADF algorithms from the original HRU 1/72 method (method 1 above)

(b) DADF algorithms compatible with the alternative implementation of the HRU 1/72 method (method 2 above).

6. Regional maximum flood
Source : Kovács (1989)

This is a revision of Kovács (1980). Manual analysis is very simple, but it has also been included in the suite of computer programs.

7. Direct statistical analysis : single station
Source : This handbook.

Manual and computer methods are provided for the following distributions :-

Distribution	Moment estimators
Normal	Conventional moments
Log normal	Conventional moments
Log Pearson 3	Conventional moments
Log general extreme value	Conventional moments

8. Direct statistical analysis : regional
Source : This handbook

Only computer implementations are practicable for regional direct statistical analyses.

The following distributions are used :

Distribution	Moment estimators
Untransformed data :	
Extreme value Type 1	conventional
General extreme value	conventional
General extreme value	probability weighted
Wakeby	probability weighted
Log(10) transformed data :	
Log normal	conventional
Log Pearson Type 3	conventional
Log EV1	conventional

Obvious omissions, but not necessarily shortcomings, are the three parameter log normal distribution, maximum likelihood estimators, and confidence bands. Reasons for the omissions are given elsewhere in the handbook.

9.3 ALTERNATIVE METHODS THAT MAY BE CONSIDERED

9.3.1 Unit hydrograph method for large catchments

The application of the unit hydrograph method to catchments larger than 5 000 km^2 is detailed in HRU 1/72. The method involves subdividing the catchment into subcatchments, synthesizing a design storm and distributing the resulting rainfall in time and space over the subcatchments, and then routing the resulting hydrographs to the point of interest.

This method has been widely used in practice.

A fundamental objection to the method is the high degree of subjectivity involved in the transposition of the design storm to the catchment, and to a lesser extent the estimate of the DADF relationships of the design storm. This almost ensures that no two analysts will get the same answer to the same problem, and raises serious doubts when the assumption is made that the annual exceedance probability of the flood is that of the design storm.

Analysts applying the unit hydrograph method to large catchments should take additional care when interpreting the results of the calculations.

9.3.2 Other candidate alternative methods

The United States Department of Agriculture's Soil Conservation Service (SCS) model is well known, and was further developed for South African conditions by Schmidt and Schulze (1987). The model was developed for use on small catchments up to 8 km^2. The authors recommend that this should be regarded as the upper range in catchment area for which the model can be used with confidence. The rational method described in this handbook is an alternative method for agricultural catchments of all sizes.

There are a number of models developed specifically for urban drainage which are not listed in this handbook. Several of them are based on the rational method but with more sophisticated hydraulic routing algorithms. These specialized applications are beyond the scope of this handbook although the basic principles in this handbook are applicable. The rational method can be applied directly to many urban drainage problems.

A criticism of the deterministic methods is the assumption that the design flood for a given return period results from a storm having the same return period, falling on a catchment having an undefined average moisture status. In the HRU 1/72 procedures empirical curves are provided to guide the user in selecting appropriate losses to account for average catchment moisture status in the various regions of the country. This procedure is explained by D.C. Midgley in a personal communication as follows:

> "Classical unitgraph procedures for design flood determination require as primary input a design storm having the design return period and an appropriate means of accounting for "storm loss" such that the gross storm input can be converted to excess rain or direct runoff. The resulting excess rain is normally broken down into uniform "blocks" of duration equal roughly to one-quarter to one-third of the response time of the catchment, and arranged in significant time sequence, eg as given by the time distribution curves in HRU 1/72 Fig C8. A unitgraph of appropriate time unit is then applied to each block and the results, suitably lagged, are summed to yield the direct runoff hydrograph to which may be added the estimated base flow to yield a design hydrograph. For spillway design, a series of hydrographs derived from increasingly prolonged excess rain are routed through storage to establish the hydrograph which demands the maximum spillway capacity.
>
> Modern tendency is to work with long-duration storms in which are embedded, for the given return period, maximum intensities of all shorter duration events.
>
> In DWA TR102 Adamson has listed maximum one-day to seven-day point precipitations for return periods of 2 to 200 years, as well as maximum recorded falls at each of 2 400 stations throughout the country. These values must be adjusted from point rainfall to catchment-averaged rainfall with the aid of an appropriate ARF diagram.

One can then readily set up the mass curve of a catchment-averaged design storm of any desired return period and of duration from one to seven days. One can check the resulting storm against the corresponding catchment-averaged storm derived from the DADF diagrams in HRU 1/72 Appendix D.

If the PMP storm is to be synthesized, it is important to check Adamson's maximum recorded 24-hour values at points within about 100 km of the problem area against those for the region given in Fig C4 of HRU 1/72, and also against the updated values recorded during Domoina (1984) and the Natal, OFS and Northern Cape floods of 1987 and 1988. It might be wise to check also by Hershfield procedures.

The next step is to disaggregate the 24-hour duration to shorter duration values with the aid of the envelope curve of HRU 1/72 or with values in the following table taken from DWA TR102.

Table 9.1 Ratio D-hour : 24-hour storm depth at the same risk level

Duration (h)	Summer Rainfall	Winter Rainfall
0,1	0,17	0,14
0,25	0,32	0,23
0,50	0,46	0,32
1	0,60	0,41
2	0,72	0,53
3	0,78	0,60
4	0,82	0,67
5	0,84	0,71
6	0,87	0,75
8	0,90	0,81
10	0,92	0,85
12	0,94	0,89
18	0,98	0,96
14	1,00	1,00

Next comes the allowance for "losses" or basin recharge.

The loss rate can be taken from "Mean Storm Losses" Fig G2 and for PMF studies from Fig G1 of HRU 1/72. Several workers have pointed to the need to introduce another curve between curves A and B for steep terrain. The storm loss can be deducted incrementally from the cumulative gross rainfall input to yield the mass curve of excess rain which can then be differentiated to reflect "blocks" of appropriate duration consistent with the probable response time of the problem catchment. Excess rain is then converted to runoff by means of unitgraph or routing procedure.

Trial storm durations do not have to be tested to establish the critical duration (ie that which will generate the maximum peak flood for the given return period) as the critical duration will reveal itself in the process. On the other hand, it would be wise to test a variety of time distributions of the long duration storm to identify the maximum for the given rainfall input. For very large catchments, storm transposition techniques can be applied following the guidelines in HRU 1/72.

By adopting the foregoing procedure and routing the resulting hydrograph through surcharge storage on the full reservoir, with different outlet and spillway configurations, one can establish the required spillway capacity and the necessary freeboard."

In the recommended method two in paragraph 9.2 above (HRU 1/72 unit hydrograph method using alternative algorithms) the storm loss algorithm addresses the problem more directly by reducing storm losses with increase in return period.

9.3.3 Runhydrograph method

The runhydrograph method for determining the flood peak-volume relationship requires further development and should not be used without first consulting the authors, (see para 2.6.3 in Chapter 2).

9.3.4 Alternative methods that are not recommended

The only recommended method for the estimation of the probable maximum flood is that given in HRU 1/72. More sophisticated methods have since been developed elsewhere in the world, but they require meteorological information that is not readily available in southern Africa and also require that the precipitation analyses will be undertaken by meteorologists experienced the relevant techniques. South African meteorologists have not entered this field.

Some fundamental objections to the probable maximum flood concept are discussed later in this chapter.

Consequently, alternative probable maximum flood estimation procedures should *not* be applied in South Africa until such time as they have been developed and evaluated.

9.4 CONCEPT OF RISK

The designer has to select his design flood from a whole series of values, each associated with a probable risk of being exceeded during the effective life of the structure.

The risk r that an event having a return period T years will be exceeded at least once during the design life L is given by:

$$r = 1 - (1 - 1/T)^L \tag{9.1}$$

The interrelationship of risk, design life and return period are illustrated in Table 9.2.

TABLE 9.2 Percentage risk r that an event of given return period T will be exceeded at least once during the design life L years.

Return period T in years	L = 1	2	5	10	15	20	25	50	100
10	10	19	41	65	79	88	93	99,5	99,9
20	5	10	23	40	54	64	72	92	99,4
50	2	4	10	18	26	33	40	64	87
100	1	2	5	10	14	18	22	40	63
200	0,5	1	2	5	7	10	12	22	39
500	0,2	0,4	1,0	2	3	4	5	10	18
1000	0,1	0,2	0,5	1,2	1,5	2	2	5	10

Note that the probability of the 200-year event being equalled or exceeded in any one year is only 0,5% (by definition), but there is a 22% probability that it will be equalled or exceeded at least once in the next 50 years, which is by no means insignificant.

Note also the use of the terminology "*equalled or exceeded*". The probability that a future event will equal any specified value is extremely small. Of more specific interest is "*the probability that a specified flood may be equalled or exceeded at least once during the design life of the project*".

Table 9.2 can be restructured to show the required return period for a specified design life and percentage risk of being equalled or exceeded at least once during the design life of the structure. This relationship is shown in Table 9.3.

TABLE 9.3 Return period *T* in years for given design life *L* and permissible risk *r*.

Permissible risk of failure *r*	*L* = 1	2	5	10	20	25	50	100
99 %	1,0	1,1	1,7	2,7	4,9	6,0	11,4	22,2
95 %	1,1	1,3	2,2	3,9	7,2	8,9	17,2	33,9
90 %	1,1	1,5	2,7	4,9	9,2	11,4	22,2	43,9
75 %	1,3	2,0	4,1	7,7	14,9	18,6	36,6	72,6
50 %	2,0	3,4	7,7	14,9	29,4	36,6	72,6	145
33 %	3,0	5,5	12,9	25,2	49,9	62,1	124	247
25 %	4,0	7,5	17,9	35,3	70,0	87,3	174	348
20 %	5,0	9,5	22,9	45,3	90,1	113	225	449
10 %	10,0	19,5	48,0	95,4	190	238	475	950
5 %	20,0	39,5	98,0	195	390	488	975	1950
2 %	50,0	99,0	248,0	495	990	1238	2476	4951
1 %	100,0	199,5	498,0	995	1990	2488	4977	9953

Note that if the design life of the structure is 50 years and the maximum permissible risk of failure during this period is 1%, then the design return period will have to be 5 000 years !

The introduction to HRU 1/72 has a very good explanation of the philosophy of design.

9.5 ESTIMATION OF CATCHMENT PRECIPITATION

9.5.1 Depth-duration relationship

All of the recommended deterministic methods assume that there is a continuous inverse relationship between storm duration and rainfall intensity. However, this is only valid for each different type of storm rainfall. For example there are abrupt discontinuities in the relationship below durations of about 15 to 30 minutes where convective cloud mergers are the major threat, and another discontinuity in the range of six to 24 hours. Six hours is approximately the longest duration of intense rainfall caused by convergence storm rainfall including cut-off low pressure systems and tropical cyclones *at a fixed point on the ground*. Multiple, consecutive storms are the most likely cause of severe floods in catchments with response times exceeding 24 hours, and prolonged seasonal rainfall for still larger catchments.

9.5.2 Area reduction factor

Similarly, the deterministic methods assume that there is a continuous inverse relationship between the area reduction factor (ARF) and catchment size, whereas this relationship is also a function of the type of storm. For example the area of rainfall produced by a cloud merger has a maximum diameter measured in tens of kilometres, tropical cyclones produce a swathe of rainfall with a maximum width of a hundred or so kilometres, whereas heavy seasonal rainfall can cover half of the southern African sub-continent. These discontinuities are located at the same time spans as those for the depth-duration relationship.

9.5.3 Areal and time distribution of storm rainfall

With the possible exception of orographic rainfall, storm rainfall is not evenly distributed in either time or space over a catchment. Until the advent of weather radar observations it was not possible to measure the areal distribution of precipitation over a catchment directly, nor how this varied with time. The analyses of weather radar data in future will provide better approximations of the distribution of flood-producing precipitation over large catchments, but it may well be a decade or more before this information is available. In the meantime the effects of the assumptions inherent in the present methods should not be overlooked.

The assumption of uniform distribution of precipitation in time and space for catchment response times of up to about six hours is acceptable for those methods which have been calibrated using this assumption. The practice of subdividing larger catchments into smaller subcatchments, storm positioning and movement, and routing the resultant hydrographs to the site should not be attempted unless the analyst is adequately experienced in this procedure.

9.6 ESTIMATION OF ANTECEDENT CATCHMENT MOISTURE STATUS

Maximum runoff from the surface of a catchment for a constant precipitation rate can take place only after the following conditions have been met:-

(i) Surface ponds of all sizes due to surface irregularities have been filled with water.

(ii) Where the surface is pervious, the soil has approached saturation and infiltration losses are minimal.

(iii) All watercourses have been filled with water up to the level required to sustain the peak flow.

The volume of water required to fulfil these requirements is large in the case of initially dry catchments, and this is reflected, for example, in the value of the runoff coefficient *C* used in the rational method for rural catchments.

The assumption of an initially dry catchment may be valid for return periods of less than 20 years, or for high intensity, very short duration (less than 30 minutes) precipitation over small catchments. It is not valid for situations where the duration of the precipitation exceeds the catchment response time which is often the case for small catchments. (See the examples in the annexures to Chapter 3).

While the probability of a cloud merger occurring directly over a small catchment is small due to the size of storm system and its ephemeral nature, larger scale systems such as those recorded at Pretoria, Port Elizabeth and East London, have a much greater probability of occurrence.

It is therefore not at all unlikely that the critical storm for small catchments with catchment response times in the range of between 30 minutes and about three hours will be immediately preceded by rainfall of the same magnitude and duration, and for catchments with response times from three to six hours, the design storm may be preceded by rainfall which will satisfy a large proportion of the pondage, soil moisture deficit and channel storage demands. Under these circumstances a runoff coefficient equal to or approaching 1,0 (rational method) or zero storm losses (unit hydrograph method) would not be unreasonable.

As was shown in Chapter 4, antecedent catchment moisture status which includes antecedent groundwater and surface water flow plays an increasingly important role in the rainfall-runoff process with increase in aridity, but this is not adequately accommodated in the original algorithms of the deterministic methods in the computer programs. An attempt was made in the alternative algorithms to account for this by making the runoff and effective rainfall coefficients a function of the exceedance probability on the basis that the low probability floods are the result of the joint occurrence of high intensity rainfall on an abnormally wet catchment.

This is a weakness in the deterministic methods that needs further research.

9.7 DIRECT STATISTICAL ANALYSIS

9.7.1 Causes of anomalous results

One of the difficulties in carrying out direct statistical analyses is the effect of upstream utilization on the observed annual maximum flood peaks. In years of low rainfall, upstream storage dams, often including a multiplicity of farm dams, and possibility major storage dams, will have an appreciable effect on the recorded flood peak at a downstream gauging station. This effect will decrease in years of high rainfall, and will be minimal in the case of extreme floods.

The resulting low outliers have a significant effect on the skewness of a data set. Although mathematical methods for identifying outliers are available, hydrological data sets are far too short for the adoption of such procedures with confidence. Regional analyses can be used to improve the estimates but rest on the assumption that the stations lie within a hydrologically homogeneous region.

9.7.2 Graphical presentations

Because the results of direct statistical analyses of reliable records are the only means of calibrating other flood frequency estimation models, it is essential that every effort be made to obtain the most reliable results before calibration studies are undertaken. Examination of the graphical presentations produced by the computer analyses is the most efficient means of identifying anomalies in the data sets.

The coefficient of variation C_V is a measure of the variability of the data about the mean value. This is the slope of a line drawn through the plotted values.

The skewness coefficient g is a measure of the skewness of the distribution of the data. This is the curvature of the plotted values on the graph. Zero skewness of the logarithms (the plotted points lie on a straight line on the log normal graph) indicates that a log normal distribution would be appropriate. The log normal fit is shown on all of the graphs presented later in this study and is a useful reference line when interpreting the results.

Anomalies in the plotted data could result from one or more of the following:-

- Station calibration errors. These show up as abrupt changes in the position of the plotted data.
- Upstream dams and other forms of catchment utilization will have the largest effect at the lower end of the plot where they will be negatively curved and will introduce anomalous negative curvature at the top end of the fitted curves.
- Differences in flood producing catchment characteristics due to urbanisation and catchment degradation may also affect the results, although the effect will be difficult to detect.
- Differences in the type, frequency and magnitude of severe storms (mixed storm populations) are readily detectable as high outliers.
- Chance differences in the frequency of severe storms may result in the absence of expected high values at the top end of the plot.

9.7.3 Composite flood frequency curves

It is sound practice to plot the results of all methods on a single log normal probability graph. A subjective best fit curve could be drawn through to the PMF if estimates of long return period maxima are required. This is discussed further in Chapter 12 on dam safety.

9.8 CATEGORIZATION OF DESIGN FLOOD PROBLEMS

9.8.1 Categories of problems

Design flood estimation problems can be categorized on the basis of catchment response time (time of concentration using the rational method) and design risk (return period) as shown in Table 9.4. The catchment sizes are related to catchment response time categories which in turn are related to the dominant types of flood producing storm rainfall mechanisms. The relationship between catchment size and response time varies over a wide range. It is the response time of the catchment that is important for categorization.

Table 9.4 Categories of design flood problems

Design return period T (years)	Catchment response time (hours) *Approximate catchment size (km^2)*		
	< 6 *0 - 300*	6 - 24 *300 - 3 000*	24 + *3 000 +*
2 - 20	A	D	G
20 - 200	B	E	H
200 - PMF	C	F	I

Calculation methods can also be broadly classified into the following categories.

- Uncalibrated deterministic methods
- Calibrated deterministic methods
- Single station direct statistical analysis methods
- Regional direct statistical analysis methods

9.8.2 Hydrograph shape

The shape of the design hydrograph is of importance to the designer of a dam in that for known characteristics of the reservoir basin and spillway dimensions it determines the maximum water level that will be reached by the design flood. For a given return period two extremes can be postulated. The minimum scenario is a flood hydrograph generated by the design storm which has a duration equal to but not exceeding the calculated catchment response time. This will have the minimum volume. The worst scenario will be a constant flow equal to the calculated design peak, in which case the flood absorption capacity of the reservoir basin will be zero. It could be argued that the latter assumption violates the assumptions associated with the specified return period. However, this violation refers to the specific design hydrograph and ignores the antecedant flow conditions. The larger the catchment and the greater the magnitude of the flood, the greater the likelihood that the design flood will be immediately preceded or succeeded by floods of much the same magnitude, resulting in prolonged high inflows. It is therefore not unrealistic to assume a constant inflow equal to the design peak inflow. A reduction in this assumption should motivated on grounds other than design hydrograph shape.

The hydrographs from a specific catchment will vary widely in shape although the annual maxima for small catchments will tend to be the result of storms which cover the whole catchment and will be less variable than the individual flood hydrographs during the year. The unit hydrograph method can be used to derive an estimate of the hydrograph shape. The usefulness of the derived hydrograph will depend on the proposed application. If a preliminary estimate of the hydrograph for a small or moderately sized catchment is required then this method will provide adequate answers. The larger the catchment, the greater the probability that the hydrograph will be the result of uneven distribution of storm rainfall in time and space over the catchment and consequently the greater the variability in hydrograph shape.

Although the unit hydrograph method can be used to develop different hydrograph shapes by using non-critical storm durations and associated rainfall intensities as well as by the subdivision of large catchments into smaller units and the routing and combination of the individual hydrographs to produce a single design hydrograph, the validity of this approach on the assumed return period should be considered and the analysis should be undertaken only by hydrologists experienced in this procedure.

Operational hydrograph shape (flood routing through dams)

From an operational point of view the rate of increase in flow which is the slope of the rising limb of the hydrograph is of major importance. For a given return period the maximum rate of inflow could result from intense local storm precipitation falling directly on the reservoir surface. The designer should therefore base his operating rules on an envelope of hydrograph shapes where both the area and duration of precipitation are variables, and not only on the design hydrographs relating to the whole catchment. The most critical situation would arise when the flood peak from a major storm is about to enter the dam and another storm produces high intensity rainfall covering a limited area just upstream of the dam. This was the case in the Vaal Dam floods described in Chapter 3.

9.9 CHOICE OF METHOD

Carrying out the analyses will take appreciably less time than the collection of information, particularly if the computer programs are used. With the exception of method three (DWA implementation of the rational method), all of the first six methods are included in a single program. Similarly, all the direct statistical analysis methods are included in the regional analysis program. Running these two programs will provide a set of results for all of these methods.

At the time of writing no comprehensive study on the relative merits of the different methods has been undertaken in South Africa. In an important investigation the user may wish to carry out such a study on a limited scale within the region where the problem site is located. The case study on the uMgeni River dams in Chapter 14 will provide guidance on this approach.

In the absence of a comparative study what weight should be placed on the results of the different methods?

The alternative algorithms used in the unit hydrograph and rational methods are based on information that was not available when the methods were developed. There are no grounds as yet for determining whether the unit hydrograph method is intrinsically better than the rational method or vice versa. One advantage of the unit hydrograph method is that it also yields the design flood hydrograph.

In the direct statistical analysis methods, greater weight should be given to the log Pearson Type 3 distribution than the other distributions. This conclusion is based on the study of a number of South African data sets as well as on the USA and Australian experience as expressed in their guidelines.

For further information refer to the chapters where the methods are described in detail and the case studies in Chapter 14.

As there is no single method which can be used with complete confidence for any specific project, the designer will usually have to use several methods, reject those which give anomalous results, and then take the weighted average of the results of remaining methods. The extent of the weighting will depend largely on the designer's judgment and experience.

Some of the factors that have to be taken into account when deciding on which methods are most suitable for the situation being studied include :

- What accuracy is required?

- Is the location typical of that in the surrounding region or are there special conditions such as topography or man-made influences which make the use of regional relationships unreliable?

- What historical data are available at or near the site being studied (eg streamflow, daily precipitation, and recording rain gauge data)?

- What is the extent of anomalous catchment utilization?

For the more important structures where costs associated with spillway or drainage system capacities, or the consequences of failure justify more detailed studies, it may be worth while to sharpen up the parameter values of the deterministic methods by calibrating them against the results obtained when using direct statistical analysis methods at reliable stations in the region.

Another aspect to be borne in mind is that while precipitation alone plays a dominant role in small rural catchments and most urban catchments, other factors such as channel and overbank storage, areal and time distribution of precipitation, and antecedent conditions all of which modify the rainfall-runoff relationship become increasingly important with increase in catchment size. These factors limit the application of deterministic methods which assume that the flood AEP is that of the AEP of the rainfall, to small to medium size catchments. Deterministic methods should be used with caution where catchment areas exceed 5 000 km^2.

Where low risk estimates are critical, thought should be given to the assumptions inherent in the probability distributions used in the deterministic as well as the direct statistical methods.

Alternative methods for some of the more common applications are given below. Detailed calculation procedures are given in Chapter 13 and in the case studies in Chapter 14.

9.10 SMALL CATCHMENTS (Categories A, B and C in Table 9.4)

9.10.1 Recommended methods

As there are unlikely to be gauging stations at or near the site deterministic methods will have to be used. The rational method can be used for all small catchments. The unitgraphs in the unit hydrograph method were derived from larger rural catchments and are not characteristic of small catchments, particularly urban catchments. The unit hydrograph analysis is also unstable for catchments with response times of less than about one hour.

9.10.2 Determination of catchment response time

Two types of flow are of interest - flow in a confined channel, and flow across a surface with no dominant channels. In the case of a small impervious catchment such as a paved parking area, surface flow is dominant and conventional hydraulic analyses can be used to determine the time interval between the commencement of the storm and the arrival of the flood peak. In larger urban catchments channel flow becomes increasingly significant, and where this consists predominantly of man-made drainage systems of open or closed conduits, hydraulic calculations can be used. These can become analytically complex in the case of extensive drainage networks. As the size of the catchment increases, the proportion of the total catchment response time consisting of surface flow decreases and becomes less relevant in determining the response time. Similarly, as the proportion of the catchment that is fully urbanized decreases, the assumptions used in hydraulic calculations become more questionable until the position is reached where empirical equations used for determining the catchment response time are as accurate as the more sophisticated and time consuming hydraulic calculations.

Whatever method is used the goal is the determination of the critical storm duration based on the assumption that there is a continuous, inverse relationship between storm duration and rainfall intensity over the whole range of likely catchment response times. Whether or not this relationship is relevant for small catchments is seldom questioned. (See 9.6.3 above).

9.10.3 Determination of precipitation intensity

This is the critical component in small catchment analyses. The relationship in the HRU coaxial plot (Fig 13.2 in Chapter 13) will be adequate for return periods of up to 20 years, and the modified Hershfield equation for longer return periods. For important structures the parameters used in the modified Hershfield equation should be re-derived from an analysis of available short duration rainfall data within the region.

9.10.4 Area reduction factor.

The relationship recommended in Chapter 13 will be adequate for all applications.

9.10.5 Runoff coefficient.

The coefficients used in the recommended methods will be adequate for most applications. For very low risk situations a runoff coefficient of 1,0 should be assumed.

9.10.6 Calculation procedure

There is no point in undertaking detailed analyses for structures which can tolerate a moderate risk of inundation or damage. If correctly used, any of the methods will give acceptable results for design return periods of 20 years or less. The rational method is an obvious candidate for small catchments and is widely used either directly or in a modified form as in some of the urban hydrology models. If hydrograph volumes are of interest a simple triangular shaped hydrograph will be adequate, with the duration of the rising limb being equal to the catchment response time and that of the falling limb from 1,5 to 1,7 times this duration.

For structures with design return periods of 50 years up to 200 years no two methods will give the same results, and large differences in the results can be expected. The user will have to apply his own experience and judgment to obtain a satisfactory solution. The computer programs and manual were developed with this in mind. They relieve the designer of the tedium of calculations so that the user can explore a wide range of alternatives.

9.10.7 Multiple small catchments

Examples are roofed or paved areas, urban or industrial drainage projects, and road culverts.

In category A situations empirical, rule-of-thumb methods such as those specified by some local authorities and based on their own experience will often give satisfactory results. In many cases under-design will result in inconvenience rather than actual damage to the structure or property.

The rational method is the preferred method for small multiple category B catchments. It is particularly applicable to roofed or paved areas where the only unknowns are the time of concentration and the precipitation variables. It would be reasonable to assume a runoff coefficient in the range 0,8 to 1,0 for most small catchments including natural catchments. In some instances, for example urban drainage systems, it may be necessary to use hydraulic routing equations to determine the time of concentration rather than the empirical formulae used in this handbook.

If the problem site is located downstream of a developed area then the effect of the urbanization of the catchment can be very large (nine-fold increases in flood peaks have been reported). This increase is due to shorter catchment response time - and therefore more intense precipitation for a given return period - as well as increased runoff volume associated with the large impermeable areas, particularly in city centres and high density residential areas.

9.11 MEDIUM SIZED CATCHMENTS

(Categories D, E and F in Table 9.4 - no flow records)

9.11.1 Introduction

The following comment is additional to that in the previous section on small catchments. All of the factors discussed there must be addressed.

9.11.2 Recommended methods for return periods up to 20 years

(Category D)

The unit hydrograph method is suitable for catchments in the range 100 km² to 5 000 km² and comes into its own in rural catchments in this category. The rational method can also be used with confidence. There is no evidence to support one method in preference to the other, but the unit hydrograph method has the advantage that the design hydrograph is produced as well.

9.11.3 Recommended methods for return periods from 50 to 200 years

(Category E)

Both of the deterministic methods are based on regionally derived parameters. For important structures an effort should be made to improve the parameter values by carrying out regional calibration analyses based on regional direct statistical analyses following the uMgeni example in the case studies in Chapter 14.

Farm dam spillways are usually designed to pass floods with lower return periods than those of larger structures downstream of them. The possibility of a series of small dams breaching during very severe floods within the catchment of a larger dam and so adding to the estimated peak flow in moderate size catchments should not be overlooked.

9.11.4 Recommended methods for maximum flood estimation

(Category F)

Both the RMF and PMF methods are suitable for catchments in this range. Note that these two methods are not equivalent.

9.12 MEDIUM AND LARGE CATCHMENTS

(Categories G, H and I - records available)

9.12.1 Recommended methods for all return periods

(Categories G and H)

A statistical analysis of available data at the site is undoubtedly the best method, but the statistical properties of the data should be compared with those of comparable catchments in the region to ensure that there are no major anomalies in the data or calibration of the station.

For important structures full regional statistical analyses should be undertaken. The uMgeni case study in Chapter 14 is an example.

9.12.2 Recommended methods for maximum flood estimation

(Category I)

The RMF method can be used with confidence, while the PMF method is problematical for catchment areas larger than 5 000 km^2. Beyond this size the analysis should be undertaken only by experienced hydrologists.

9.13 LARGE CATCHMENTS

(Categories G, H and I in table 9.4 - no records)

9.13.1 Recommended methods for all return periods.

(Categories G and H)

Deterministic methods become increasingly suspect as the catchment area exceeds 5 000 km^2. However, for large catchments estimates of flood volumes and hydrograph shapes are often more important than estimates of the flood peak-frequency relationship. In this situation the unit hydrograph method has a number of advantages in that the catchment can be subdivided, the storms placed in various locations, moved in appropriate directions, and a whole range of likely design hydrographs can then be generated and the effects on the problem structure tested.

If the value of the structure and consequences of failure justify additional computational effort and the use of more sophisticated procedures, a regional statistical correlation model should be developed and regional parameter values determined.

9.13.2 Recommended methods for maximum flood estimation

(Category I)

The RMF is the only reliable method for maximum flood estimation in catchments of this size. Reference should be made to the original publication.

9.14 UPPER LIMIT FLOOD

9.14.1 The probable maximum flood (PMF)

Benson (1964) wrote :

It is necessary that engineers admit to themselves and to the public that there is no such thing as complete safety, and that some degree of risk is involved in any engineering structure, even one whose failure would entail heavy damage and loss of life.

The designer must therefore decide on the extent of the risk that he is prepared to accept and then determine the magnitude of the flood associated with that risk bearing in mind the other risk-related aspects of structural design. If the decision is to be made on an economic basis then the risk is that associated with the minimum total cost.

In the case of a major dam it is very difficult to determine the probable total financial losses associated with the failure of the structure. The position is further complicated by the possibility of loss of life should the dam breach during a flood which exceeds the design flood. (See Chapter 12 for more details.)

However, when State-owned projects are involved the public usually object to being considered as cost statistics. Public pressure has forced Federal agencies in the USA to attempt to determine the maximum possible flood which could occur and use that in dam design. Realizing that no accurate estimate of this figure could be derived, the term *probable maximum flood* (PMF) was coined. Statisticians immediately objected to the terminology. They argued that one can conceive of a maximum possible flood but not a probable maximum flood.

Protagonists of the PMF approach maintain that there must be upper limits to :

(i) The moisture content of air moving over a catchment.
(ii) The rate of movement of this air.
(iii) The fraction of the moisture which can be precipitated.

From this it would follow that there must be an upper limit to the rate of precipitation over the catchment which was termed the probable maximum precipitation (PMP).

Opponents to the concept maintain that the idea of an upper limit to any natural phenomenon is incompatible with scientific thought, and that there is an ambiguous relation with the concept of the design of major structures (see Benson 1964).

Even if the probable maximum precipitation could be determined, the catchment would have to be in optimum runoff producing conditions (ie completely saturated) at the beginning of the flood. The joint probability of maximum precipitation falling on a large, completely saturated catchment is so small that the risk must be far less than that associated with most other possible causes of structural failure. It may therefore be argued that there is little logical basis for the use of the PMF as a design criterion. However, an authority may make a *policy decision* that the PMF be used, but it should not be assumed that the calculated value is indeed the absolute maximum possible flood, but rather that it is a value having a close to zero probability of being exceeded.

The ASCE Task Committee on the re-evaluation of the adequacy of spillways of existing dams suggested that a 1 in 10 000 year flood could be used as a substitute for the PMF although there were also objections to this. (Task Committee 1974).

The April, 1988 bulletin of the Australian National Committee of the International Commission on Large Dams includes papers and discussions of a workshop on spillway design floods. From this it is clear that although the PMP/PMF concept is alive and flourishing in Australia and the USA, practitioners in those countries are becoming increasingly concerned about the ultraconservatism and consequently very high cost of making provision for upper limit floods, and also because the supposed upper limit increases with each new improvement to the method.

The following short quotes from the publication illustrate these concerns :-

> *".... 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).*

> *"The term PMF could have two interpretations. It could refer to an absolute upper limit of flood discharge with zero probability of exceedance, or to a 'reasonable' value of very low probability. .... the latter interpretation is intended in the 1987 ARR .... Unfortunately, it is unavoidable that what is 'reasonable' is a subjective decision, but uncertainty and the need for such decisions are necessarily involved with design for rare extreme events. .... it should be noted that not only is the intention not to provide complete safety, but that it is unattainable" (Pilgrim).*

> *"There is a growing need for estimates of the probability of PMP" (Kennedy, et al).*

> *"The risk of dam failure does not change and the PMP does not change. It is just the estimates of PMP which change" (Robinson).*

> *"The revised probable maximum peak discharge .... is approximately 45 times the highest recorded flow at the gauging station upstream of the site and nearly three times greater than the original PMF estimate". (McConnell and Hausler).*

> *"The increasing references to the AEP of the PMP logically demand that the whole 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).*

9.14.2 ICOLD congress (San Francisco, 1988)

Bouvard (1988) was the general rapporteur for the session on *The design flood and operational flood control* of the sixteenth congress of the International Commission on Large Dams (ICOLD). A record 93 papers on this subject were presented at the congress, and they are a good cross-section of current international practice.

In his 83-page summary Bouvard directly addressed the question of whether or not flood flows are bounded. Although there were a number of the papers which described the use of PMP/PMF for dam design, there were other authors who were more critical. Bouvard selected the following quotations from papers presented at the congress :

"The PMF is based on the PMP and the determination of the PMP and the runoff coefficient ..... is very speculative .... The PMF may be not only highly overestimated but also seriously underestimated."

"PMF values are more tied to computational procedures than they possess physical or statistical significance."

"Only meteorology services have the expertise and equipment to determine the PMP. Computing the PMF from the PMP is a complicated exercise requiring evaluation of the numerous physiographic parameters that describe the basin.

"ANCOLD believes it is inappropriate for advisers such as meteorologists or hydrologists to influence the safety margin by unreasonably compounding low probability events."

"No basic analysis of the underlying physics is made, no substantial field observations are available, yet attempts are made to insert the convergence effect, the area where the dew point is reached, the speed of the wind and its effect on the dewpoint, not to mention orographic effects."

Bouvard then continued with his own comment:

"'Reasonable' or a similar word is found in virtually all descriptions of PMP-PMF applications, and this is disturbing inasmuch as reasonableness is not defined. Not infrequently checks are indeed made to see how 'reasonable' a PMF appears when compared with some reference figure, eg the highest recorded flood. If the PMF is three or four times higher, sighs of relief are emitted by all concerned. Failing which, a 'reasonable' result can readily be achieved by changing some of the many constants in the PMP-PMF evaluation procedure. Not infrequently then, a result may depend less on the procedure than on what one feels might be meant by reasonableness.

Bouvard's parting shots on the PMF were:

"Statistical methods of flood evaluation, ... consider that floods are not upper bounded, implicitly prohibiting absolute safeguards. Deterministic PMP-PMF methods on the other hand, postulate the existence of an upper bound and implicitly allow them. Hence, a real incentive to select a PMF design flood - a 'Psychological Maximum Flood' offers considerable peace of mind. ... The problems however are virtually insoluble and lead to the wide range of partial answers that now abound."

On the other hand Bouvard referred favourably to the paper by Alexander and Kovács (1988) on our South African experiences where we commented that " *The weaknesses in conventional hydrologic analysis were exposed .... and the need to rely on regional maxima rather than conventional statistical and deterministic methods was reinforced."*

9.14.3 Philosophy and application of PMP/PMF estimation methods

Readers who nevertheless may wish to investigate the application of the PMP/PMF estimation methods in southern Africa will find the following publications helpful:

- World Meteorological Organization (1986). This is the internationally recognized method for determining the PMP and is referred to by most authors on the subject.

- Hansen (1987) provides a historical review and describes methods used for PMP estimation in the USA. In the same issue of the Journal of Hydrology Cudworth and Stallings detail the methods used by the US Bureau of Reclamation, and US Army Corps of Engineers respectively.

- Practical applications are detailed in several papers presented by Wang eg Wang (1984), Wang and Jawed (1985) and Wang (1988)

9.14.4 The HRU 1/72 method for determining the PMF

The only accepted method for determining the PMF in South Africa is that given in HRU 1/72 and detailed in Chapter 7 on deterministic methods. It is based on envelopes of point precipitation values and minimum storm losses shown in Figs HRU C.4 and G.1 respectively. These are reproduced in Chapter 13. The weather services in the USA and Australia have produced estimates of probable maximum precipitation storms which include the time and space distribution of PMP storm rainfall. Equivalent South African storms are detailed in Appendix D of HRU 1/72 as well as in the technical report series of the Department of Water Affairs.

9.14.5 Regional maximum flood (RMF)

In South Africa the method for determining the RMF is detailed in the Department of Water Affairs' Technical Report TR 137 and described in Chapter 8 on empirical methods. The map showing the *k*-factors has been subsequently updated and is shown in Chapter 13.

The RMF approach should not be confused with the PMF approach. The RMF is based on *observed* maximum floods while the PMF is based on *estimated* upper limit floods which in turn are based on *estimated* maximum possible rainfalls derived from observed maximum rainfalls and maximized observed storms.

The RMF offers several advantages over the PMF as a rational basis for the determination of the upper limit flood for dam design. The RMF is an upper envelope of floods *that have occurred* in the region and is consequently a reasonable *lower limit* to the estimate of the maximum flood that could reasonably be expected to occur at the site during the life of the structure. Similarly, it would be unreasonable to recommend a design flood greater than a flood that has been experienced anywhere in the world for a catchment of the same size. The design maximum flood should have a *k*-value somewhere between the RMF given by Kovàcs (1988) for the region and a value of k=6,0 based on world maxima. This is still a wide range, and there is merit in incrementing the *k*-value given in Kovàcs in all cases except strategic dams or dams whose failure can not be tolerated under any circumstances, Vaal Dam being an example where other criteria should be used. Note that it is more logical to apply a factor to the *k*-value than to the resultant flood estimate, as this retains the ability to make a direct comparison of the relative magnitude of the result with that of floods experienced in the region, subcontinent, or elsewhere in the world.

It must be borne in mind that there are other inbuilt factors of safety in dam design and that hydrologists should heed the request that they should not unreasonably compound low probability events. This aspect is discussed again in Chapter 12 on dam safety.

9.15 SPILLWAY DESIGN FLOOD

The determination of the spillway design flood for a major dam where the failure of the dam could result in loss of life, and over design could result in the project not being economically viable is the most severe situation likely to be encountered in flood risk analysis. The case study for the uMgeni River dams in Chapter 14 was chosen to demonstrate the recommended methodology. In situations where only very low risks can be tolerated then a much more comprehensive risk analysis should be undertaken which includes all factors which could result in the failure of the structure. This analysis requires expert knowledge in several fields, only one of which is hydrology.

More details are provided in Chapter 12.

9.16 SPECIAL APPLICATIONS

9.16.1 Transport route location and bridge design

Until very recently flood risks have not been accommodated in bridge design other than to allow for the passage of the design flood (Liebenberg, 1988) notwithstanding the recommendations and examples in HRU 1/72. Nor has the risk of traffic interruption been accommodated in methods used for transport route location (Edwards and Stanway, 1988). The 1987-88 floods destroyed several major bridges and caused widespread dislocation of road and rail communication systems. The civil engineering profession is currently re-examining bridge design standards to allow for the loads on bridge structures caused by floods as well as the risks of interruption of communication systems.

The bridge designer's objective

The bridge designer's objective is to find the most economical solution within *tolerable* safety limits. It is very important to note that absolute safety can never be guaranteed. The designer has to select the level of risk that can be tolerated for each major component of the structure separately in order to achieve a minimum cost solution within the bounds of budget constraints and minimum safety requirements for the structure as a whole, and the design standards for structural elements.

Numerical simulation models using the appropriate random number generators described in the annexures to Chapter 2 are useful tools for this type of analysis. It is a simple matter to incorporate both economic and risk factors in a single model which can be used to determine the optimum economic route location at river crossings as well as the optimum economic bridge design within specified constraints.

The transportation engineer's objective

A challenging application of flood hydrology is the determination of the risk of traffic interruption along a major highway or system of trunk roads due to floods.

The transportation engineer's objective is the determination of most economic long term location and design standards that include the actual and inconvenience costs to road users during the occasions when routes are out of commission due to road inundation or flood damage.

The risk of traffic interruption along a route is not the same as the design risk of the structures along the route. The actual risk depends on the degree of independence of the flood-producing mechanisms at the various sites. For example it would be logical to assume that a single storm would be the most likely cause of the design capacity of a series of road culverts being exceeded, whereas it is unlikely that this would be the case for bridges crossing different rivers hundreds of kilometres apart. In practice the probability of interruption would lie between the two extremes of independence and direct correlation of flood-producing storm mechanisms along the route.

In theory each catchment has its own combination of critical depth-area-duration rainfall characteristics which could lead to the conclusion that floods generated in a small catchment are statistically independent from those of an adjacent much larger catchment. It is a matter of observation that this is not so in the real world as was evidenced in the extreme floods described in the annexures to Chapter 3. This illustrates once again that severe floods are caused by prolonged and widespread rains rather than by storms having properties unique to a specific catchment.

The solution to the transportation engineer's problem lies in the study of geographically centered storm rainfall over a wide area. This is feasible only if rainfall observations are based on groups of ground stations combined with weather radar and satellite imagery. Such research is difficult and still in its infancy.

9.16.2 Agricultural lands

The US Soil Conservation Service (SCS) method should be used for catchments with areas less than 8 km^2 - see Schulze and Arnold (1979) and subsequent publications by Schulze and coauthors. For larger catchment areas the rational method should be used.

9.16.3 Pans and lakes with no outflow

The water levels in water bodies within enclosed catchments depend almost as much on the antecedent precipitation and evaporation as on the storm event itself. Probably the best approach is to determine an approximate *volumetric* relationship between precipitation and runoff based on recent observations, and then to use the historical precipitation record to simulate the behaviour of the lake over a long period of time. A statistical analysis of the resulting water levels will provide a base for extrapolation to longer return periods.

9.16.4 Flood lines along public streams

In terms of the South African Water Act, no township may be established or extended unless lines are shown on a general plan of the township indicating the maximum flood levels likely to be reached on average once every twenty years.

The Act does not specify the method to be used for determining the flood levels other than stating that it shall be to the satisfaction of the authority from whom approval for the establishment of the township has to be obtained.

For most applications it would be reasonable to assume that the flood levels are those which would be reached by a constant flow equal to the instantaneous peak that would be generated in the catchment. This assumption overcomes the problems associated with attempting to route flood hydrographs through the system.

Presumably existing development within the catchment, particularly urban development, as well as existing structures in the stream channel have to be taken into account when determining the flood peak and the corresponding flood level respectively. The Act does not specify whether or not possible future development or structures have to be considered.

9.16.5 Dolomitic areas and arid regions

These regions can be identified by the absence of clearly defined drainage channel networks. While floods can and do occur, the inundation is usually confined to local low-lying areas. If part of a catchment consists of an area of undefined drainage then this area should be excluded. River flow in these regions is erratic and conventional analyses are likely to be suspect. Flood estimates are best based on historical flood levels identified by local residents at or near the site and a subjective factor applied depending on the risk that can be tolerated.

9.17 SUBSEQUENT EVALUATION

Where important structures are involved every effort should be made to commence gauging at the site as part of the development of the project. This could provide a valuable final check on the assumptions used in the calculations.

It would also be a worthwhile exercise to re-assess the flood risks at major dams once per decade based on the additional data collected in the interim, and the latest available methods of analysis.

9.18 A FINAL WORD OF CAUTION

There is a growing feeling among a number of hydrologists and engineers that the unusually high precipitation events over many areas of South Africa occur more frequently than the pure statistical analyses of either precipitation or runoff data would indicate.

The Weather Bureau publication on extreme values of rainfall (Weather Bureau, 1974 p. 16) states *"For Port Elizabeth .... (the) monthly expected values are too small compared with the monthly recorded values because an exceptional single high maximum rainfall occurred in almost every month with a return period of hundreds to thousands of years"*.

The Nahoon River flood of August, 1970 had a mean return period of 1 200 years based on seven different calculation methods. This was a well documented event and two major dams on the adjacent Buffalo River (Bridle Drift and Laing) also recorded floods of the same relative magnitude.

The Pretoria storm of January, 1978 was probably the best documented severe storm in South Africa, both as far as precipitation and stream-flow are concerned. Radar observations also covered the early development of the storm. Another well observed (radar) event was the cloud merger in the Nelspruit area during which 100 mm of precipitation was recorded in 16 minutes.

Both the Pretoria and Nelspruit storms resulted from unusual but by no means very rare meteorological conditions. Similarly, the exceptional floods in the Port Elizabeth, East London and Hluhluwe areas resulted from meteorological phenomena which are not as rare as statistical analyses based on precipitation maxima or annual flood peaks at individual sites would indicate. The more recent floods in the south-eastern Transvaal and northern Natal in January and February of 1984 caused by the tropical cyclones Domoina and Imboa, the latter following close on the heels of Domoina, also had very long return periods based on statistical analyses. This was also the case in the damaging Laingsburg floods of January, 1981.

The basic assumption of statistical analyses and indeed in all methods based on historical data is that the statistical population of floods at a site (the series of annual maxima) result from a single type of meteorological phenomenon and are consequently from a homogeneous statistical population. This is clearly not the case over most of South Africa. Unfortunately (from a hydrological point of view) these floods are too infrequent to provide an adequate basis for direct statistical analysis, while meteorologists are as yet unable to determine the magnitude-frequency relationships of the causative storms.

This problem is dealt with in Chapter 6 on regional statistical analysis methods. As there is as yet no statistical or deterministic method that can deal adequately with these mixed populations, the only recourse for the designer is to include appropriate factors of safety in the design of the structure if only a very low risk of failure can be tolerated.

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Chapter 10

FLOOD ROUTING AND FLOOD CONTROL MEASURES

Contents

10.1 INTRODUCTION

The flow in a river varies continually with time. This variation is important when the flow is considered from a hydraulic point of view. If the variation of the flow rate is very slow it may be assumed that the principles of steady flow are applicable. In many cases, however, it is not possible to adopt the assumptions associated with steady flow and it must be accepted that the problem falls in the domain of unsteady flow. An example of this is the prediction of the change in shape and height of a flood wave which is moving down a river. The determination or prediction of this change is certainly one of the most important applications of the principles of unsteady flow and is generally known as flood routing.

There is a difference between flood routing gradually varying flow and the more dramatic cases of rapidly varying unsteady flow. Examples of the latter are the dam-break wave and a surge moving upstream as a result of an earth movement suddenly causing an obstruction in a canal or river. Normally the rising and falling surfaces of the wave analyzed by flood routing are far more gradual than in the latter examples. The computation method must therefore be appropriate for the case being studied.

Flood routing is applied mainly to problems in the following three categories. Firstly, it is used to predict how a flood hydrograph will change if it moves through a dam. Secondly, it is used to evaluate the effect of flood peak reduction works in their ability to modify flood hydrographs. Thirdly, it is employed in the field of hydrology in the synthesis of flood hydrographs. In the last application the general concept of flood routing is extended to include the conversion of rainfall to runoff.

It is thus clear that flood routing finds application in the field of hydrology as well as in hydraulics. Consequently, two approaches to flood routing have arisen, viz the 'hydraulic' and 'hydrologic'. The hydraulic approach is based on the solution of basic differential equations which describe unsteady flow in open channels while the hydrologic approach contains certain simplifications. The latter approach is simpler in general but does not give satisfactory results for more complex problems. Therefore where backwater and storage effects become important, only the hydraulic approach will be able to give accurate results.

This chapter is limited to hydrological applications. Hydraulic flood routing procedures are addressed in a separate SANCOLD document.

10.2 HYDROLOGICAL FLOOD ROUTING METHODS

In general, detailed knowledge of the channel geometry is not needed in the hydrological flood routing methods. An empirical relationship between channel storage and discharge can be developed, based on data from previous flood events.

Consider the simple case of a river reach when no losses take place as the result of percolation or evaporation and there is also no lateral flow from tributaries or seepage from the banks of the river. If a flood moves past point A, the same volume must pass point B further downstream although the flow patterns at the two points will differ. The peak flow at the downstream point occurs later and is smaller than that at point A. The two different hydrographs are shown in Fig 10.1(a) as the inflow and outflow hydrographs for the particular river reach. The reason why the two hydrographs are not the same is that some water must temporarily fill certain storage in the river reach. The difference between the ordinate values of the inflow and outflow hydrographs in Figure 10.1(a) represents the rate of change of temporary storage. This is shown as a function of time in Fig 10.1(b).

Where the inflow and outflow hydrographs intersect at point *I*, the rate of change in storage is equal to zero. At this point inflow is equal to outflow.

The cumulative area under the curve in Fig 10.1(b) represents the storage volume at time t after the beginning of the flood. A plot of this volume against time gives a storage-volume curve as shown in Fig 10.1(c). This curve has the property that its maximum value occurs when the total inflow is equal to the total outflow.

The outflow hydrograph can be derived by manipulation of the continuity equation.

In Fig 10.1 the rate of change of storage is calculated from the continuity equation :

$$I - O = \frac{dS}{dt} \tag{10.1}$$

If expressed in finite difference form :

$$\frac{I_1 + I_2}{2} - \frac{O_1 + O_2}{2} = \frac{S_2 - S_1}{t_2 - t_1} = \frac{\Delta S}{\Delta t} \tag{10.2}$$

where I = rate of inflow to the reach (m^3s^{-1})
O = rate of outflow from the reach (m^3s^{-1})
S = the stored volume within the reach at time t (m^3)
t = the time from an arbitrary origin

The subscripts *1* and *2* refer to successive values and Δ denotes the difference between successive values.

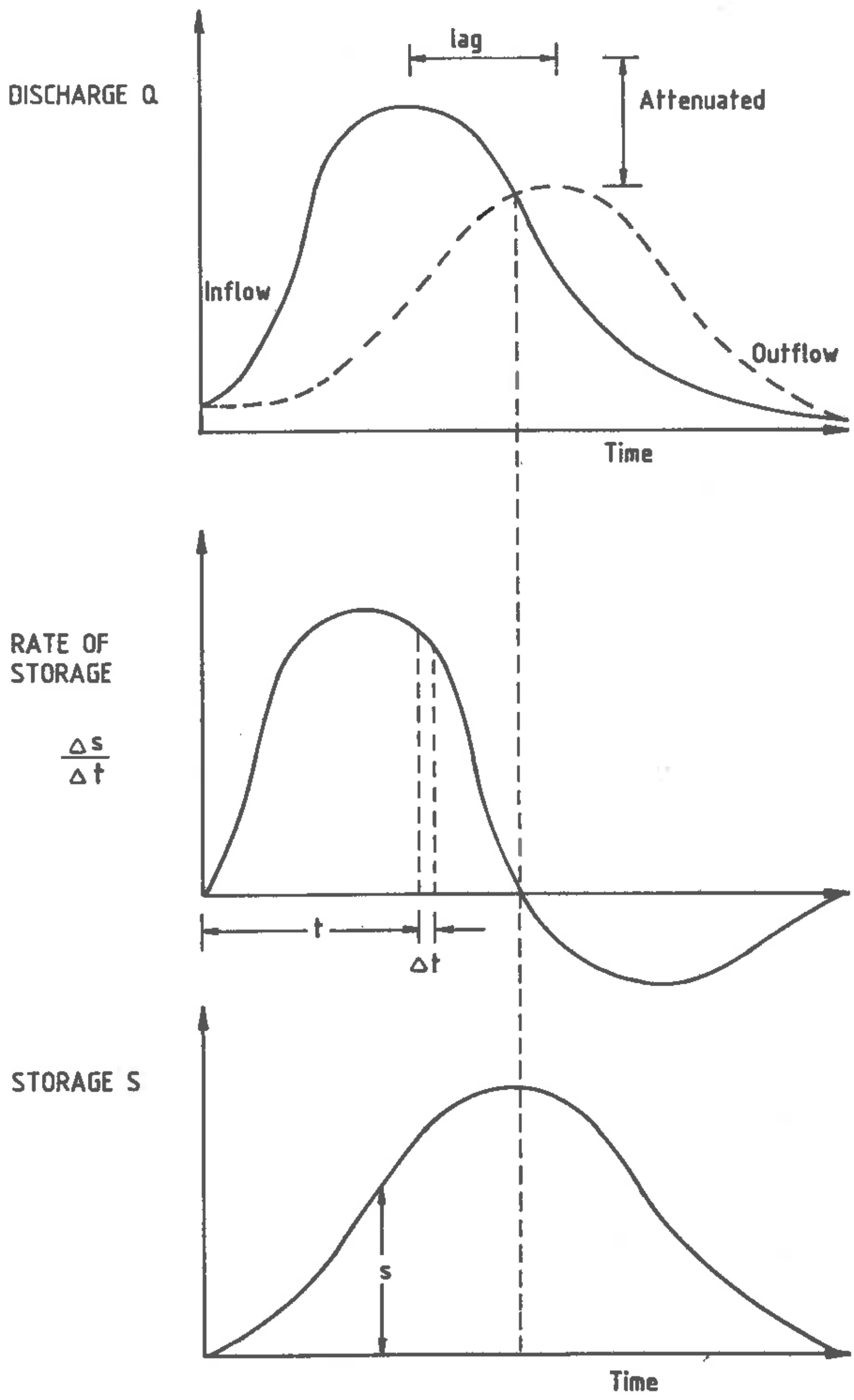

FIGURE 10.1 : Relationships between inflow, outflow and storage in a channel reach due to a passing flood.

In equation (10.2), there are two unknowns, O_2 and S_2. To calculate the outflow hydrograph (O_2), further information about the storage within the system is needed. This information can be determined from detailed maps but this can be a time consuming task. An alternative approach is to analyze observed flood hydrographs and to establish the behaviour of storage as as function of discharge. The best known method was developed by Mc Carthy and applied to the Muskingum River and is today known as the Muskingum method of flood routing. It can be derived from basic principles as follows :

From equation (10.2) :

$$O_2 = I_1 + I_2 - O_1 - 2\left(\frac{S_2 - S_1}{\Delta t}\right) \tag{10.3}$$

For uniform flow conditions the discharge in a river, O_1 can be calculated by using the Manning equation :

$$Q = \frac{A}{n} R^{2/3} \bar{S}^{1/2} \tag{10.4}$$

where
- A = cross-sectional area (m²)
- R = the hydraulic radius (m) = A/P where P = wetted perimeter
- $\bar{S}$ = the slope of the energy line
- n = the Manning roughness coefficient

If the section is wide and the depth reasonably constant:

$$A = B\,y$$

$$P = 2y + B \approx B$$

$$\therefore \quad \frac{A}{P} = \frac{By}{B} \approx y = \textit{hydraulic depth}$$

Then
$$Q = \frac{By}{n} y^{2/3} \bar{S}^{1/2}$$

$$= \frac{B\bar{S}^{1/2}}{n} y^{5/3}$$

$$= a\,y^u \tag{10.5}$$

where
$$a = \frac{B\,\bar{S}^{1/2}}{n}$$

and
$$u = 5/3$$

Storage volume
$$S = B\,\ell\,y^m = b\,y^m \tag{10.6}$$

where m is a function of channel geometry and

$$b = B\ell$$

For rectangular section $m = 1$

During the passage of a flood wave, the rate of inflow is not equal to the rate of outflow. Thus the actual storage S, in a reach, may be assumed to be a weighted mean of both S_i and S_o.

S_i = Storage represented by depth at the point of inflow;
and S_o = Storage represented by depth at the point of outflow

Thus $$S = S_i X + (1 - X) S_o$$

Substitute Si and So

$$S = X b \left(\frac{I}{a} \right)^{m/u} + (1 - X) b \left(\frac{Q}{a} \right)^{m/u}$$

For a river : $m/u = X$ and $b/(a)m/u = K$

Thus $$S = K [X I^x + (1 - X) O^x]$$

For a rectangular channel, since $m \approx 1$

$$x = \frac{m}{u} \approx 0.6$$

For a natural channel, $m > 1$

$$X = \frac{m}{u} \approx 1$$

$$S = K [X I + (1 - X) O] \qquad (10.7)$$

which is the Muskingum storage equation.

Substituting (10.7) in the continuity equation (10.3) :

$$O_2 = I_1 C_1 + I_2 C_2 + O_1 C_3 \qquad (10.8)$$

where $$C_1 = \frac{(KX + 0.5\ \Delta t)}{(K - KX + 0.5\ \Delta t)} \qquad (10.9)$$

$$C_2 = \frac{(-KX + 0.5\ \Delta t)}{(K - KX + 0.5\ \Delta t)} \qquad (10.10)$$

$$C_3 = \frac{(K - KX - 0.5\ \Delta t)}{(K - KX + 0.5\ \Delta t)} \qquad (10.11)$$

and $$C_1 + C_2 + C_3 = 1 \qquad (10.12)$$

The X-value in the Muskingum equation must be regarded as a dimensionless weighting factor used to weight the storage according to the inflow and outflow conditions in order to determine the average for the reach.

The K-value is related to the time of travel of the flood wave through the reach under consideration, hence the unit of time.

10.3 APPLICATION OF FLOOD ROUTING PRINCIPLES

10.3.1 Routing through dams

One of the simplest cases of flood routing is the routing of a flood hydrograph through a reservoir with a small surface area, where it can be assumed that the water surface is horizontal. In this case the additional storage below the backwater curve can be neglected, and balancing of the inflow, outflow and the volume of water in storage becomes a simple exercise. This is simply the application of the continuity principle which can be expressed as follows :

Inflow volume - outflow volume = change of the volume in storage. In terms of equation (10.2) :

$$\left[\frac{I_1 + I_2}{2} \right] \Delta t - \left[\frac{O_1 + O_2}{2} \right] \Delta t = S_2 - S_1 \qquad (10.13)$$

The second principle applied is the dynamic equation which is governed by outflow conditions. At any dam with or without gates, the rate that water can be released is determined by the level of the water in the dam and therefore indirectly by the volume of water in storage. This leads to a special condition namely that the rate of outflow will reach a maximum when the inflow rate is equal to the outflow rate. This is also apparent from the typical inflow and outflow hydrographs in Fig 10.2.

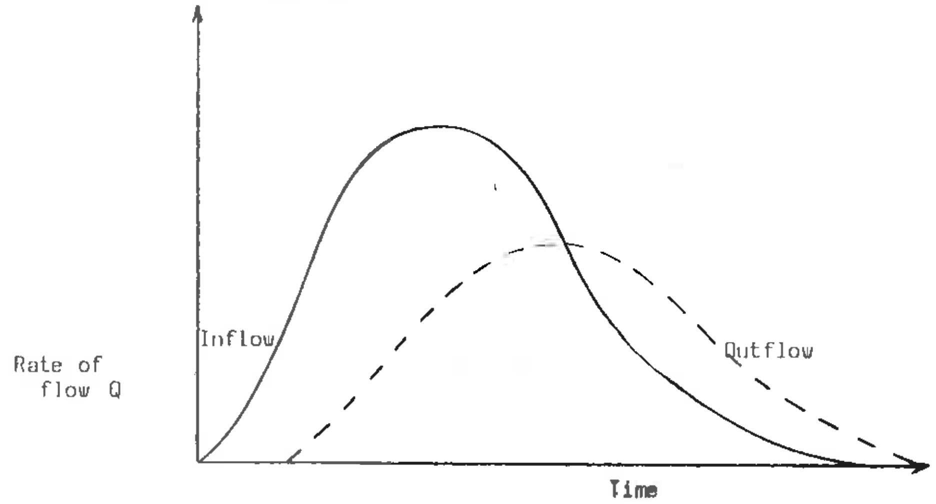

FIGURE 10.2 **Inflow and outflow hydrograph for a dam without flood control gates**

This principle also forms the basis for the development of reservoir operating rules which will be dealt with in paragraph 10.4.

Flood routing calculations can best be understood by working through an example. The following hydrograph is measured upstream of a dam such that $K = 24$ hours:

TABLE 10.1 Inflow hydrograph

Interval	Time	Inflow (m^3s^{-1})
1	0:00	100
2	3:00	250
3	6:00	500
4	9:00	1 500
5	12:00	1 800
6	15:00	1 600
7	18:00	1 050
8	21:00	650

Assume the spillway and dam basin characteristics fit the Muskingum model and calculate the outflow hydrograph. Also assume the dam is already overflowing at interval 1 by 100 m^3s^{-1}).

Solution :
From equations (10.8), (10.9), (10.10) and (10.11).

$$O_2 = I_1 C_1 + I_2 C_2 + O_1 C_3$$

where $$C_1 = \frac{(KX + 0.5\ \Delta t)}{(K - KX + 0.5\ \Delta t)}$$

$$C_2 = \frac{(-KX + 0.5\ \Delta t)}{(K - KX + 0.5\ \Delta t)}$$

and $$C_3 = \frac{(K - KX - 0.5\ \Delta t)}{(K - KX + 0.5\ \Delta t)}$$

In the case of a dam, the outflow rate is solely dependent on the water level in the dam, as is the storage volume. Thus the value of X is zero.

If $X = 0$
$K = 24$ hours
$\Delta t = 3$ hours
Then $C_1 = 0{,}059$
$C_2 = 0{,}059$
$C_3 = 0{,}882$

Calculate the outflow hydrograph using equation 10.18 :

O_2 = (100 x 0,059) + (250 x 0,059) + (100 x 0,882)
= 108,9 m^3s^{-1}
O_3 = (250 x 0,059) + (500 x 0,059) + (108,8 x 0,882)
= 140,1 m^3s^{-1}
O_4 = (500 x 0,059) + (1500 x 0,059) + (140,1 x 0-,882)
= 241,3 m^3s^{-1}
. .
. .
O_8 = 672,9 m^3s^{-1}

TABLE 10.2 Inflow and outflow hydrographs

Interval	Time	Inflow (m^3s^{-1})	Outflow (m^3s^{-1})
1	00:00	100	100
2	03:00	250	109
3	06:00	500	140
4	09:00	1 500	241
5	12:00	1 800	407
6	15:00	1 600	559
7	18:00	1 050	649
8	21:00	650	673

It should be noted that the Muskingum reservoir routing has been provided to illustrate the method and should not be used for normal reservoir routing unless the parameters correspond. In general, a routing procedure considering the storage-capacity and stage-discharge relationships should be used for reservoir routing.

The procedure can also be used in reverse for measurement purposes when the inflow hydrograph is unknown. The water level is observed, say hourly, and assuming that the spillway is calibrated the outflow at the end of each time period can be calculated. By means of the capacity curves, the inflow volumes can then be calculated. The inflow volume for each time period is then divided by the length of the time interval in seconds to obtain the average inflow rate (m^3s^{-1}) for each time interval.

Certain assumptions must be made in practice eg that the water level rises evenly over the whole surface during time intervals. This is not correct because a certain volume goes to temporary storage in the backwater and is not accounted for. Errors can occur, especially when calculations are based on a single observation at the wall. Wind action could also contribute to large errors. A wind blowing towards the dam wall could cause a too high water level reading at the wall which would result in an unrealistic high short term calculated inflow rate and vice versa. Seiching due to flood wave surges, gate operation, wind and pressure disturbances can also occur and may give anomalous results.

10.3.2 Routing in rivers

Flood routing along a river has much in common with routing through a dam as described above. When a flood wave moves through a certain river reach the peak is attenuated and delayed because of friction and storage that takes place in the river course and flood plains. As for dams, the continuity principle holds. If this is applied to a river reach, then :

In a given time increment :

Inflow volume - outflow volume = change in volume in storage.

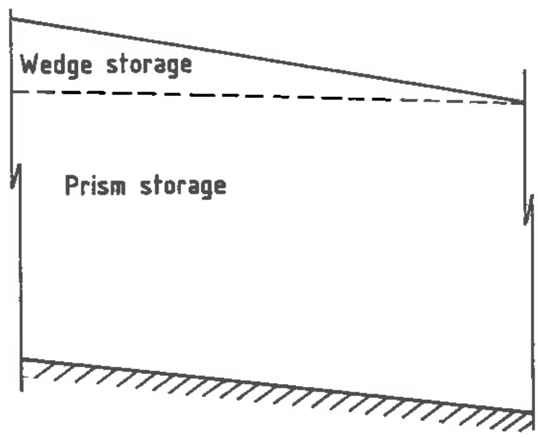

FIGURE 10.3 Wedge storage and prism storage in a river reach

Direct application of the dynamic equation is cumbersome in this case. It can not be generally said that the outflow rate from a reach is governed by the volume of water in storage. The outflow can be approximately related to the downstream flow depth. Similarly, the inflow can approximately be related to the upstream flow depth which means that the volume in storage in a particular reach depends upon the inflow to and the outflow from the reach. Because of this the storage can be divided into two components viz prism storage w which is dependent only on outflow and wedge storage which depends on (I - O). This leads to equation (10.7) as previously stated, ie

$$S = K[O + X(I - O)]$$

Worked example.
The following hydrograph is measured at a gauging structure on a certain river :-

TABLE 10.3 Flow hydrograph		
Interval	Time	Inflow (m^3s^{-1})
1	12:00	100
2	15:00	250
3	18:00	500
4	21:00	1 500
5	24:00	1 800
6	03:00	1 600
7	06:00	1 050
8	09:00	650

Assume the geometry of the river reach is such that $K = 3{,}5$ hours and $X = 0{,}4$. Assume the base flow in the river is 100 m^3s^{-1}. Calculate the expected hydrograph at the outflow section of the river.

Solution :
From equations (10.9), (10.10) and (10.11) for

$$K = 3{,}5, \quad X = 0{,}4 \quad \text{and} \quad \Delta t = 3 \text{ hours}:$$

$$\begin{aligned} C_1 &= 0{,}805 \\ C_2 &= 0{,}028 \\ C_3 &= \underline{0{,}167} \\ &\ \underline{1{,}000} \end{aligned}$$

Use equation (10.8) to calculate the ordinate values of the outflow hydrograph:
Summary :

TABLE 10.4 Inflow and outflow hydrographs			
Interval	Time	Inflow (m^3s^{-1})	Outflow (m^3s^{-1})
1	12:00	100	100
2	15:00	250	104
3	18:00	500	233
4	21:00	1 500	483
5	24:00	1 800	1 339
6	03:00	1 600	1 718
7	06:00	1 050	1 604
8	09:00	650	1 131

10.3.3 Restrictions

(i) Boundary values for the value of K :
Assume K = 10 hours and X = 0,4 in the example in paragraph 10.3.2. The calculated outflow hydrograph is as follows :

TABLE 10.5 Outflow hydrograph		
Interval	Outflow (m^3s^{-1})	
1	100	(100)
2	50	(101)
3	47	(80)
4	-105	(125)
5	437	(140)
6	1 049	(696)
7	1 453	(1 273)
8	1 424	(1 566)

It is impossible to have negative outflow values. This is a common problem when using a K-value which is large in comparison with Δt. The implications are that it is impossible to route a flood over a long distance where the lag-time is high. To overcome this shortcoming, the river reach must be divided into sub-reaches. The hydrograph is then routed through the first sub-reach and the outflow of the first reach becomes the inflow to the second reach. The values in brackets above, were calculated by using two sub-reaches with K= 5 hours and X = 0,4 for each reach.

Although the negative values have disappeared, there is still a decline in the flow between interval 2 and 3. Unlike the previous negative flow rates, this decrease could be explained.

It is, however, not very likely that a base flow of 100 m^3s^{-1} could suddenly reduce to 80 m^3s^{-1} within 6 hours. This indicates that the mathematical model still does not reflect the true situation. The solution is to use either longer time intervals (Δt) or to reduce the value of K even further by choosing more sub-reaches. The latter is the obvious choice because coarser Δt values would affect the accuracy of the answers (see paragraph 10.3.4). Satisfactory results would be obtained if the value of K complies with the following :

$$K \leq 0.5\ \Delta t/X \tag{10.14}$$

$$K > 0.5\ \Delta t/(1 - X) \tag{10.15}$$

(ii) River losses and contributions :
The Muskingum model does not make provision for river losses. In fact, the whole model is based on the conservation of mass. Evaporation, seepage, pumping activities in the reach etc. could be significant in low-flow routings. The user must decide whether this is an important consideration and make the necessary provision in the form of deducting an acceptable value from each outflow value for each time interval. Equation (10.8) could for instance be adapted as follows :

$$O_2 = I_2\,C_1 + I_2\,C_2 + O_1\,C_3 - D\,O_1 \tag{10.16}$$

where D = some acceptable loss function.

If any tributaries add water to the reach under consideration, the principle of superposition could be used. The hydrograph in the main channel is routed in the usual way. The hydrograph from the tributary is routed separately, from its point of measurement, past the confluence up to the point of the required outflow hydrograph. At this point the ordinate values of the two separate hydrographs are added to provide the full outflow hydrograph.

This is not very satisfactory, because the geometry of the tributary channel could be very different from that of the main channel. Because the value of K and X are dependent on the channel geometry, a very significant change in both of these values could occur at the confluence. A more acceptable approach would be to route the tributary hydrograph to the confluence, superimpose it at this point on the hydrograph in the main channel and route the combined hydrograph then to the end of the reach. This however, is not possible because of the restriction described in (iv). The solution to this problem is also given in (iv).

(iii) *The Manning equation*
As indicated in paragraph 10.2, the Manning formula plays a role in the derivation of the Muskingum equation. The Manning equation is valid only for steady uniform flow conditions, whereas flood routing is in the domain of unsteady non-stationary flow. It is not correct to assume that there is an invariable relationship between stage and discharge at

ends of routing reaches for the rising and falling portions of the hydrograph. The same applies to the relation between stage or discharge at the lower end of the reach and the volume of storage in the reach.

The reason for this is because of the well known hysteresis effect encountered in the calibration curve of a discharge measuring point in a river.

If a complete set of discharge measurements are taken at a point for a specific hydrograph, the relationship between discharge and flow depth may form a loop as shown in Fig 10.4.

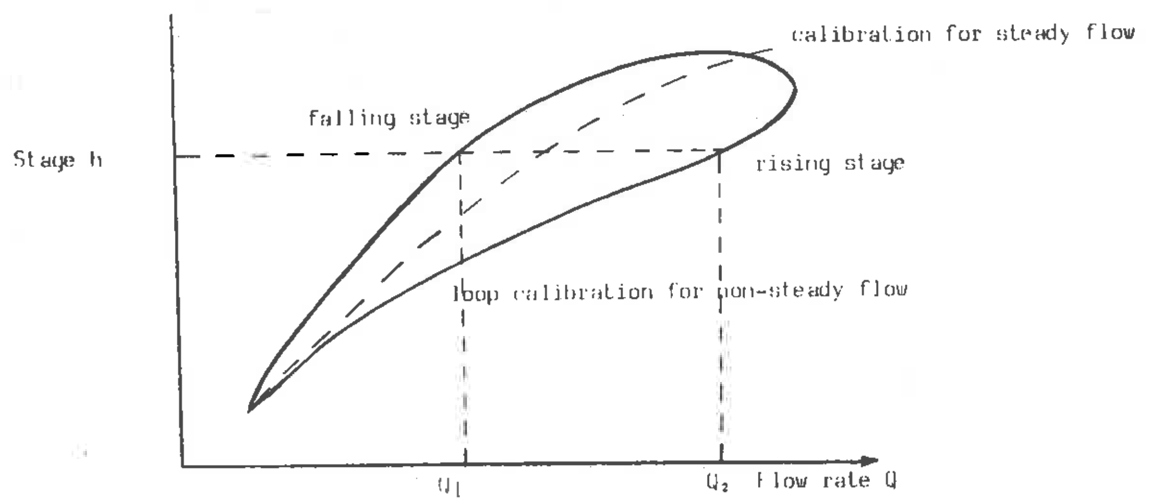

FIGURE 10.4 The loop calibration curves

The implication of this loop calibration is that the discharge rate Q_2 for the rising limb of the hydrograph is larger than the discharge rate Q_1 for recession part of hydrograph, both measured at the same stage reading. This is due to a difference in the slope of the water surface for a rising and falling stage. This must therefore also have an effect on the inflow and outflow rates of reaches and the available wedge storage. The dimensions of this loop are a function among other factors, of the rate of rise and fall of the water surface (dh/dt) at the observation point. The Muskingum model cannot accommodate this phenomenon. It implies that different K and X values should be used for the rising and falling limbs of the hydrograph. The hydraulic approach, on the other hand does account for this in the differential equations for unsteady non-uniform flow.

(iv) Flood routing over long distances

Flood routing over long distances can create problems. The multiple method of routing must be used as already described. In calibrating the model, a K and X value are determined such that the calculated outflow hydrograph best fits the observed hydrograph. Say for instance that an observation point is available halfway between the point of inflow to this long river reach, and the point of outflow. If the specific intermediate hydrograph calculated by means of the multiple routing method could be compared with the actual hydrograph at this midpoint, it would be a pure coincidence if they compare favourably. The reason for this is because the K and X values are manipulated in a "black box" fashion to give the best end result without any concern about intermediate relationships. This poses the problem, that no tributary flows can be added to the main stream at the point of confluence.

As already indicated, the value of K and X are dependent on the geometry of the river channel. Over a very long reach, significant changes in the geometry can occur. Some of the assumptions made in para 10.2 might only be applicable in the upper reaches of the river and not downstream, or vice versa. Better results could be obtained by changing the normal routing procedure in such a way that different K and/or X values can be used for different sub-reaches. There is physical justification for this approach and it also given an opportunity for finer adjustments to the model. It could also solve the problem as mentioned in (ii) above.

The results of flood routing calculations should never be accepted without checks against observed levels.

10.3.4 Effect of various K- X- and Δt-values

The effect of various K- X- and Δt-values on the shape of the outflow hydrograph can best be illustrated by a graphical representation. Figs 10.5, 10.6, 10.7 and 10.8 are self-explanatory and are a useful guide when calibrating the model. The same inflow hydrograph as above is used for illustration purposes.

From Fig 10.5 it is clear that various Δt values have a negligible effect on the shape of the outflow hydrograph, except when Δt becomes very coarse relative to the rate of change of the flow rate. Serious errors can be made on the estimation of the position and value of the peak flow rate as illustrated at the $\Delta t = 6$ hours-line.

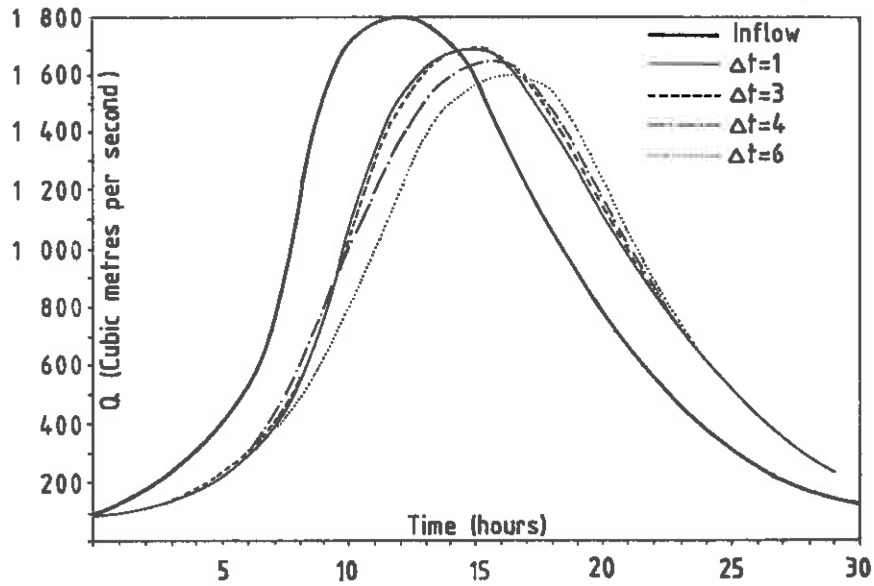

FIGURE 10.5 Various Δt-values for K = 2,5 and X = 0,4

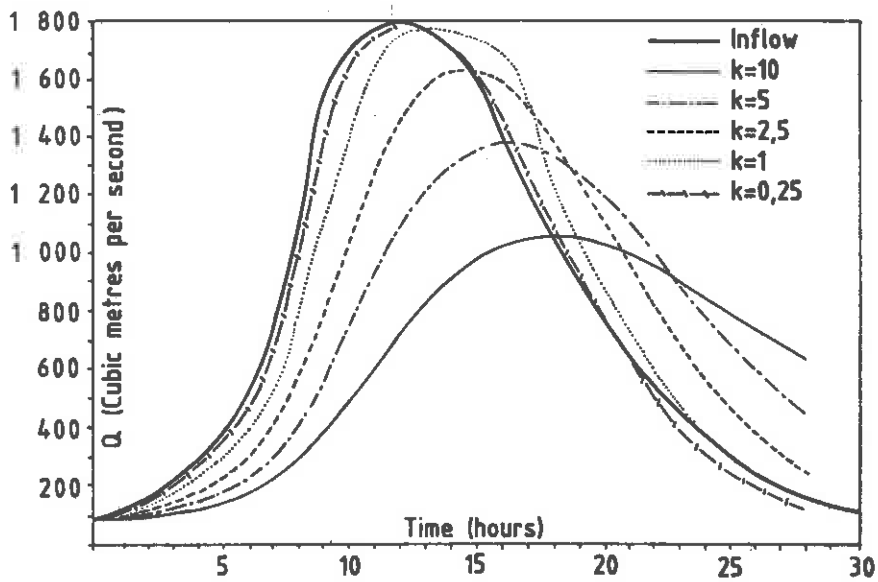

FIGURE 10.6 Various K-values for X = 0 and Δt = 2 hours for one reach

Decreasing K-values actually implies that more sub-reaches are considered. The total lag-time must therefore remain constant for the reach. In the above example, the K-values were changed without considering more than one sub-reach, which implies that the total lag-time for the reach has changed. In Fig 10.7 the effect is illustrated in changing the K-values by actually dividing the reach into sub-reaches and keeping the total lag-time for the reach constant on 10 hours.

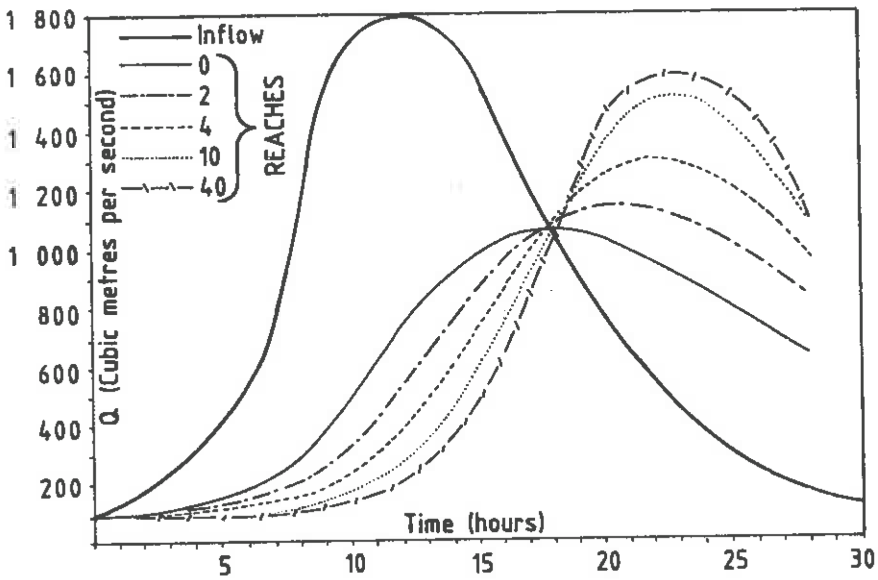

FIGURE 10.7 **Various K-values for $X = 0$ and $\Delta t = 2$ hours by dividing into sub-reaches. Overall $K = 10$ hours**

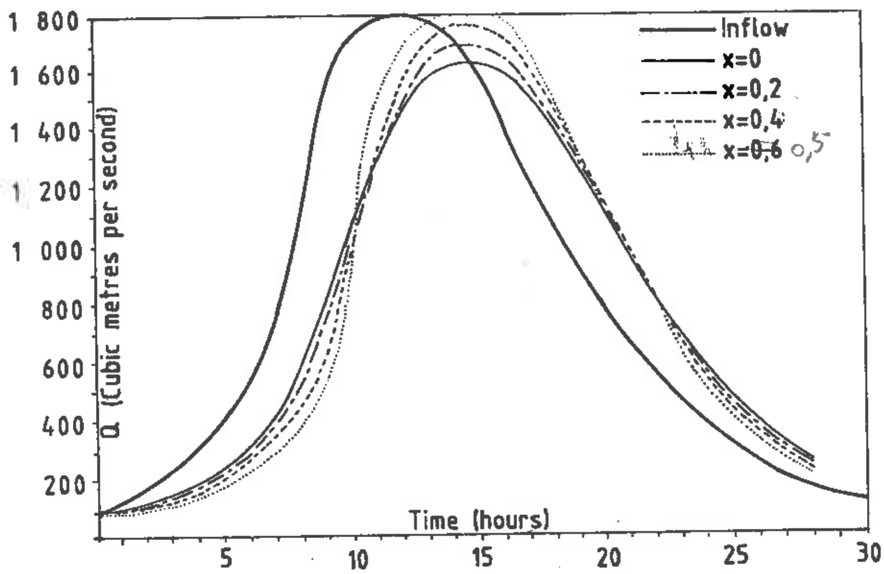

FIGURE 10.8 **Various X-values for $K = 2,5$ and $\Delta t = 2$ hours**

From Fig 10.7 it is clear that the outflow hydrograph is attenuated more for an increase in the value for K with a constant X-value. The opposite happens for an increase in X-values for a constant K-value, as shown in Fig 10.8. This phenomenon must be kept in mind when calibrating the mathematical model.

10.4 FLOOD CONTROL

10.4.1 Conflicting objectives

The risk of structural failure of a dam equipped with flood release gates and operated to achieve a measure of flood control is greater than for an equivalent conventional dam. There are a number of reasons for this including the need to deliberately increase the water level in the dam to utilize the flood absorption capacity of the dam basin, possible equipment failure, communication breakdown and human error. These factors should be taken into account during the design of the dam as well as when developing operating rules.

Even where dams are equipped with flood release gates specifically designed for flood control, the overriding constraint when operating the gates is to discharge incoming floods through the dam without endangering the safety of the structure.

The two objectives - minimizing the risk of failure of the structure and minimizing the risk of downstream damage by flood control procedures - are not compatible, in that minimizing the risk for one objective can be achieved only by increasing the risk of the other. The first objective requires the minimum possible rise in water level in the dam during the passage of the flood, whereas the second objective requires the full utilization of the flood absorption capability of the dam by deliberately allowing the water level to rise to the maximum allowable water level, or the maximum achievable water level where this is less than the maximum allowable water level.

Because of their possible effect on the safety of the dam, operating rules for dams with flood release gates should be developed as part of the design process for new dams and should be subject to inspection and evaluation in the case of existing dams. Factors which should be taken into account when developing operating rules for flood control measures are detailed below. Emphasis is on the less well known aspects.

10.4.2 The design objective

No South African dams with gated spillways have been built with the sole purpose of providing flood protection, and in most cases the reason for incorporating wholly or partially gated spillways has been one of economics or more rarely sediment evacuation (eg Floriskraal Dam). Only in the case of the two large Orange River dams was flood control a major consideration in spillway design. The recent raising of Vaal Dam is the only case where additional flood absorption storage has been provided. In all the other cases no storage capacity below full supply capacity is available solely for flood control purposes. At the time of writing the construction of a flood control dam upstream of Ladysmith in Natal is being considered.

Where dams have been equipped with flood release gates there is an understandable desire to make use of their flood peak attenuation potential even if they are not designed for this purpose. This is a major policy decision and should not be taken without a full appreciation of the additional requirements that have to be met, and the possible consequences of malfunction of the system.

10.4.3 Design flood

The first step in the development of operating rules for dams with flood release gates is the estimation of the magnitude of the flood that will have to be passed through the dam without a predetermined maximum water level being exceeded, which in turn relates to the stability of the dam.

The conventional philosophy of the design flood hydrograph is that of a hydrograph with a single maximum flood peak which will result in the maximum water level being reached at the dam wall. However, it does not follow that it is this flood that will result in the maximum water level being reached and consequently represent the greatest risk of failure of the structure. A sequence of closely spaced moderate floods is more likely to result in non-optimal operation of the flood release gates and consequently result in higher water levels at the dam wall than a single larger flood. The reason for this is that in the case of a dam with a fully gated spillway the objective when a single moderate flood is passed through the dam, is to utilize the flood absorption capability of the dam to attenuate the flood peak and thereby reduce downstream damage. If this is carried out efficiently, then at one stage during the passage of the flood the dam water level will be at the maximum allowable level. Should a second severe storm occur in the upstream vicinity of the dam at this time, there is a very real possibility that the maximum allowable water level will be exceeded.

Examples are the Vaal Dam catchment floods described in Annexure 3F to Chapter 3, and the February-March 1988 floods in the Orange River at the Hendrik Verwoerd Dam (Fig 10.9). In both cases the flood peak attenuation of the second peak was appreciably less than for the first peak.

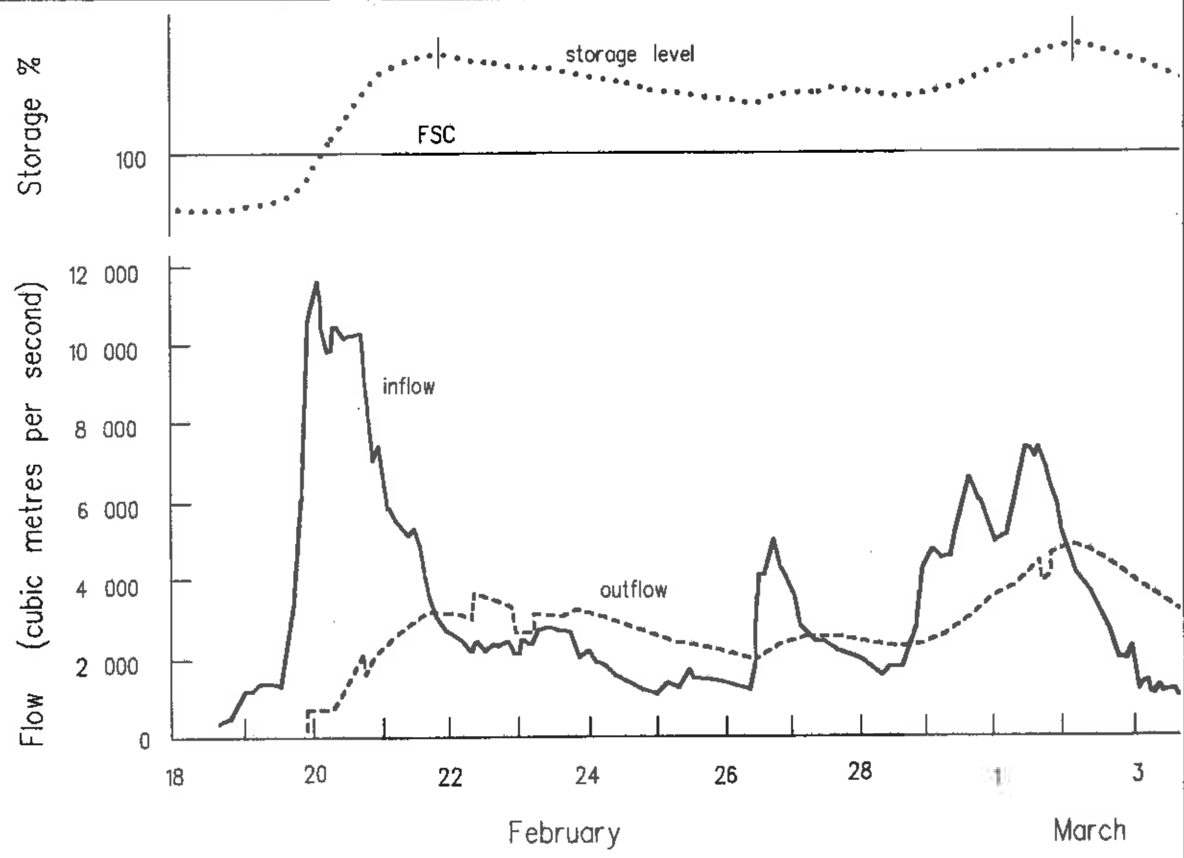

FIGURE 10.9 Flood routing through Hendrik Verwoerd Dam February-March, 1988

The following scenario has a much higher probability of resulting in unacceptably high water levels at the dam wall than the occurrence of a single flood event with peak equal to the design flood.

1. A sequence of several closely spaced, moderate floods.
2. The storms which produce the floods have areas of intense precipitation less than the area of the catchment, and precipitation intensities correspondingly higher than large area storms.
3. Successive storms positioned progressively closer to the dam wall, and climaxing in a small area, high intensity storm located over the dam basin itself.
4. Partial mechanical and/or electrical failure resulting in malfunction of the spillway gates.
5. Partial failure of rainfall and river flow monitoring, relaying, or processing instrumentation.
6. Incorrect meteorological forecasts. (Incorrect in the sense that heavy rain occurred while forecasts had predicted little or no rain during the forecast period).

7. Unforeseen circumstances in the downstream area necessitating delays in increasing flood releases.
8. Human error including incorrect interpretation of current conditions, inadequate provision for unforeseen conditions, conflicting responsibilities and instructions.

In the above scenario there is a high degree of correlation between the occurrence of the separate storms themselves as well as between the storms and the resultant equipment failure, communication breakdown, and human error. This combination has a much higher probability of occurrence than that of the design flood.

10.5 OPERATING RULES

10.5.1 Objectives

Operating rules have to be developed to achieve specific design objectives. Five different situations have to be considered :-

- Dams that were not designed for flood peak attenuation purposes, and have no flood release gates (most dams in South Africa are in this category).
- Dams that were designed to achieve a measure of flood peak attenuation but have no flood release gates (eg Beervlei Dam - the radial gates were designed for sediment evacuation).
- Dams with flood release gates but where flood peak attenuation is not an objective (eg Driel Barrage).
- Dams with partially gated spillways intended to achieve flood peak attenuation as one of the objectives (eg Hendrik Verwoerd and PK le Roux Dams).
- Dams with fully gated spillways intended to achieve flood peak attenuation as one of the objectives (eg Vaal and Bloemhof Dams).

No operating rules are required in the first two categories. All dams equipped with flood release gates require a set of operating rules, but the objectives of these rules will differ in the last three categories. Where the term 'gated spillways' is used below it includes all gates specifically designed for discharging flood water. These may be located at the crest of the dam (eg Vaal Dam), at river bed level (eg Floriskraal Dam) or at some intermediate level (eg the Orange River dams).

In all cases it is assumed that there is a fundamental requirement that dams have to be full after the passage of a flood. For the purpose of illustration it is assumed that the dams are full at the time the flood is generated in the catchment.

Where flood peak attenuation is not a consideration, the optimum operating rule is to open the gates to match the inflow and so keep the water level at the full supply level (or allow a slow increase in water level) until all gates are fully open and thereafter allow nature to take its course.

In this situation only the rate of change of water level in the dam needs to be monitored, and a reliable mechanically or electronically controlled gate operating system would be preferable to human intervention. The reason for this is that changes in water level can be monitored more accurately and more frequently, and the operating rules can be programmed on a computer. The program must include an algorithm to prevent the discharge from hunting. Provision has to be made for action to be taken in the case of equipment failure.

10.5.2 Optimum operation

Where flood peak attenuation is an objective the operating rules become more complex and the application of judgment and experience is a prerequisite for efficient operation of the system.

Optimum operation of a dam equipped with flood release gates requires full knowledge of the magnitude and shape of the incoming flood hydrograph; the gauge level-storage volume relationship; the discharge characteristics of the uncontrolled spillway and flood release gates; and possible constraints on releases from the dam prior to the arrival of the flood peak.

In the case of a fully gated spillway, as soon as details of the incoming flood are known, the gates are opened to maintain a constant discharge such that the water level in the dam will rise to a level equal to, but not exceeding the maximum allowable level. This discharge is then maintained until the water level drops to the full supply level when the discharge is reduced to equal the inflow. This is shown in Fig 10.9(a).

Fig 10.9(b) shows the optimum operation of a dam with a partially gated spillway. In this case the proportion of the flow passing over the uncontrolled spillway will determine the maximum water level in the dam that can be attained. Initially, the gates are opened to release the calculated minimum peak discharge and then gradually closed as the discharge over the spillway increases such that a constant discharge is maintained. At the time when the maximum water level is reached all gates will be closed and the outflow will be confined to that over the uncontrolled spillway. Thereafter the gates can remain closed and the water level in the dam allowed to subside naturally, or partially opened to bring the water level down more rapidly.

In both of the above cases, the earlier the availability of information on the inflow hydrograph the greater the flood peak attenuation that can be achieved. Also there will be no danger to the structure if complete and reliable information on the flood hydrograph is available prior to its arrival at the dam. In practice, however, full prior knowledge is not attainable and reasonable provision must be made for unforeseen circumstances. The extent of this provision will depend on a number of factors including the reliability of the early warning system.

In Figs 10.9(a) and 10.9(b) the assumption is made that there are no restrictions on the magnitude of the discharges prior to the arrival of the flood. In practice there are usually some restrictions on these prereleases such as the need to issue flood warnings, or unacceptable premature flooding of road bridges or occupied areas. In this situation the prerelease is limited to the maximum non-damaging discharge, and maintained at this discharge until the inflow equals this discharge. Thereafter, the discharge is increased to keep pace with the inflow until the optimum discharge is reached after which the discharge is held constant.

After the passage of the flood peak the water level should be reduced to the full supply level as soon as is possible without causing further damage in order to allow for the possibility of renewed floods developing in the catchment. However, when the full supply level is reached the gates should be closed gradually to minimise scouring of the river banks as the flood water covering the flood plain downstream of the dam returns to the river channel and so minimise sloughing of the river banks as bank storage is released.

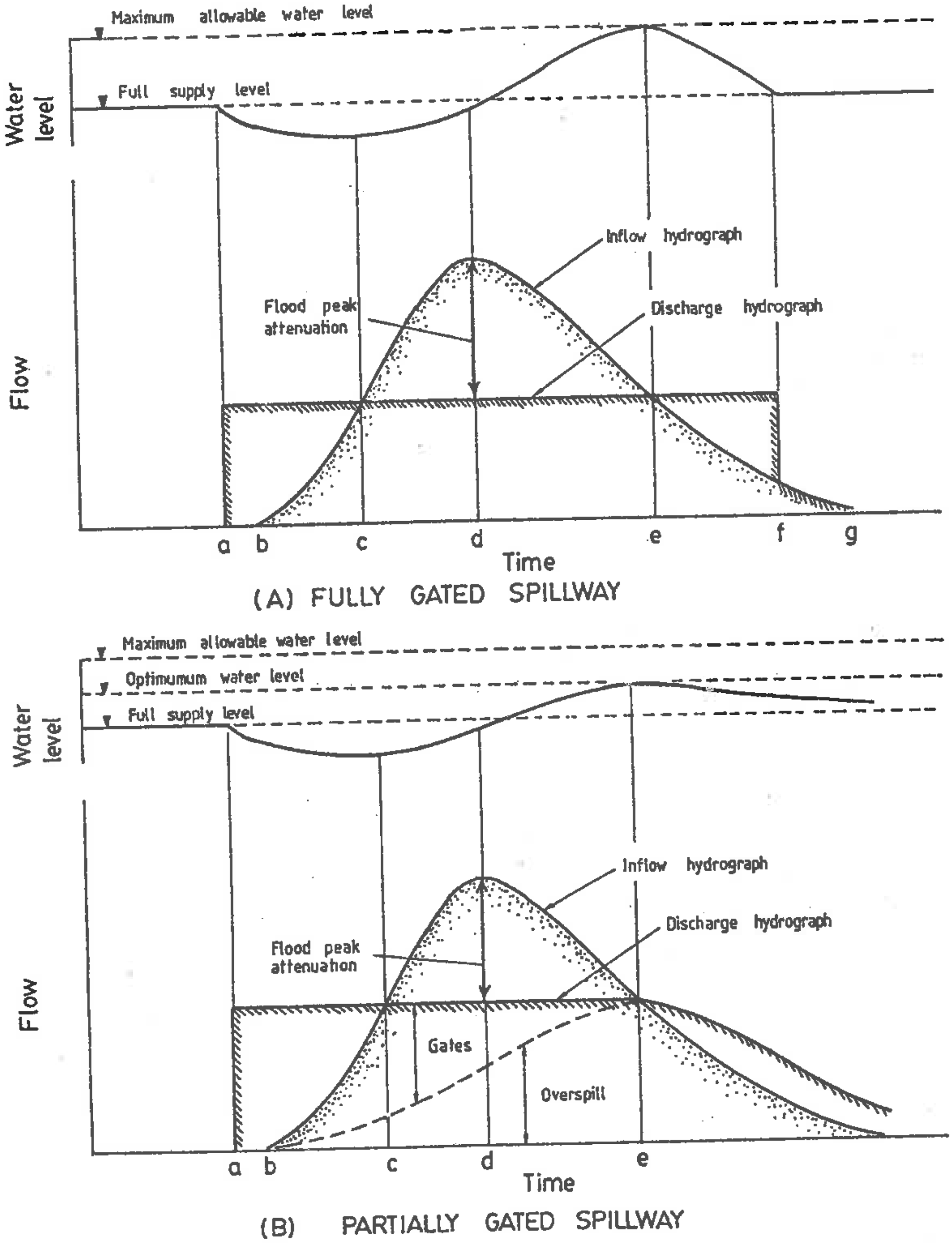

FIGURE 10.10 Optimum operation of dams with gated spillways in order to achieve maximum flood peak attenuation.

10.5.3 Early warning system

There are three levels of early warning: meteorological forecasts, rainfall measurements and flow measurements. Meteorological forecasts provide warnings but the information is qualitative rather than quantitative. The forecasts vary over a time range of a day or two prior to the event, to weather radar data which provides immediate, accurate information on the location and movement of the storm itself, but only a rough indication of rainfall intensity. Rainfall data varies from routine daily observations to recording rain gauges which can be interrogated by telemetry every ten to fifteen minutes. The usefulness of the data will depend on reliability of the assumed rainfall/runoff relationship. Flow measurements provide the most accurate information but become available only shortly before the arrival of the flood peak at the dam. The accuracy of the predicted magnitude and shape of the flood hydrograph at the dam will depend on the distance between the gauging station and the dam, the size of the intervening catchment and the accuracy of the flood routing model.

The weight placed on the different sources of information will depend on the situation at the dam at the time that they are received. For example at time *e* in Figs 10.9(a) and 10.9(b) a meteorological forecast that heavy rain can be expected in the immediate vicinity of the dam within the next few hours is of more value than an accurate flow measurement at a point well upstream of the dam. Given this situation the operator cannot ignore the weather forecast and must take appropriate action to reduce the risk of the water level in the dam rising to undesirable levels even if this results in higher discharges causing additional damage, which may subsequently prove to have been unnecessary if heavy rain does not occur. In this example the operator has to exercise his judgment and experience when weighing up the *certain* increase in downstream damage if the discharge is increased against the *remotely possible* failure of the structure and disastrous consequences if the discharge is not increased.

This is not a responsibility that should be required of an inexperienced operator. The operating rules should be explicit on this point - whenever the water level in the dam is within a specified range and there is any doubt about inflows in the immediate future, the discharge from the gates should immediately be increased such as to maintain the water level at that level and prevent it rising further, even if this would result in additional downstream damage.

10.6 LEGAL ISSUES

10.6.1 Legal requirements

The two obvious legal requirements are that flood damage downstream of the dam should not occur earlier than it would have occurred naturally nor should it exceed damage that would have occurred had the dam not been built. A less obvious legal constraint is that related to flood warnings. If for example, information is provided to the effect that no increases in flood release will take place, and this subsequently occurs, the operator could be held responsible for the resulting damage even if the damage is less than would have occurred had the dam not been built.

The line of authority and thereby responsibility for taking (or avoiding) decisions and making public announcements should be specified. The operator on site should not be burdened with having to make important decisions which are beyond his competence. Actions to be taken when there is a breakdown in communication should be listed, and responsibility for the consequences of the action becomes that of the authority issuing the instructions and not the individual who has to carry them out.

10.6.2 Warnings to downstream areas

There is an obvious need to inform persons in downstream areas of the likelihood of flooding but the arrangements must be such that this does not in itself become a constraint on action that is necessary to minimize damage. Once it is obvious that downstream damage will occur general warnings should be issued. These should specifically mention that changes in the release from the dam can be made at any time without further warning. There should not be any undertaking to inform persons individually of imminent increases in flood discharge as this may cause delays in taking action which would minimize flood damage, or in an extreme case, may jeopardize the safety of the structure with all the consequences that that would entail.

10.7 GENERAL COMMENT AND SUMMARY

There are several examples of design inadequacies, equipment failure, and non-optimal operation of dams with gated spillways in South Africa. There is little doubt that the combined risk of failure from all sources is greater in these dams than those with conventional, uncontrolled spillways.

Therefore a decision to construct a new dam with flood release gates or operate an existing dam with gated spillways for flood control purposes where this was not a design consideration, should not be taken without a full appreciation of all the consequences.

Where this decision is made, the designer and the future operator should jointly develop the operating rules. When doing so they must appreciate that full knowledge of the incoming flood or succession of floods is rarely if ever available and that a single extreme flood poses a smaller risk of failure of the structure than a succession of moderate floods.

Equipment failure, communication breakdown and operator incompetence are risks that are as real as that of damaging flood and must be accommodated in the operating rules.

Time taken to solve a hypothetical flood routing problem in the design office should not be equated with that of a real world situation where the lack of information; need to take time to assess the situation including possible effects of future unforeseen conditions; time required to issue warnings and take appropriate action; interruptions; and the stress of having to make an immediate irreversible decision which may have serious consequences are important factors that will affect the efficiency of dam operation.

Finally, all dams equipped with flood release gates should have a concise set of operating rules posted in a prominent position in the operator's office. An evaluation of the adequacy of the rules and availability of competent staff to carry them out should form part of the routine inspection procedures.

Chapter 11

CONTROL OF FLOOD PLAIN DEVELOPMENT

Contents

11.1 INTRODUCTION

The term *flood plain* refers to all land outside the river channel that may be inundated by floods however rarely this may occur.

In South Africa we do not have the extensive alluvial flood plains that are found in the USA or the Indian sub-continent, nor are our limited flood plains developed to the same extent as those for instance in Japan. Nevertheless, floods have caused extensive damage in the past, and this damage can be expected to increase in future due to increased encroachment on the flood plains in both urban and rural areas.

Most flood damage in urban areas is related to loss of life, destruction of buildings, and disruption of water supplies, sewage reticulation and communications. In rural areas the damage is principally the loss of production due to inundation, and the damage to land due to erosion and the deposition of sediment.

Other potential consequences in both urban and rural areas are the contamination of water sources, and the incidence of water related diseases such as cholera and malaria.

Alternative options for limiting the effects of flood damage are the construction of engineering works such as flood retention dams or flood levees, control of land use on flood plains by legislation, and financial compensation for flood damage by the State or State-aided insurance schemes.

Increasing demands for residential, industrial and recreational areas within our towns and cities require maximum beneficial utilization of all available land within their boundaries. This includes areas prone to flooding. However, encroachment of the flood zone not only involves a measure of risk to occupiers and owners of property within the flood zone, but may also result in the raising of water levels during floods to the extent that other development at higher levels may be endangered.

Some development within flood zones cannot be avoided. This includes roads, railways, and the provision of services. Road and rail embankments may raise upstream water levels and concentrate flood flows. Bridge openings are often restricted by debris during major floods causing upstream water levels to rise still further.

The magnitude of floods can be appreciably increased by urban development within the catchment area of a stream unless special measures are included in the layout and storm water drainage design of the upstream development.

The consequences of flooding are often more serious than mere inundation and damage to the contents of buildings. There are many recorded floods in South Africa where lives were lost when residents were trapped in their homes, and helpers died in rescue attempts. Buildings and other structures in the direct path of the enlarged river channel have been totally destroyed. Other buildings further from the main channel have collapsed due to scour, or in the case of older buildings, due to poor construction methods. Severe structural damage has been caused by floating debris.

There have been instances where development within the flood zone has obstructed the path of flood water, raised the water levels, and concentrated the flows along roadways to the extent that these have become obstacles to evacuation rather then escape routes.

These are all factors which should be taken into account when making decisions relating to future development within flood zones.

11.2 FLOOD DAMAGE REDUCTION MEASURES

11.2.1 Construction of flood protection works

The most effective flood peak reduction measure is the construction of flood retention dams. An example is the Pongolapoort Dam which absorbed the whole of the exceptional Domoina flood. This was due to the large capacity of the dam and the fact that it was nearly empty at the time. This is unlikely to be the case in future now that the dam is being utilized for water supply.

There are no dams in South Africa that have been built and operated solely for flood damage reduction as this is seldom economic even in countries with extensive flood plains. However, all dams have a flood peak attenuation potential, and some have been equipped with flood control gates for this purpose (see Chapter 10 on flood routing and flood control).

Another engineering measure is the construction of flood levees between the area to be protected and the river channel.

It is extremely important that the following factors be taken into account when considering the construction of any of these alternatives.

- It is not practically possible to build engineering works which can be guaranteed to provide full protection against all possible floods.
- Consequently, all engineering works have a probability of functional failure which can be quantified.
- Nevertheless, the general public will assume that all flood protection measures are 'safe'.
- Experience in the USA and elsewhere has shown that flood damage has increased over the years notwithstanding the construction of extensive flood protection works.

- *Therefore, while the construction of flood protection works will provide protection against frequent, minor floods they will not, and can not provide absolute protection against extreme floods. Consequently when these floods do occur they may cause loss of life and damage in excess of that which would have occurred had the works not been built and development on the flood plain restricted.*

11.2.2 Flood retention storage

There may well be situations where flood protection works have to be provided. The basic issues related to the provision of flood retention storage are discussed in detail in Chapter 10 on flood routing and flood control. Although flood peak reduction may reduce the area inundated as well as physical damage to the flood plains; agricultural lands, particularly those with permanent crops, are susceptible to the duration of inundation. As the optimum operating procedure is one that will minimise total damage, this does not necessarily imply maximum reduction of the flood peak at the expense of lengthened duration.

11.2.3 Flood levees

Flood levees should have adequate freeboard for the effects of wave action and flood debris. The fact that levees may restrict the flood waterway and so increase flood levels should be taken into account in their design. Regular inspections and maintenance of the embankments and drainage systems are essential.

11.2.4 Publicity

The public downstream of the dam should be reminded from time to time that full protection is not possible and that they should take the necessary precautions to protect their interests such as insurance cover, physical protection works and a plan of action should an extreme flood occur.

11.3 STATUTORY REQUIREMENTS

11.3.1 Water Act, No 54 of 1956

Legislation controlling development on flood plains is the most effective measure for limiting loss of life and damage due to floods in urban areas. Clause 169A of the Water Act was submitted to Parliament for the first time in 1975.

In an explanatory memorandum the Department pointed to the March 1972 floods in Marienthal downstream of Hardap Dam. The flood peak into the dam was 6 100 m^3/s and resulted from 178 mm of rain falling in the catchment over a period of 16 hours. The controlled peak discharge from the dam was 3 700 m^3/s which was significantly higher than previous maxima prior to the construction of the dam (3 180 m^3/s in 1908, 2 830 m^3/s in 1922 and 1 530 m^3/s in 1934). The flood caused considerable damage in Marienthal, much of which could have been avoided had the Municipality taken heed of repeated warnings that new extensions would be vulnerable to flood damage.

Other examples quoted were the February 1975 floods in the Vaal River which caused damage in Standerton, Vereeniging and Parys.

Requirements of Section 169 of the the Water Act

The following are the requirements of the Water Act as amended by the Water Amendment Act of 1978.

169A. Insertion of certain plans, and approval by Minister in respect of establishment or extension of townships in certain areas.
(1) No township shall, after the commencement of the Water Amendment Act, 1978 be established or extended under any law on any land unless -

(a) the following particulars have been inserted on the relevant lay-out plan in a manner to the satisfaction of the authority empowered under the relevant law to approve of the establishment or extension in question -

(i) in respect of any water course with a known and defined channel and with a catchment area exceeding one square kilometre, the lines indicating the maximum level likely to be reached on an average every twenty years by flood-waters on the land in question, and

(ii) in respect of any low lying land without surface drainage on which water from an area exceeding five square kilometres collects naturally, the lines indicating the maximum level likely to be reached on an average every fifty years by such water on the land in question; and

(b) if the land in question is situated in an area which, in the opinion of the Minister, is likely to be inundated by flood-water and which he has defined by notice in the Gazette, the Minister has, subject to subsection (2), previously approved of such establishment or extension, and the establishment or extension in question if effected in accordance with the conditions which the Minister may have deemed fit to impose on giving the said approval.

[Sub-s. (1) amended by s. 7 of Act No 27 of 1976 and substituted by s. 18 of Act No 73 of 1978].

(2) If the whole or any portion of an area which the Minister has defined under subsection (1) (b), is situated within any guide plan area as defined in section 1 of the Environment Planning Act, 1967, the Minister shall only grant the approval referred to in subsection (1) (b), in consultation with the Minister of Planning and Environment.

[S. 169A inserted by s. 25 of Act No 42 of 1975].

No areas in South Africa have yet been proclaimed under subsection 1(b) of section 169A of the Act.

The Act does not specify whether or not future development in the catchment must be taken into account when determining the flood-frequency relationship, or whether or not future development within the flood zone, including the proposed township itself must be taken into account when determining the flood water level.

The Water Act merely requires that the 20-year flood line be shown. Authorities responsible for the approval of the township layout may impose any restrictions that they wish on development above or below the 20-year flood line.

11.3.2 The nature of flood risks

The intention of Clause 169A is twofold :-

(a) To *alert* the relevant authority to the danger of flooding.
(b) To *empower* the Minister to control development in areas which he may specify.

Parliament considered that the insertion of the 20-year flood line on the layout plan would be adequate to achieve the first objective. To date the Minister has not exercised his powers in terms of the second objective.

Clearly, the relevant authority is expected to take some action once having been alerted to the fact that a danger of flooding exists. This action will in turn depend on the nature of the risks at the particular site.

Risk of loss of life
This is the most important consideration. Conditions which give rise to this risk are those where there is little prior warning of the flood, or where rising floodwater levels block escape routes. Both of these factors were present in the Laingsburg floods of January 1981 where more than 100 lives were lost (Roberts and Alexander, 1982). This risk will also be present in much smaller urban catchments where short duration intense precipitation storms (say 2 to 6 hours duration) could occur during the night and trap residents in their homes.

Structural damage
This is primarily a function of the water velocity. Other factors are the effect of floating debris, and the possible deposition of sediment.

Damage to the contents of buildings
The period of warning will determine the extent to which portable items can be saved. The action of removing these items may place the lives of owners at risk, however. The duration of inundation may be important from an agricultural point of view.

11.3.3 Specification of flood risk

While legislation required that the 20-year flood line be shown on layout plans, the local authority could specify more stringent requirements, or a developer may require an assessment of flood lines that may be reached by extreme floods.

Three alternative approaches could be considered :-

Extreme floods
Geomorphological mapping of the area including sediment deposits and vegetation types which would indicate the effect of previous flooding could provide information on the extent of past extreme floods. This could be a valuable adjunct to flood profile analyses based on the PMF or RMF.

Largest recorded flood
The flood profile of the largest documented flood or a specified recorded flood is an alternative that has been used in overseas countries. It has the advantage that the profile can be determined accurately and cheaply and there will be none of the inconsistencies which occur when flood profiles are determined by different persons using different assumptions and methods.

Return period floods
The advantages of determining separate flood profiles for flood magnitudes associated with a range of return periods are that local authorities can specify limits to development within the flood plain that are appropriate to the flood risk.

The 20-year, 50-year and 100-year floods are appropriate for most applications. From Table 9.1 in Chapter 9 it will be seen that there is a 64%, 33% and 18% probability respectively that these floods will be equalled or exceeded at least once in a 20-year period.

The state of the art in hydrological, and to a lesser extent hydraulic analyses is such that different combinations of available methods for flood frequency analyses, flood routing analyses and flood profile determinations will produce different flood profiles.

In view of these discrepancies, a case can be made out for local authorities to specify standard calculation methods.

11.3.4 Statutory requirements of local authorities

The whole purpose of Section 169A of the Water Act would be negated if the actions of a relevant authority did not go beyond the requirement that the 20-year flood line be shown on a layout drawing.

Authorities should consider exercising the following functions :-

(a) Specify the appropriate analytical methods to be used, and the qualifications of persons competent to carry out the necessary hydrological and hydraulic analyses.

(b) Require that the person carrying out the analyses report on the risks of danger to life and property within the flood-prone area.

(c) Require that the developer provide a township layout that would minimise the risk to life and property.

(d) Control activities within the flood-prone area which may increase the risk of flooding adjacent properties.

(e) Control the development within a catchment that would otherwise increase flood risks in downstream areas.

Some specific control measures that could be considered are :-

(i) Define the "floodway" as the strip of land between the river bank and (say) the 20-year flood profile. With the exception of natural or artificially established vegetation including trees along the river bank which play a valuable role in stabilising the river banks and limiting sediment deposition on the adjacent flood plain, all other trees, hedges and fences must be parallel to the flow of water.

(ii) No other development that will inhibit the free flow of water along the floodway should be permitted. This prohibition includes fill material on flood plains, road and rail embankments, terraces, and power or telephone pylons.

(iii) Buildings within the zone between the 20-year and 100-year flood profiles may be erected subject to approval of their size, floor elevations, building materials and type of construction. The developer should submit an assessment of the effect of the obstruction caused by the structure on raising the 100-year flood profile.

(iv) No old age homes, hostels, hotels or caravan parks should be allowed below the 100-year flood line.

(v) Housing properties may be permitted above the 20-year flood line provided that the ground floor of houses as well as exit routes from the houses are above the 100-year flood line.

The National Building Regulations also contain provisions relating to buildings vulnerable to flood damage.

11.3.5 Flood preparedness

Local authorities with flood prone areas within their boundaries should have readily accessible maps of these areas which indicate possible flood levels including levels associated with the possible breaching of upstream dams. A clear line of responsibility for decision making and action should be established.

Residents and owners of property within flood prone areas should be reminded of the risks and informed of the action that they should take in the event of serious floods occurring. They should not assume that each owner will be informed of imminent flooding or worsening of the previously announced situation.

11.3.6 Overseas practice

[Note : Reference to overseas practice in this section is principally from United Nations (1969), Johnson (1970) and Liebman (1970).]

The approach in the USA is altogether different from that followed in the South African legislation.

In the USA the National Flood Insurance Programme is administered by the Federal Insurance Administration. In return for making subsidised flood insurance available, the participating community agrees to regulate new development in the flood plains.

The first step is to determine the elevation of the 100-year flood which is termed the *base flood elevation*. An area within the base flood surface is determined by 'squeezing in' the base flood boundary until the base flood elevation is raised by 0,3 m. This simulates the effect of completely obstructing the base flood zone progressively towards the centre of the river. When the imaginary obstruction has blocked the flood flow enough to raise the base flood elevation a maximum of 0,3 m, the limits of the obstruction define the boundary between the *floodway* and the *fringe areas*.

When performing the calculation, an equal degree of encroachment is achieved by ensuring that there is an equal loss of flood water conveyance on each side of the river. This means that the areas of encroachment will not necessarily be the same on the two sides.

No obstructions of any nature are permitted in the floodway if they may cause a rise in the water level. All new development within the fringe areas must be such that each subdivision contains sufficient area of land above the 100-year base flood elevation to reasonably allow a residence to be constructed, where the lowest part of the lowest floor of the residence is at least 0,5 m above the 100-year base flood elevation. In addition, the structure must be at least 5 m horizontally away from the base flood surface and there must be adequate emergency access to the structure during periods of maximum flooding.

Non-residential structures may be constructed within the fringe area (but not within the floodway) provided they are designed to preclude or withstand an inundation of at least the 100-year flood.

Compensatory excavation will normally be required for fills within the fringe zone.

The USA policy detailed above has the advantage that the calculations are straight forward, the limitations on development unambiguous, and Federally subsidised insurance guaranteed.

In South Africa no State-aided insurance is available, and the consequences of flooding should be fully understood, particularly in the case of proposed development at a lower elevation than that of the 100-year flood.

11.4 FLOOD PEAK AND FLOOD PROFILE CALCULATIONS

The first assessment that must be made is the magnitude of floods and their associated annual exceedance probability.

The next step is to calculate the corresponding water level in the area of interest. Here the assumption is made that it is the water level reached by a constant flow equal to that of the flood peak.

The flood peak and flood profile calculations are relatively straight forward, and the calculations for a series of different frequencies of occurrence do not require much additional effort. The following range of return periods (years) will usually be adequate :-

20, 50, 100, 200, Regional Maximum Flood.

It is most important that the present and possible future development within the catchment be taken into account when determining the flood peaks, and that existing and possible future structures within the flood zone or which may influence water levels within this zone be taken into account when determining the flood profiles.

11.5 RISK OF INUNDATION

Provided the influence of present and possible future development within the flood zone is taken into account, the flood profile associated with the regional maximum flood (RMF) which is a measure of the highest floods experienced in the region will indicate the level above which flooding is unlikely, and for all practical purposes development above this level can be considered as being beyond the influence of future floods. The depths of inundation for the RMF are good indicators of the danger potential in different areas.

Obviously, the risk of flooding will be greatest on the river banks and will decrease to near zero at the RMF level. The flood levels associated with 20, 50, 100 and 200-year floods will be located between the river bank and the RMF level. The water level will be equal to or higher than the 20-year level once in 20-years on average, ie there is a 5% risk of this occurring in any one year. Similarly, there is a 1% risk of the water level reaching or exceeding the 100-year level in any one year, etc.

However, from the owner's point of view it is more meaningful to consider what the risk of flooding will be during the period of occupation, or mortgage period of the residence, or design life of the structure. The table below shows the risks for various periods.

TABLE 11.1 Risk of flooding during the period of occupation of a building

Return period of the flood	Risk of being equalled or exceeded during the next :		
(years)	10 years	20 years	50 years
20	40%	64%	92%
50	18%	33%	64%
100	10%	18%	40%
200	5%	10%	22%

The table shows that during the next 20 years there is a 33% risk that the 50-year flood will be equalled or exceeded at least once an 18% risk that the 100-year flood will be equalled or exceeded, and a 10% risk that the 200-year flood will be equalled or exceeded.

These risks are not insignificant.

11.6 CONSEQUENCES OF FLOODING

The above table plus the flood peak and flood profile calculations indicate the *risk of inundation*, but inundation is not the only risk that has to be faced.

11.6.1 Loss of life

This is the most serious consequence of flooding, and is directly related to the rate of increase in water level rather than the ultimate depth of inundation. Coupled with this is the length of prior warning and access to escape routes.

For example, residents within the flood zone of the Orange River near Upington usually have more than 24-hours advance warning of floods, and can easily make their way to higher ground.

Residents in an urban area alongside a stream downstream of a catchment which is fully developed can expect high flash floods characterised by short response times (tens of minutes). Floods could arrive during the night without warning. In these small catchments high ground is usually near at hand, but the streams are steep and high velocities are common even near the edge of the flooded area, making escape hazardous.

Even moderately sized catchments (up to 4 000 km^2) in the drier and therefore more sparsely vegetated parts of South Africa, or even in steep coastal areas can develop flash floods which rise to a peak in less than 10 hours. Many of these rivers have wide, almost level flood plains (in some cases these flood plains slope downwards away from the river bank). Water levels rising two or three metres in as many hours have caused many deaths within developed areas located on flood plains in South Africa. Possibly the main reason has been that by the time that occupants have appreciated the seriousness of the situation (water entering the house), the depths and velocities of water along escape routes are already too high to be negotiated in safety.

A moderately sized person begins to lose stability in a metre of water flowing at 0,6 metres per second. Children, the aged, and the infirm would have difficulty in negotiating lower water depths and velocities. As a general rule, water depths greater than one metre cannot be negotiated by non-swimmers or conventional vehicles with safety.

Assuming that families evacuate their houses when the water level reaches the lowest floor level in the house, then the water depth/velocity relationship along their evacuation route should not exceed the values shown in Table 11.2. Note that both momentum $f(v^2y)$ and buoyancy determine these limits.

TABLE 11.2 Maximum permissible water velocities and depths along escape routes

Velocity m/s	Depth m
0,5	1,0 (maximum)
1,0	0,7
1,5	0,5
2,0 (maximum)	0,4

11.6.2 Structural damage

Most newly constructed buildings will be able to withstand the effects of inundation alone. However, the risk of structural damage increases rapidly with increase in the velocity of the water. Damage can be from scour, battering by floating debris, or failure due to pressure differences on either side of doors, windows, walls, or the building itself.

11.6.3 Damage to the contents of buildings

Potential damage to the contents of buildings will depend on the likely advance warning time and the portability of the items. In areas susceptible to flash floods it is unlikely that more than the few items that can be transported in the family car will be saved from inundation.

11.6.4 Damage due to the deposition of sediment

Most floods carry a lot of sediment. In the more arid areas heavy deposition of sediment can be expected in areas where rapid decreases in velocity occur. This is usually in the immediate vicinity of buildings as well as inside the buildings themselves.

11.6.5 Insurance

Another important risk is the possibility that properties subject to inundation may have difficulty in obtaining insurance cover, particularly after the first occasion when damage is caused. Properties subject to damage from the 20-year flood have a 64% risk of being damaged at least once in a 20-year period. This is significantly greater than the normal risks covered by standard insurance cover.

11.7 INFLUENCE OF STRUCTURES WITHIN THE FLOOD ZONE

11.7.1 Increase in water level

Structures, embankments and fences within the waterway will obstruct the flow and raise the water level thereby increasing the risk of damage to other buildings at higher elevations.

11.7.2 Concentration of flow

Obstructions also have the effect of concentrating flows and increasing velocities along the clear passages between them. A particularly unfavourable situation exists where the streets are in a grid pattern with major roads parallel to the course of the river. Flow is concentrated along these roads which then become barriers to evacuation instead of escape routes. Wherever possible, roads within the flood zone should be angled towards the direction of flow in such a way that they provide calm-water escape routes to higher ground.

11.7.3 Increase in sedimentation

High velocity overbank flows within built-up areas may have high sediment loads which are deposited in the slack-water areas between the houses adjacent to the streets.

11.8 FLOOD DAMAGE RELIEF

11.8.1 Direct State-aided relief

Severe regional economic problems may arise from widespread flood damage which may give rise to indirect losses to whole sections of the community. In this situation the State may declare the affected area a flood disaster area in which case funds can be made available for partial compensation to those directly affected. This avenue of assistance is not available to individuals who may suffer equal damage from isolated events.

11.8.2 State-aided insurance

A more equitable form of State assistance that is provided in the USA is the provision of State-aided flood insurance. The participating insurance companies are responsible for settling claims up to a predetermined threshold with the State accepting responsibility for the balance. Sophisticated risk-related insurance schemes are available, where the premium is based on the risk which in turn requires the establishment of flood profiles in the affected areas. This has the advantage that the insurance premiums are a disincentive to farming in a high risk area but do not necessarily make farming uneconomical. This type of insurance is not available in South Africa, but may well be an alternative to costly protection works.

11.8.3 Private insurance

Conventional property and crop damage insurance cover is available. It is doubtful whether the total values are such that insurance companies in South Africa would institute separate flood risk related insurance policies, although if owners in high risk areas had to pay higher premiums this would make them more aware of the risks and more likely to consider establishing their concerns in safer locations.

11.9 ECONOMIC ASPECTS

11.9.1 Economic analysis

Conventional economic analysis requires an assessment of whether the loss prevented is sufficiently large to justify protection costs. This requires the determination of risk - stage - damage relationships with hydrological flood frequency analyses as the starting point and a frequency distribution curve of damage as the end product. Potential flood damage should include the estimated costs of restoration to pre-flood conditions and capitalized loss of income.

11.9.2 Other considerations

Other important considerations are loss of life, disruption of communications and the possibility of irretrievable flood damage to the river channel and flood plain.

11.10 GUIDELINES

No manual or set of rules can be expected to cover all situations related to development within flood zones in South Africa. In most cases potential developers and the responsible authorities will have to rely on the judgement of their professional advisers based on a well-founded appreciation of the risks to the present and future owners and occupiers.

11.11 FLOOD PLAIN MORPHOLOGY

11.11.1 General

Flood plains are prime areas for agricultural development due to their fertile, near-horizontal alluvial deposits. Residential properties with river frontages have higher values than those further from the river. Open, flat areas on flood plains have a number of advantages for industrial development.

However, property owners on flood plains seldom appreciate that it is the river itself that formed the flood plains, and where the river has flowed in the past it will surely flow again, and that where sediment was deposited it will deposit again! Furthermore, the more extensive the flood plain the greater the likelihood that the position of the river channel itself will change with time, and possibly dramatically during exceptional floods.

As has already been seen the construction of flood control works such as flood retention dams and levees will reduce the frequency of damage from smaller floods but will not prevent the occurrence of extreme floods and the resulting rare, but extensive damage that they may cause. Because the river banks and the adjacent flood plain generally consist of non-cohesive, readily erodible material, they are particularly vulnerable to flood damage which may be aggravated by flood plain development or ill-conceived attempts at river channel canalisation.

Because of the time-variant nature of the river channel and flood plain formative processes it will never be possible to make accurate, quantitative predictions of how a river will respond to man-made intervention along its course any more than it will be possible to predict whether or not it will rain on a specific day one year from now. Nevertheless qualitative predictions based on the most probable effects can and should be made.

11.11.2 Driving, response and control variables

The dominant driving variables in a river system are :-

(i) River flow and its variability.

(ii) Sediment availability including the grain sizes of the sediment delivered to the river channel and composing the river bed.

(iii) The vertical difference in elevation between the water surface levels at the entrance and exit of the reach being considered. The product of the river flow and this difference in elevation determines the total amount of energy and expended within the river system.

The major response variables of interest are the quantities and rates of water and sediment flow which leave the system.

The following control variables may also be considered as response variables if it is their response that is of interest.

(a) *Geometrical properties* : These include :-

(i) the cross-sectional shape of the channel (area, width, depth, shape);

(ii) the geometrical shape of the river course (straight, irregular, sinusoidal);

(iii) the slope of the river channel;

(iv) the straight-line slope of the valley in which the river flows.

(b) *Hydraulic properties* : The main physical properties of the water itself are its density and its viscosity. The velocity of the water is the dominant variable in all river mechanics processes and is primarily dependent on the energy gradient. Due to the viscosity of water, flow in nature is practically always turbulent.

(c) *River bank competence* : The competence of the river bank exercises a major control in both channel geometry and the flow processes.

(d) *Bed and channel forms* : These interrelate with the energy dissipation rates and vary in size and shape from ripples and dunes (bed forms) through to alternate bars and point bars (channel forms).

11.11.3 River channel geometry

An interrelationship exists between the valley slope, the meandering pattern of the river channel and the average particle size of sediment being transported through the system.

Bank competence also plays an important role in determining channel geometry. Cohesive bank material or banks protected by vegetation will maintain a narrower and deeper channel than banks consisting of non-cohesive material unprotected by vegetation. In the latter case the equilibrium condition is a wider and shallower channel.

11.11.4 The role of riverine vegetation

The cross-sectional dimensions and the location of a river channel on an alluvial flood plain are controlled to a large extent by the stability of the river banks. The role that riverine vegetation plays is not only the physical protection of the erodible banks but also the reduction in the velocity of overbank flow as it leaves the main channel with consequent deposition of sediment on the river bank rather than on the adjacent flood plain.

Riverine vegetation on the natural levees formed by the deposition of sediment assists in maintaining a narrow and relatively deep channel which is hydraulically efficient and has a high potential sediment transporting capability.

The equilibrium condition of a river flowing through an alluvial flood plain once the protective vegetation has been removed is very different. The unprotected banks will erode causing the river channel to become wider and shallower. Dramatic changes will occur in the river channel and and adjacent flood plain as the system changes to a new equilibrium condition. Further bank erosion may take place and the eroded material will be deposited further downstream - principally on the now unprotected flood plain.

Bank erosion, resulting from the removal of riverine vegetation, is the most destructive force at work within an alluvial flood plain. The harm it does is often irreparable in terms of the loss of agriculturally productive land. There is ample evidence of this in many South African rivers.

11.12 FLOOD PLAIN PROTECTION

11.12.1 Identification of risks

Aerial photography and field surveys of sediment deposits are appropriate methods for identifying areas prone to flooding and sediment deposition. A flattening of the longitudinal river profile and increase in channel sinuousity are indicators of reaches of the river where sediment deposition on the adjacent flood plain can be expected.

River confluences and reaches of a river upstream of narrow geological controls such as gaps through poorts are areas of possible unstable channel geometry and location.

11.12.2 Guidelines for flood plain protection

The golden rule in the protection of a flood plain is to guide and contain the river channel in its most stable position and to reduce the risk of excessive deposition of sediment on the flood plain. The most efficient as well as the most aesthetically pleasing method to achieve both of these objectives is to encourage the growth of vegetation, including trees, along and immediately adjacent to the river banks.

The waterway between the riverine vegetation and (say) the 20-year flood line should be kept clear of obstructions. Open grass-covered parks with occasional trees would be ideal.

Where development on the flood plain outside the floodway is permitted it should be such that it will not place the lives of occupants at risk or adversely affect the interests of others - for example by obstructing the flow to the extent that it raises the water level, or causes local scouring or the deposition of sediment.

11.12.3 Road and rail embankments

Road and rail crossings of flood plains should be planned with care. Conventional design consists of the bridge structure across the river channel, with approach embankments across the flood plain. The elevation of the bridge deck is usually the minimum height above the river bed that will allow the passage of the design flood without interfering with the traffic across the bridge. Economic considerations are usually limited to the most economic solution for the financing authority.

Additional factors that should be considered in cases where there are other users on the upstream flood plain are :-

1. Approach embankments obstruct high flood flows, reduce flow velocities and thereby aggravate upstream sediment deposition.

2. Rivers tend to follow a straighter course during floods and may short-circuit bends in the river. If the bridge crosses the river at one of these bends, not only will the river attack and possibly breach the approach embankment, but the obstruction caused by the embankment may upset the upstream flow conditions with unpredictable effects on channel scouring and sediment deposition.

3. Dune and bar formation in the river channel during floods may create trains of high standing waves with surface water levels appreciably higher than the average water level associated with a flood of that magnitude.

4. The design clearance between the maximum water level and the bridge beams should allow for wave action as well as possible obstructions caused by trees and other floating debris.

11.12.4 Canalisation of river channels

South Africa does not have any navigable rivers, and the need for canalisation is limited to flood plain protection and development. Canalisation may be considered as an option for stabilising a river channel in its present position, or diverting the channel to a new location in order to permit development of the adjacent area.

It is a matter of observation that straight, regular channels do not occur in nature. River canalisation is expensive and unsightly. Maintenance costs may be high. The following additional factors should be taken into account when considering the viability of canalisation.

1. Will the channel cross-sectional geometry and slope be such that the deposition of sediment within the channel at low flows will not reduce its capacity at higher flows?

2. What will the consequences be when flows exceed the design flow?

3. Particular care must be taken at the exit of the canalised section. Hydraulic calculations of the transition based on the assumption of a non-erodible wetted perimeter are not valid for erodible material. The transition should be such that the exit geometry, flow conditions and energy level match those of the natural channel at that point as closely as possible.

11.13 UPDATE

The views expressed in sections 11.11 and 11.12 of this chapter are based on more detailed presentations by the author formulated a decade ago but are still valid (Alexander 1976, 1978, 1979a and 1979b). For a later review refer to Stevens and Nordin (1987), Carson and Griffiths (1987) and the references therein. The basic differences in approach between that used in sections 11.11 and 11.12 and that used by most authors on the subject is that their main concern is usually the long term equilibrium condition associated with rivers in alluvium whereas the view of this author is that rivers flowing in alluvium are never in an equilibrium condition, and that human intervention can cause serious, irreversible changes.

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Chapter 12

FLOODS AND DAM SAFETY

Contents

12.1 INTRODUCTION

12.1.1 Caution

All references to the probable maximum flood (PMF) and regional maximum flood (RMF) in the quotations from the SANCOLD guidelines given in this chapter refer specifically to the recommended methods three (PMF) and six (RMF) listed in Chapter 9 on the design flood. Detailed methods of analysis are given in Chapter 13 and examples in the case studies in Chapter 14. The methods are also included in the accompanying computer programs. Comment on other methods for determining the PMF are given in section 12.7.2 below, but these alternative methods are not included in the SANCOLD guidelines, nor are they recommended in this handbook.

12.1.2 SANCOLD guidelines for the safety evaluation of dams

Regulations under Section 9C of the Water Act relating to the safety evaluation of dams were published in July, 1986. These contained conditions and requirements relating to the classification, design, construction, operation and maintenance of dams, as well as the qualifications and experience required for persons designing dams (Croucamp, 1986).

In September, 1986 the South African National Committee on Large Dams (SANCOLD) published interim guidelines on safety evaluation of dams in relation to floods. A revised edition will be published during 1990. The most significant changes that will be in the revised edition are incorporated in this chapter.

The purpose of the regulations is to ensure that dams are designed, built, operated and maintained in such a way that they are not a threat to public safety. The purpose of the guidelines is to provide guidance on acceptable methods for determining the design flood.

A distinction must be made between legal requirements, codes of practice, and guidelines. The first two are obligatory, whereas guidelines allow the user a measure of freedom of choice. In general, codes of practice are appropriate where the consequences of the action can be predicted with a high degree of assurance, for example codes of practice for concrete or steel construction, whereas guidelines are more appropriate where natural processes are involved which have a high degree of unpredictability, for example the strength characteristics of soils and rocks, and the statistical characteristics of floods.

A case can be made out for codes of practice for some applications of flood estimation methods, for example in flood line determination within the jurisdiction of a single authority where the need for consistent estimates of the flood level along the length of the stream are the prime consideration. Another example is in multiple, moderate risk investigations such as road culvert and urban drainage design where the time spent on investigating a wide range of alternative methods is not cost effective. The use of the log

Pearson Type 3 distribution with regional parameter estimates was made obligatory for Federal agencies in the USA but this decision has been frequently criticized since then (see Chapter 6 on regional statistical analysis methods).

The SANCOLD (1986) guidelines are a compromise between the need for standardization, and the freedom to apply methods appropriate to the problem at hand. Most guidelines on this subject, including the SANCOLD guidelines, make it clear that they do not prescribe the use of specific hydrologic techniques, and that sound engineering judgment must be exercised in deciding upon design flood and safety evaluation flood values.

Extracts from the SANCOLD guidelines are printed in italics below. These are quoted to establish the link between the guidelines and this handbook, and not as a substitute for the guidelines which should be studied in their own right.

12.2 CLASSIFICATION OF DAMS

The regulations classify dams according to their maximum wall heights as well as their hazard ratings which are then combined to produce a safety risk category. These are shown in Tables 12.1, 12.2 and 12.3 respectively below :-

Table 12.1 : Size Classification

Size class	*Maximum wall height (m)*
Small	*More than 5 and less than 12*
Medium	*Equal to or more than 12 but less than 30*
Large	*Equal to or more than 30*

Table 12.2 : Hazard Classification

Hazard rating	*Potential loss of life*	*Potential economic loss**
Low	*None*	*Minimal*
Significant	*Not more than 10 lives*	*Significant*
High	*More than 10 lives*	*Great*

**For classification purposes potential economic losses less than R1 million and more than R10 million (1986) would be considered "minimal" and "great" respectively.*

Table 12.3 Categorization of dams having a safety risk			
Classification	*Hazard rating*		
Size Class	*Low*	*Significant*	*High*
Small	*I*	*II*	*II*
Medium	*II*	*II*	*III*
Large	*III*	*III*	*III*

12.3 REQUIREMENTS IN RESPECT OF SPILLWAYS

The regulations require that information in respect of spillways be submitted as part of the application for a permit to construct or to impound. The required information varies from essentially geometric description in the case of a category I dam to disclosure of the calculation methods and criteria adopted as well as the results of calculations in the case of a category III dam. It is further required that the adequacy of the spillway should be evaluated in each safety inspection report on category II and category III dams.

Standards for deciding on the adequacy of spillways are not prescribed in the regulations. The underlying principle embodied in the regulations is that the engineer should apply accepted current practice, taking into account site-specific conditions. In these guidelines, the endeavour is to assist the practising engineer in the process of deciding upon the appropriate flood magnitude to adopt for sizing the spillway of a new dam or for evaluating the adequacy of the spillway system of an existing dam.

12.4 LEGAL CONSIDERATIONS

The following are selected quotations from the guidelines.

From a legal point of view, the main question is whether the engineering judgements exercised reflect reasonable care and prudence. Terms that frequently crop up in the courts are: "reasonable precautions to prevent injury", "a reasonable exercise of power given (to prevent damage)", "neglecting to take adequate precaution" and "reasonable care to see that unnecessary damage (is not caused)".

Two factors that are decisive in considering reasonableness are cost and the level of safety provided.

Two rulings from court cases are quoted and the following conclusions reached :

12.4.1 Reasonableness of cost

It thus follows that a public body is legally obliged to provide a structure to suit the circumstances, to do this within its financial resources and thus to reduce costs plus possible damages to reasonable proportions.

12.4.2 Reasonableness of risk

This extract starts with a court decision :

"In a country where rainfalls of great volume and severe intensity are common, and where meteorological data are scanty, I think that those on whom a duty in favour of others is cast to deal with flood water should be expected to provide a considerable margin of safety. A defence of vis major (or "act of God") should not be upheld save on the clearest evidence".

The flood that could be considered as "an act of God" would be "so extraordinary and devoid of human agency that reasonable care would not avoid the consequences".

And the guidelines continue :-

The question to be addressed by every spillway designer or safety evaluator remains: "What constitutes reasonable care and prudence in selecting the magnitude of the flood for which a dam should be designed or checked for safety?"

The problem faced by designers of dams, and by the public who use, pay for, and are affected by these structures, is to decide just how much protection should be provided.

It is not feasible nor even possible to provide absolute safety against all natural hazards. The objective should be to balance the benefits of providing a dam to meet a given need against the cost of increasing its safety beyond that associated with a reasonable level of risk. One must recognize that this does not mean trying to eliminate all risks but rather reducing them to acceptable levels.

12.5 DEFINITION OF FLOOD CRITERIA

The guidelines define four flood criteria.

The recommended design flood (RDF) is the flood event which has the recommended return period. The RDF with appropriate freeboard provides the basis for design of the spillway system.

The safety evaluation flood (SEF) is that incoming flood which, when routed through the reservoir and spillway system, may cause substantial damage to the structure and surroundings but must not be such as to cause the dam to fail catastrophically.

The imminent failure flood (IFF) is the inflow flood which, when routed through the reservoir, can be passed by the spillway with reservoir level just threatening failure of the dam. For an embankment dam, the IFF could be taken to be the incoming flow which just causes overtopping of the embankment. For concrete dams the IFF would be derived after consideration of structural stability and downstream threat to foundations under overtopping conditions.

Zero incremental impact flood (ZIF) is the minimum flood for which failure of the dam would cause no significant increase in the downstream damages and potential loss of life, ie zero incremental impact under present and foreseeable future conditions. This ZIF has to be determined by trial.

12.6 SELECTION OF DESIGN FLOODS

12.6.1 Quantification of total risk

Before proceeding with further quotations from the guidelines it is necessary to note the following :-

(a) There is no flood estimation method which is superior to all other methods in all circumstances.

(b) The reliability of the calculated flood peak-frequency relationship decreases with the length of the return period. In the case of direct statistical analysis methods the uncertainty can be reduced by undertaking regional analyses (Chapter 6).

(c) Where dams are equipped with gate controlled spillways, the risks of failure of the structure due to equipment malfunction or human error are very often significantly greater than those associated with the flood itself (Chapter 10).

(d) The probability of failure of a dam is *not* that of the exceedance probability of the flood. There is a second probability which is the probability that the dam will fail should the flood occur. The probability of failure of the structure is the product of the exceedance probability of the flood and the probability associated with the mode of failure of the structure should the flood occur. If these are independent the probability of failure is the product of the two probabilities and this is appreciably smaller than the exceedance probability of the flood on its own.

(e) Experience in South Africa and elsewhere has shown that contemporary structural design standards are such that modern dams can withstand floods well in excess of their design floods without failure (Alexander, 1988).

(f) More modern dams have failed from causes other than from floods that exceeded the design value. There have been a number of failures associated with floods significantly lower than the design value.

(g) Underestimation of the flood magnitude is not linearly related to increase in risk of structural failure. Due to the flood absorption characteristics of the dam basin, the additional inflow will not result in a proportional rise of the water level in the dam and therefore of a proportional increase in head on the spillway. Furthermore, the incremental head required to discharge the additional flow over a dam spillway or through flood control gates may be appreciably less than the proportionate increase in inflow.

(h) Incremental downstream risks associated with increase in flood discharge are not linearly related to the increase in flood discharge. Firstly, the increment in water level is not linearly related to increase in river flow. Secondly, large floods will cover the flood plain and therefore the (already reduced) increase in water level will inundate an incrementally smaller area of the rising ground adjacent to the flood plain.

If it were possible to quantify all of these risks, it would be a relatively simple procedure for the designer to select the tolerable risk of failure - however small - and then determine the optimum combination of risks that would not exceed this optimum. For example in the case of a specific earth embankment dam it may be estimated that there is an 80% probability that the wall will breach catastrophically should the water level rise more than 1 m above the crest. This level may be equivalent to a 500 year return period flood. If the design life of the structure is 50 years, there is a 9,5% probability of the water in the dam rising to this level during the design life and consequently a 7,6% risk of the dam failing during the design life. However, in the case of an alternative mass concrete wall design, the risk of the wall failing catastrophically may be 1% and consequently there is only a 0,095% risk of failure of the structure during the design life of the project. If the risk associated with the earth embankment is too high, the costs of adjusting the design to reduce the risk to an acceptable level would be compared with the cost of the alternative concrete wall.

Another example is the relatively higher risk of failure of dams equipped with flood control gates due to power failure, equipment failure, or human error. The probability of gate failure is appreciably greater than that of the design flood being exceeded, and care has to be taken that the joint probability of gate failure and flood exceedance does not exceed the tolerable risk of failure of the structure. This requires the implementation of measures that will ensure that these potential risks are reduced to an acceptable level of total risk.

In practice it is only the flood peak exceedance probability that can be estimated with a fair degree of accuracy, while most other risks are unquantifiable. Nevertheless the designer should appreciate the basic concept of risk analysis insofar as it affects the safe design of the structure.

The guidelines describe three ways of accounting for risk. These are generalized design standards, zero incremental impact, and risk-based analysis. These methods are detailed in the guidelines and summarized below.

12.6.2 Generalized design standards

Table 12.4 from the guidelines tabulates the minimum values for the RDF in terms of return period floods; and Tables 12.5 and 12.6 the SEF in terms of the regional maximum flood (RMF) and return period floods.

Recommended design flood (RDF)

Table 12.4. : Recommended minimum return period values for design flood (RDF)

Dam size	*Hazard rating*		
class	*Low*	*Significant*	*High*
Small	Q_{20}-Q_{50}	Q_{100}	Q_{100}
Medium	Q_{100}	Q_{100}	Q_{200}
Large	Q_{200}	Q_{200}	Q_{200}

Safety evaluation flood
References to the *transition zone* and the *flood zone* in the tables below refer to the corresponding catchment sizes in TR 137 (Kovåcs, 1989) which are reproduced in Table 8.5 in Chapter 8.

Table 12.5 : Recommended safety evaluation flood (SEF) values for catchments in the flood zone

Dam size	*Hazard rating*		
class	*Low*	*Significant*	*High*
Small	Q_{100}	Q_{200}	RMF
Medium	Q_{200}	RMF	RMF $(k + \Delta)$
Large	RMF	RMF $(k + \Delta)$	RMF $(k + 2\Delta)$

Where Q_{100} and Q_{200} are derived by using methods recommended in this handbook

and where Δ $= 0{,}2$ for $k \geq 5{,}0$
and $= 0{,}3$ for $k < 5{,}0$

Sien tabel 6.5

Table 12.6 : Recommended safety evaluation flood (SEF) values for catchments in the transition zone

Dam size	*Hazard rating*			
class	*Low*	*Significant*	*High*	
Small	Q_{100} PMF · RMF/RMF	Q_{200} PMF · RMF/RMF	RMF	PMF × RMF/RMF
Medium	Q_{200} "	RMF PMF · RMF/RMF	1,3 RMF	PMF
Large	RMF PMF · RMF/RMF	1,3 RMF PMF	1,7 RMF	PMF

To satisfy the requirement that no significant damage be caused during normal operating conditions the RDF values in Table 12.4, with the recommended freeboard allowances, may be adopted for the design of a new spillway or for the redesign of an existing spillway. The design must then be checked to ensure that the recommended SEF can be accommodated without causing catastrophic failure by overtopping.

The SEF values in Table 12.4 are also intended as a sieve to identify existing dams that require further investigation of spillway adequacy. Spillways that can accommodate the indicated SEF, suitably routed, without causing catastrophic damage need no further investigation. For spillways that fail to meet the criteria, site-specific analyses are recommended to ascertain the degree of inadequacy and required remedial work.

The SEF values may also be used to evaluate the adequacy of the spillway of a less important new dam but site-specific analysis is strongly recommended for evaluation of the safety status under extreme flood conditions for all high hazard dams as well as for medium and large dams having significant hazard ratings.

Although small dams having low hazard ratings are not subject to comprehensive safety evaluation by professional engineers, values of the RDF and SEF are included in Table 12.4 as a general guide. Sizing of spillways for such dams is usually decided purely on financial grounds.

12.6.3 Zero incremental impact approach

The cornerstone of this method is that spillway capacity is determined from an evaluation of the incremental impact experienced in the area downstream of the dam. The minimum criteria set by the general design standards may be considered as the starting point of the analysis and the endeavour is to assess whether the risks associated with the project would justify relaxing or tightening of the generally accepted criteria. The approach is a recent development and has been recommended for the safety evaluation of the high hazard dams in

This method requires the imminent failure flood (IFF) be evaluated. The guidelines list the following procedure to be followed in the case of an existing dam :

(a) *Determine the SEF recommended for the category of dam in Table 12.4.*

(b) *Determine the IFF.*

(c) *If the IFF is less than the recommended SEF, assess the incremental impact due to a failure during the SEF. If the incremental impact is clearly so severe that overtopping and failure of the dam simply cannot be countenanced design adequate measures to avert failure of the dam.*

If, however, the incremental impact is not significant, proceed to step (d).

(d) *Determine the ZIF by stepwise trials.*

(e) *If the ZIF is less than the IFF, consider the consequences of dam failure and loss of project services at the probable frequency associated with the IFF. If it is judged that such risks can be tolerated, no additional work to provide further safety against extreme flood would be indicated.*

(f) *If the ZIF is greater than the IFF, or if it is considered that the consequences of dam failure are unacceptable, proceed with a risk-based analysis to develop a further basis for a decision on alternatives.*

The following aspects are relevant to the assessment of the IFF as listed by the Australian National Committee on Large Dams (ANCOLD), 1984:

(i) *The imminent failure flood (IFF) should be based on a reasonable but not optimistic assessment of the conditions that could lead to a major failure. Some damage of an embankment dam, such as scouring of the crest and downstream of the spillway energy dissipater, can be accepted provided they will not cause failure, even though substantial repairs may be required.*

(ii) *For embankment dams, it may be practicable to select the IFF as the flood giving a stillwater level at crest level (excluding camber) allowing wave splash onto the crest, provided other factors such as stability are satisfactory. Leakage through the crest road foundation above the core can also be significant.*

12.6.4 Risk-based analysis

By routing the hydrographs downstream one can establish the frequency distribution of downstream damage without dam. The hydrographs can then be routed through the reservoir and downstream river reaches with various capacities of spillway system, with (if appropriate) reservoir operation rules, to establish the frequency distribution of with dam damages as a function of the capacity of the spillway system. The dam must be presumed to *fail as the capacity of each spillway system tested is exceeded. There will thus be a frequency distribution of failure and therefore of damages due to failure. By integrating the frequency distributions (that of damages with and without dam failure) one can establish for each the total damage, or the average annual damages. The net or incremental damage is the difference between the two integrations.*

It stands to reason that the greater the capacity of spillway system that has been provided the lower will be the frequency of failure. It may be noted, however, that the annual damage likely to result from dam failure is not closely related to the capacity of the spillway system. The dam-break damage will be much the same whatever the size of the spillway. It is its frequency that diminishes with increasing capacity. Once the dam-break damage has been established, therefore, one can merely multiply the damage cost by its annual probability to yield the annual damage cost.

12.6.5 Guideline conclusions

The guidelines conclude with the following remarks:

It is well to emphasize that the guidelines do not constitute a Code of Practice and that there is no intention to prescribe the use of any specific hydrologic technique. It is nevertheless expected that modern techniques be adopted and that sound engineering judgement be exercised in deciding upon design flood and safety evaluation flood values, as well as in reaching decisions as to whether an existing spillway is to be considered satisfactory or requiring to be upgraded.

It goes without saying that site-specific investigations may well lead to acceptance of higher or lower flood values than have been indicated as minimum values in Table 12.4. It follows, however, that adoption of lower values must be strongly motivated by carefully checked site-specific analyses.

12.7 UPPER LIMIT FLOOD

The designer of a dam in a high risk category should consider the following factors when applying the guidelines.

The concept of an upper limit flood derived from the probable maximum precipitation is described in Chapter 3 on storm rainfall and detailed in Chapter 7 on the deterministic methods and Chapter 9 on the design flood. The determination of the regional maximum flood (RMF) is detailed in Chapter 8 on empirical methods. These sections should be read again before carrying on with the comment below.

12.7.1 Extrapolation of the flood frequency relationship

Risk-based analysis requires that annual (exceedance) *probabilities or estimated return periods be assigned to extreme events. Since there are few places where flow records extend back as far as 100 years, it follows that statistical return periods cannot be accurately assigned much beyond about 50 years. The band of confidence embracing the frequency distribution curve widens rapidly for return periods longer than about half the length of the record.*

Despite the fact that rainfall records are usually longer than those of river flow, much the same reasoning applies to flood hydrographs established by hydro-meteorological techniques in which flood response is deduced from the causative storm rainfall. Nevertheless, it is possible, by means of storm transposition and maximization techniques, to make a fairly good estimate of the Probable Maximum Precipitation (PMP) and, by unitgraph or other technique, to arrive at a reasonable estimate of the Probable Maximum Flood (PMF) (-but see para 12.7.2 below).

The PMF is the value to which the frequency distribution of flood peaks should become asymptotic at some extremely low probability or large value of return period which, for purposes of risk-based analysis, could be arbitrarily set at between 10^4 and 10^5 years. At a *somewhat shorter return period would lie a value now referred to in South Africa as the Regional Maximum Flood (RMF) which can be estimated with acceptable reliability by methods developed by Kovács in Technical Report TR 137.*

Whatever the statistical frequency distribution that may have been fitted to an array of recorded or calculated flood peaks, one recognises that flood magnitude cannot continue to increase indefinitely with lengthening return period (or decreasing probability) but rather must reach the PMF asymptotically at some very large value of return period. In 1973 the ASCE Task Committee on Re-evaluation of the Adequacy of Spillways of Existing Dams (ASCE, 1973) recommended the arbitrary assignment of a return period of 10 000 years to the PMF. In the United Kingdom, however, a 10 000-year return period is normally assigned to the half-PMF.

Referring to Fig 12.1 as an example one therefore may have a frequency distribution of flood peaks, derived either from an actual record or by hydro-meteorological technique, up to say the 100-year event as well as a ceiling value - the PMF. For purposes of economic analyses one can then sketch in the frequency distribution curve between the 10^2-year and 10^4-year to 10^5-year return periods as a rough guide to the frequency distribution of flood peaks in terms *of preliminary economic optimization.*

Fig 12.1 represents an approach similar to that adopted in the interim guidelines on design floods for dams published by the Australian National Committee on Large Dams in 1984, which in turn is based on the method for constructing composite flood frequency curves proposed by the United States Bureau of Reclamation (1981) where the relationship is assumed to become asymptotic to the PMF value which is arbitrarily allocated a return period of 10^4 years. Both the South African and Australian guidelines suggest that this relationship can be used for economic risk-benefit-cost analyses.

The graphical presentations used in Chapter 2 on statistical methods have either logarithmic or linear vertical scales and horizontal probability scales. Data sets plotted on these scales approximate a straight line on logarithmic scales and strong upward curvature on linear scales. However, if the same data set is plotted on linear vertical and horizontal scales the strong negative curvature would be apparent. This is also borne out by the mathematical formulation of the cumulative probability distribution functions which do not have an upper bound. In these cases the exceedance probability is asymptotic to zero when the flood magnitude approaches infinity, but has very low but non-zero values at moderate flood magnitudes.

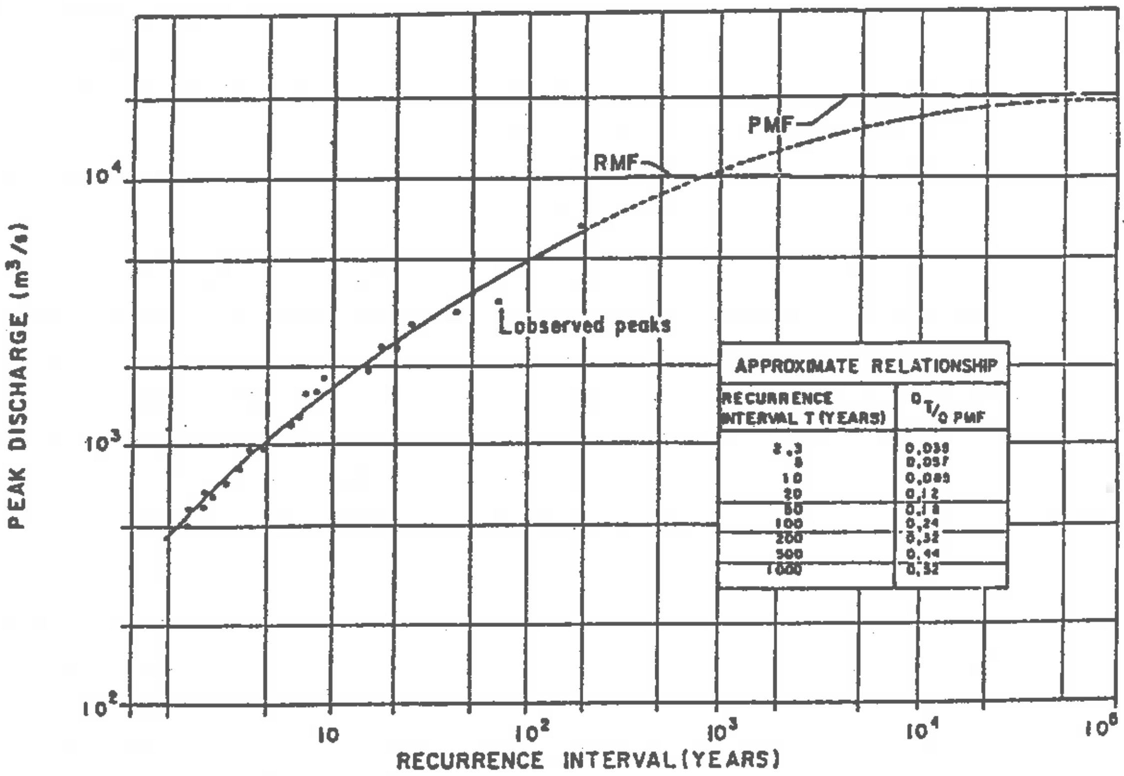

FIGURE 12.1 : Extrapolation of a typical flood frequency distribution

12.7.2 Probable maximum flood (PMF)

There have been severe criticisms of the concept of a probable maximum flood. These criticisms are based on statistical grounds as well as on knowledge of the hydrological processes. This is dealt with in detail in Chapter 9 on the design flood. Because of the confusion and conflicting views that exist the position is summarized again below.

The concept of a definable upper limit cannot be sustained on the grounds of the meteorological or hydrological processes. For example meteorology has not yet advanced to the stage where the depth-area-duration-frequency properties of storm rainfall can be determined from meteorological considerations. These still have to be based on point precipitation data (see Chapter 3 on storm rainfall). Development of the probable maximum precipitation (PMP) requires estimates of simultaneous maxima of the depth, duration and area of the storm duration. These estimates must be suspect, particularly in South Africa with its wide range of storm rainfall mechanisms.

From a hydrological process viewpoint the evaluation of the PMF via the PMP is even more untenable. If the PMF is defined as the boundary between the possible and the impossible flood then it follows that the catchment must be in the maximum possible antecedant moisture condition immediately prior to the PMP storm. For small, relatively impervious catchments in moderate to high rainfall areas this is not an unreasonable assumption because the duration of the PMP storm is likely to exceed the catchment response time, and so raise the antecedant moisture status to near saturation. However, this becomes less probable the larger the catchment, the greater the permeability, and the lesser the mean annual rainfall until it becomes unacceptable for large catchments in arid areas.

Another point to consider is that the PMP estimates used in procedures in the USA and Australia are based on PMP storms which include the area and time distribution of the rainfall. Although similar earlier information is given in Appendix D to HRU1/72 as well as recent publications in the technical report series of the Department of Water Affairs and the Weather Bureau, the problem of storm maximization and transposition remains. Fig C.4 from HRU 1/72 which is reproduced in Chapter 13 shows envelopes of maximum recorded point rainfalls in South Africa and the world. These are not PMP values. Similarly, the estimates of maximum storm losses in Fig C.1 which is also reproduced in Chapter 13 are empirically based estimates.

12.7.3 Regional maximum flood (RMF)

The RMF method (Kovâcs 1980, revised in 1989) is also based on envelope values of South African maximum observed floods. Rodier and Roche (1984) published a world catalogue of maximum observed floods. Both publications recommend the use of the index suggested by Francou and Rodier (1967) for categorising flood peaks into hydrologically homogeneous regions (see Chapter 8 on empirical methods).

The RMF approach does not purport to provide an estimate of the PMF but if considered in conjunction with the world catalogue of maximum observed floods it will provide an estimate that is at least as realistic as the PMP/PMF approach based on PMP storms.

12.7.4 Conclusion

Problems relating to the selection of the upper limit flood have been dealt with in some detail because this is a critical aspect of dam safety evaluation. The groupings of RMF and PMF in Table 12.1 should not be expected to give the same results. Different designers will subjectively give more weight to one method than another depending on their past experience and the nature of the problem.

All designers should be aware that while prudent conservatism is desirable, particularly in high risk situations where there is no severe economic disbenefit, this does not imply the need for the selection of improbable flood magnitudes with exceedance probabilities out of proportion to all the other risks associated with dam design, construction and operation.

The quotation from Benson (1964) in Chapter 9 is worth repeating :

It is necessary that engineers admit to themselves and to the public that there is no such thing as complete safety, and that some degree of risk is involved in any engineering structure, even one whose failure would entail heavy damage and loss of life.

12.8 OTHER CONSIDERATIONS

12.8.1 Initial storage

The normal practice is to assume that the reservoir is full at the start of the design flood. This is a reasonable assumption due to the high state of antecedent catchment moisture and corresponding antecedent river flow associated with subsequent severe floods. The possibility that the initial water level will be higher than the normal operating level is discussed in the next section.

12.8.2 Flood sequences

In most spillway designs the required spillway discharge capacity is that of the design flood peak, and the flood peak attenuation characteristics of the dam basin are ignored. This is a conservative assumption in the case of uncontrolled spillways. However, in the case of dams equipped with partially or fully gate controlled spillways, the gates can usually be operated to achieve a degree of flood peak attenuation using operating rules such as those described in Chapter 10. It must be appreciated that an inevitable consequence of flood peak attenuation is an increase in the reservoir water level, and a decrease in the ability to attenuate subsequent floods until the water level is back to the operating level.

Consequently, in the case of dams with partially or wholly gate-controlled spillways the critical situation may well arise from a *combination of several hydrographs in close succession rather than from a single hydrograph.* The larger the catchment the greater the probability that a succession of hydrographs may occur. The Vaal Dam floods of February, 1975 detailed in Appendix 3F to Chapter 3 are an example.

Our current knowledge is such that the relationship between hydrograph volumes, shapes, spacing in time and exceedance probability cannot be determined theoretically. Indeed such a wide combination of these variables is possible for a specified exceedance probability, that it would be close to impossible to determine the reasonableness of any selected combination. As a result several agencies have adopted empirical policy rules.

For example the standard project flood (SPF) is adopted as the antecedent flood by the US Army Corps of Engineers (Stallings,1987). The SPF flood discharges are typically 40-60% of the probable maximum floods for the same catchments.

In the UK it is assumed that the reservoir is full before the flood and that the inflow is equal to the long term average river flow. These conditions when coupled with an allowance for wave height and wave run-up represent a combination of extremes which can lead to a probability of exceedance well beyond the 1:100 000 years or more which can be considered as too extreme a risk to be designed for (Kennard and Bass, 1988).

12.9 CONCLUSION

Flood risk evaluation for dam safety should not be undertaken by a person who does not have a sound knowledge of flood hydrology. The evaluation requires a high degree of experience based judgment to determine the narrow band of design flood magnitude which provides a reasonable margin of safety without excessive caution that would require unwarranted expenditure by the beneficiaries, loss of potential income, and reduction in benefits for years to come.

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Chapter 13

CALCULATION PROCEDURES AND COMPUTER PROGRAMS

Contents

13.1 INTRODUCTION

This chapter provides users with information required for applying the methods recommended in this handbook. Refer to the relevant chapters in the handbook for the theoretical basis and relative merits of the methods presented here as well as for information on the interpretation of the results.

Details are provided for both hand and computer calculation methods but the latter are preferred as they will provide consistent error-free results (in so far as the algorithms are correct), and both the input data and the results of the calculations can be kept as a permanent record. Emphasis is therefore on the computer implementations.

13.2 EQUIPMENT

Computer
The programs will run on any IBM-compatible microcomputer. As graphical presentations are an essential part of the calculations, the computer must have a graphics card.

Printer
Either an Epson-compatible dot matrix printer or HP LaserJet-compatible laser printer can be used for recording the results of the calculations.

Plotter
The graphs on the screen can be dumped onto the printer, or higher resolution graphs from the regional analysis program can be plotted by a plotter that recognises Roland DXY or Hewlett Packard HPGL plotting languages. Commercial software is available for using the HP LaserJet as a plotter to produce higher resolution graphics than is possible with the screen dumps.

13.3 BASIC

All of the programs accompanying this handbook were originally written in GW-BASIC or Borland's TurboBasic and subsequently compiled using TurboBasic. It is realized that BASIC has shortcomings when used for hydrological calculations, but the overriding consideration is its simplicity, universal availability and usage. The programs can all be run directly without having to use TurboBasic. ASCII versions of the principal programs are provided on the disks.

All programs are written in a 'no frills' portable BASIC. They are well structured within the constraints of simplicity and being readily understood by the users. Liberal use is made of remarks as well as partitioning of the programs so that users with a moderate knowledge of programming will be able to modify the programs to suit their own needs. It should be relatively easy to compile the ASCII versions of the programs directly or to re-write them into other languages.

All intermediate calculation results are presented on the screen and automatic optimisation algorithms have been avoided deliberately - for example in the calculation of the flood peak in the unit hydrograph method. The user can therefore follow all the calculations as they progress.

The screen graphics commands used are standard commands but require an appropriate graphics card in the microcomputer. SCREEN 2 which has a coarse 600 x 200 pixel resolution should run on most graphics cards. Alternative selections for higher resolution graphics are provided in the programs. The best resolution is obtained by drawing the graphs by a plotter.

13.3.1 Origin of material used in the programs

All of the programs were written by the author with some student assistance. The algorithms used in the direct statistical analysis methods programs were based on the theoretical background detailed in the handbook.

In all the other programs an attempt was made to follow the original methods as closely as possible. Where these methods required interpolation from graphs and tables (most of the Wits HRU methods), equations were fitted to the graphs, and the tables were included in the programs. Due to curve fitting procedures and linear interpolation of the tables, minor discrepancies between the original methods and these computer implementations must be expected. However, these are well within the range of uncertainty inherent in all methods. Alternative implementations for the unit hydrograph, and rational methods were developed by the author.

13.3.2 Caution

Although all reasonable care was taken in the development of the programs, there may well be undiscovered errors or bugs in them. Users should satisfy themselves that the programs meet their particular needs. It is recommended that the results obtained when using the programs be tested against the results of earlier calculations. If there are unacceptably large, persistent differences the reasons for these should be sought.

13.4 GETTING STARTED

13.4.1 Creating backup disks

The programs are not copyright. The disks are protected with write protect tabs to prevent accidental corruption or erasure. Back-up copies should be made before running the programs, and the originals kept for reference purposes. Refer to the computer's DOS manual for details. It is strongly recommended that separate disks be used for each investigation, and that these disks and the computer print-outs be archived with the rest of the calculations for the project.

The second generation disks are your "master" disks. With the passage of time you may wish to make permanent alterations to the programs. For each new investigation copy a set of the latest versions of the master programs onto working disks, copy the data onto the data files, run the programs, and then store the disks in the calculation files for the project.

Make a set of working disks and use these from here onwards.

13.4.2 Program listings

The README file as well as the ASCII versions of the programs can printed on a printer by using the **TYPE X:####.###>PRN** command where X is the drive, either A or B, and ####.### is the name of the file. Alternatively use the non-document option of your word processing package to load and print the ASCII files. Print these program listings and file them for record purposes.

13.4.3 Suite of programs

Each disk is issued with a list of programs. These will be the latest versions at the time of issue of the programs and may change as programs are expanded, revised or supplemented.

The file README contains information on the programs, particularly changes made subsequent to the publication of this handbook.

13.4.4 Types of programs

The following suffixes are used:

.BAS	BASIC programs
.EXE	Compiled programs
.#00	Master programs with provision for storing data
.DEM	Programs containing demonstration data
.SUB	Sub programs used for compiling REGFLOOD and REGPLOT
.DTA	Data files generated by REGDATA

Other suffixes can be selected by the user

13.4.5 Available programs

README
This short file details latest revisions and other information that has become available subsequent to the printing of the handbook.

SETUP.EXE
This program sets up the printer and allows you to test your screen graphics facilities.

DETFLOOD.EXE
Use this program for the rational, unit hydrograph and regional maximum flood calculations.

STFLOOD.EXE
This is an exploratory program for direct statistical analysis of single station records. Unlike REGFLOOD it makes provision for incorporating data directly in the program, or keying in data or statistical properties directly from the keyboard. It does not have facilities for printing or plotting the results.

REGDATA.EXE
These programs incorporate data which the program uses to create the data files which are required for the program REGFLOOD. Instructions for entering the data are given on the screen when the program is run.

REGFLOOD.EXE and **REGPLOT.EXE**
These are long and fairly complex programs for carrying out direct regional statistical analyses.

Files with the suffix **.SUB** are included in the main REGFLOOD and REGPLOT programs when these are compiled, so the ASCII versions must be on the same disk as the ASCII versions of REGFLOOD and REGPLOT.

A number of demonstration data files with the suffix .DTA are included on the disks. These are required for the case studies in Chapter 14.

13.4.6 Running the programs

The programs have a number of error traps to prevent the programs from hanging up if the incorrect response is typed, but these cannot cover every eventuality. If the screen goes blank or the program freezes when running a compiled program you will have to reset the computer by pressing the reset button and starting all over again, so be careful when responding to the screen instructions. If you have a BASIC compiler rather run the ASCII versions of the programs from within the compiler. An error message will indicate the nature of the error.

Use the following procedure when running the programs for the first time in order to determine whether they will run satisfactorily on your microcomputer. If your computer hangs up due to an incorrect entry, use one of the remedial measures described below to get going again.

The instructions below assume that the working disk is in drive A. Substitute the actual drive if the disk is not in this drive.

1. Remove all resident programs from memory as the flood calculations programs will need all the available memory. Check the AUTOEXEC.BAT file to ensure that this does not include any extraneous programs and the CONFIG.SYS file if you previously increased the number of buffers and files, for example for running CAD programs. Refer to the DOS manual for details.

2. Load MS-DOS. This will be automatic if you have the MS-DOS disc in drive A or on the hard disk. You must use the version of MS-DOS supplied with the microcomputer as different makes of microcomputer have different implementations of MS-DOS. Thereafter type in the commands shown in bold type. Include the quotation marks where these are shown, and press the carriage return key after each command. This key is variously marked RETURN, or ENTER, or with a reversed L-shaped arrow.

3. You will have to load the appropriate graphics command if you want to dump the screen images onto a printer. These commands are needed for the screen dump but not the text output or screen graphics display. If you are using a dot matrix printer load the DOS file GRAPHICS.COM by typing **GRAPHICS** then pressing the ENTER key. If you are using a LaserJet printer you must first load the command **HPSCREEN** which is on the disk that is issued with the printer.

4. Insert disk #1 in drive A.

   Before running the setup program make sure that your printer is switched on.

   Enter **SETUP.** Run the program and follow the instructions. Reply yes to all the questions if you are using A4 (or 12 inch) length paper in the printer. The program will have to be run again if the printer is switched off during the session. If you get garbage on the printer check the printer's manual to ensure that the DIP switches have been correctly set.

   The option to test the graphics output need only be run once. Note the highest resolution that the graphics card will produce and which of the screen resolutions can be dumped on your printer, as you will need this information when running the programs. If you do not get any graphs on the screen it is probably because you do not have a suitable graphics card in the microcomputer. If the screen graph is satisfactory but you get garbage on the printer it is probably due to one of the following.

- You did not load the graphics command (para 3 above).
- The PRINT SCREEN command does not recognise the selected SCREEN resolution (there are differences between GWBASIC and compiled BASIC). All versions should be able to dump SCREEN 2.
- The printer does not accept the standard IBM/Epson graphics commands.

5. Enter **STFLOOD.** The program details will appear on the screen.
6. Read the first two pages on the screen.
7. When you reach the menu select the first item (Vaal at Grootdraai Dam).
8. No printer options are provided in this program. Press the PRINT SCREEN key when a table appears on the screen. After the table has been printed advance the paper to the next page by pressing the form feed button on the printer.
9. Follow the instructions on the screen until you reach the "FURTHER OPTIONS" menu. If you get that far then all the other calculations on the disk with text outputs will be performed satisfactorily.
10. Next see if the screen graphics are displayed. Select "1 Graphs on the screen" option.
11. Then select "2 Medium resolution graphics". This should work with most graphics cards.
12. In the next menu select "1 Log normal". When the screen graph appears, press the PRINT SCREEN key.
13. After the completion of the dump press the form feed button on the printer to eject the page. Press any other key to return to the graphics menu.
14. Return to the main menu and test the other options. No more dumps are required at this stage. Stay in the program until you become familiar with its operation.
15. Exit from this program then type **DETFLOOD.** This program includes the unit hydrograph, rational, and regional maximum flood methods and also includes sample data. Exit from the program when you are satisfied that you have mastered it.
16. Now type **REGDATA.** This program is used to generate data files for the program REGFLOOD. Run the program as it stands to generate the SAMPLE.DTA file. Note the instructions for creating additional data files.
17. Change disks and type **REGFLOOD.** This is a long program and you may have some difficulty when running it. If the program hangs up (nothing happens when you press the required key) press the reset button on the computer and try again. The most likely causes are incorrect responses or insufficient memory. You will

have to load the appropriate GRAPHICS command after each reset. If you have a BASIC compiler then it is preferable to run the ASCII version of the program from within the compiler when the reason for the error will be identified. You will need a BASIC compiler if you wish to make changes to the program.

18. The REGFLOOD program will take some time to master, so be patient when using it for the first time. More details on operating the program are given below.

    Once you have mastered the programs using the sample data you will be ready to carry out your own analyses.

19. Yes/no options in the menus make use of the "Y=1 N=0" construct. In most cases you will be using the "No" answer in which case simply press the ENTER key.

20. Similarly, it is not necessary to key in information such as the description of the site, your name, date, etc unless the output is being sent to the printer. Press the ENTER key alone if this information is not needed.

21. When the output is to the screen only, a "Press any key to continue or S to stop" message will appear to enable you to examine the program output. If you wish to exit from the program at this point press the S key, otherwise press any other key to continue.

22. If you wish to exit from the program at any other time, for example when you have entered an incorrect response to a question or when the computer hangs up then press the CTRL key and while holding it down press and release the BREAK key. This won't help if you are using a compiled program, so if all else fails press the reset button and start again.

13.5 COMPUTER PROGRAMS

13.5.1 Changes to the programs

The ASCII versions of the programs can be accessed and changed to meet the user's requirements. However, it is important that the changes be documented for future reference. The first three lines of each program are the name of the program, the revision number and the author of the program respectively. When the program is run the user is invited to enter his name and the date of the analysis. This information is recorded on the printed calculations. For example the first five lines of the program REGFLOOD are :

```
PROGRAM$     = "REGFLOOD.TB"
REV$         = "Rev 1f 89/08/29"
AUTHOR$      = "WJR Alexander"
```

```
REM
REM  $INCLUDES WAKEBY.SUB and screen graphics file SCREEN.SUB
```

If you have made any modifications to a program add appropriate short comments to the second and third lines, and detail the changes in the fourth line. For example.

```
PROGRAM$     = "REGFLOOD.TB"
REV$         = "Rev 1f 89/08/29 modified 90/01/25"
AUTHOR$      = "WJR Alexander modified by AB Smith"

REM  Changes made to output formats to fit our calculation sheets
REM  $INCLUDES WAKEBY.SUB and screen graphics file SCREEN.SUB
```

Note the position of the quotation marks in the first three lines.

13.6 DATA ASSEMBLY

Make a number of photocopies of the data sheet Annexure 13A for future use.

13.6.1 Deterministic methods

Enter the following data on the data sheet.

1. Locate the site on 1:50 000 or 1:250 000 topographical maps.

2. Determine the following catchment characteristics for the site.

   (a) Demarcate the catchment boundary on the 1:50 000 topographical maps, or 1:250 000 maps if the catchment covers more than four 1:50 000 sheets.

   (b) Measure the area of the catchment. Subtract areas of significant internal drainage (eg large pans) if any. Use transparent graph paper with 2mm squares. One hundred squares have an equivalent area of one square kilometer on a 1:50 000 scale map. Count the number of squares to determine the area.

   (c) Produce a longitudinal profile along the longest tributary from the site to the watershed. Use dividers for measuring the main stream length. These should be set at 0,2 km for 1:50 000 maps and 1,0 km for 1:250 000 maps. When the latter maps are used the length should be multiplied by a factor of 1,2 to correct for loss of resolution.

   The distances along the length of the stream where the contour lines are crossed should be used to plot the profile. Where waterfalls and rapids are clearly evident as discontinuities in the profile, the profile should be adjusted downwards to eliminate them.

(d) Determine the height differences along the equal area and 1085 slopes.

(e) Locate the centroid of the catchment by eye and measure the distance along the main channel length from the site to a point opposite the centroid.

3. Read the veld type zone from Fig 13.1. If the catchment straddles more than one zone undertake two separate calculations assuming the two different veld zone numbers and interpolate the results. For manual calculation read the corresponding values for C_t and K_u from Tables 13.1 and 13.2.

4. Determine the mean annual precipitation (MAP) over the catchment. There are several ways of doing this. The following method is recommended on the grounds of readily available information and the need for a consistent method. The catchment MAP is the average of the quaternary catchments within which the catchment of interest is located as shown in the HRU series of publications *Surface water resources of South Africa.* It is not necessary to refine the estimate except where the catchment MAP is expected to differ from the HRU MAP by more than about 10% to 15%. Note that this method was used to calibrate the alternative unit hydrograph and rational methods below and consequently a "more accurate" method for determining the catchment MAP will provide less accurate results!

5. Refer to the DWA Technical Report TR 102 *"Southern African storm rainfall"*. Locate all rainfall stations in or near the catchment. For each station note the length of the record (an indication of its reliability), and the mean annual precipitation (MAP). The MAP will be used to adjust all the other values. Select the station which has a MAP nearest to the average catchment MAP derived in the previous paragraph. It is not necessary to calculate average or weighted average TR 102 values of several stations unless there is a wide variation of values between the stations.

6. Determine whether the catchment is located in the coastal or inland region from Fig 13.2.

7. Determine the catchment characteristics required for the rational method as listed on the input data sheet Annexure 13A for the alternative rational method, and the calculation sheet Annexure 13B for the DWA method.

8. Derive the lightning ground flash density from Fig 13.3.

9. Note the presence of any dams upstream of the site. The risk of one or more of these dams breaching will increase the flood risk at the site (or increase the peak for the same risk). Conversely these dams may attenuate the flood peak for low return period flows. In most cases the effect of upstream dams can be ignored.

10. Identify the regional maximum flood (RMF) region in which the site is located (Fig 13.4), and determine the value of the RMF *K*-factor.

11. Similarly, determine the 24-hour, 2-year return period rainfall from Fig 13.5.

12. Enter the required PMP values on the form Annexure 13A if these differ from those in Fig 13.6.

13. Add any other comment relevant to the analyses and interpretation of the results.

13.6.2 Data for direct statistical analysis methods

The single station statistical analysis methods program STFLOOD incorporates several sets of historical data, including one on maximum daily rainfall, which are useful for demonstrating the use of the method as well as for interpreting the graphical displays.

A data file SAMPLE.DTA which was produced by the program REGDATA is included in on the disk so that the program REGFLOOD can be run without first having to create a data file for it.

When carrying out a new set of analyses proceed as follows:-

1. The locations of gauging stations are given in the Department of Water Affairs (DWA) Hydrological publication Nr 12 *List of hydrological gauging stations.* See also the HRU report series on the surface water resources of South Africa where the positions of the stations are mapped.

2. Obtain the monthly flow records of the required gauging station (for the program STFLOOD), or records of all stations in the region (for REGFLOOD) from the Division of Hydrology of the Department of Water Affairs. Discuss the reliability of the data, particularly in the high flow range. It may be necessary to extend the rating curves to obtain a better estimate of the high flows of interest. After this has been done abstract the series of maximum annual peak flows ie the single largest value of the twelve monthly peaks in each hydrological year in the record for each station.

3. For single station analysis this data can be incorporated in the program STFLOOD, or entered from the keyboard. Data for the program REGFLOOD have to be incorporated in suitable data files as described below.

13.6.3 Creation of data files for regional statistical analyses

Introduction

The program REGFLOOD requires the prior creation of data files containing the data for the set of stations being considered.

There are several methods that can be used to create data files. Of these a sequential data file is the simplest and least susceptible to errors by inexperienced users. The data can be checked by listing the program used to create the file. Experienced programmers may create a single more sophisticated random access file if they so wish - this will require minor modifications to the main program. The advantage of a random access file is that data can be added from time to time, and that the names of all available data sets can be listed on the screen and only those required for the analysis selected.

The program REGDATA is used for creating sequential data files for REGFLOOD and REGPLOT. Load the program by typing **REGDATA** and follow the screen instruction.

Proceed as follows :-

1. Enter **REGDATA**.
2. Follow the instructions to create the data file **SAMPLE.DTA.** (As it is already on the disk it will be overwritten.)
3. Follow the instructions to create a new data file.

Thee number of stations that REGFLOOD can process depends on the maximum record length, the number of stations, and the available memory in the computer. Where possible limit the data files to about 20 stations rather than attempting to process a large number of stations in a single batch. If there are more than about 20 stations in the region, process them in batches of 20, select the 20 most representative stations, then create a new data file from REGDATA containing the selected stations.

Note that the starting year is the year starting October and the end year is that ending in September. The difference between the two is the number of values that have to be read into the data file.

All data values have to be integers as decimal values are added to the data to identify the year of occurrence. This is required to retain this information during the ranking process, and is stripped off again after the ranked data have been tabulated. If decimal values are required in the analysis multiply all data by 10 (or 100) before entering them in the program, and note this in the comment so that it is not overlooked in the subsequent analyses.

Any other relevant comment can be entered, but do <u>not</u> use commas or colons to separate out the comment - rather use hyphens as in this sentence. The comment must not extend beyond about 60 characters as the space on the graphs is limited.

It is essential that all of the required information is included in the program. If the information is unknown or not required, enter -1, otherwise you will get an error message when the program is run.

Note that the name of the data file must not be the same as a name that has been used previously and that the last four characters must be .DTA. The following examples would be acceptable :

> **VAAL.DTA**
> **NATAL.DTA** etc.

Not more than 8 characters may be used in the description in front of .DTA.

PROJECT$ is the name of the project eg "VAAL RIVER STUDY". Do not use commas or colons as delimitators - rather use a hyphen as in this sentence.

If an error message occurs when running the program, the most probable causes are :-

- Too many or too few commas, or a comma after the last value on a line.
- The use of the decimal comma instead of the decimal point.
- Use of lower case **met**, **imp** or **dta** where capitals are required.
- The omission of an item where one was expected. Type -1 where the specific information is not known or not required.
- Search for the error in the data previous to the point indicated in the error message.

You can now use the program REGFLOOD to analyse the data for the sites that have been included in this file.

13.7 RECOMMENDED METHODS

The following methods are recommended for flood risk analysis in South Africa. Details of the implementation of these methods are described later. No significance should be attached to the order in which the methods are listed.

13.7.1 METHOD 1 : Unit hydrograph method - original algorithms

Source University of the Witwatersrand Hydrological Research Unit Report No 1/72. (HRU 1/72)

Computer program **DETFLOOD**

A revised depth-duration-frequency diagram for point rainfall was published in HRU 2/78. The two associated publications for Namibia are HRU 2/80 on large area storms and HRU 13/81 on design flood determination.

This method can be applied manually using the graphs reproduced in the appendices to this chapter, or by using the computer program DETFLOOD. The results will not be identical as curve fitting procedures had to be used to convert some of the graphs into computer algorithms. These differences are not meaningful.

13.7.2 METHOD 2 : Unit hydrograph method - alternative algorithms

Source This handbook.

Computer program **DETFLOOD**

The basic methodology is the same as that in HRU 1/72, except that alternative algorithms are used for the rainfall depth-area-duration-frequency (DADF) relationship, and effective rainfall.

This method can be applied manually but the computer method is preferred due to the interpolation required for determining short duration point rainfall.

13.7.3 METHOD 3 : Probable maximum flood - original algorithms

Source HRU 1/72

Computer program **DETFLOOD**

Both the manual and the computer versions follow the HRU 1/72 methodology except that the computer program does not (as yet) make provision for subdivision of the catchment and routing procedures for the resultant hydrographs. The program uses the upper envelope of maximum rainfalls in South Africa shown in Fig 13.6, but the user is given the option of entering alternative values.

13.7.4 METHOD 4 : Rational method - DWA algorithms

Source Department of Water Affairs (DWA) calculation sheet which is reproduced in the Annexure 13.B.

Computer program (not included)

This method is flexible in that the user can use his own DADF relationships, but the method for determining the C-coefficient is fixed.

This method is not included in the computer programs.

13.7.5 METHOD 5 : Rational method - alternative algorithms

Source This handbook.

Computer program **DETFLOOD**

This is a simplified version of the DWA method. The coefficients are the same, but the catchment is treated as a whole and not divided into subcatchments. The runoff coefficient C_T can be adjusted to permit calibration of the method against the results of the direct statistical analyses.

Two computer versions are provided :-

(a) DADF algorithms from the original HRU 1/72 method (method 1 above)

(b) DADF algorithms compatible with the alternative implementation of the HRU 1/72 method (method 2 above).
The inclusion of a calibration algorithm.

13.7.6 METHOD 6 : Regional maximum flood

Source Department of Water Affairs technical report TR 137 (Kovàcs, 1989).

Computer program **DETFLOOD**

The method is based on the original Francou-Rodier equation where the expected maximum flood is a function of the area of the catchment and a factor *K* which the user derives from a map on which the regional *K*-values boundaries are shown (Fig 13.8).

Manual analysis is very simple, but it has also been included in the suite of computer programs.

13.7.7 METHOD 7 : Direct statistical analysis - single station

Source This handbook.

Computer program **STFLOOD**

Manual and computer methods are provided for the following distributions. The computer output is in the form of tables with screen dump options.

Distribution		Moment estimators
N/MM	Normal	Conventional moments
LN/MM	Log normal	Conventional moments
LP3/MM	Log Pearson 3	Conventional moments
LGEV/MM	Log general extreme value	Conventional moments

13.7.8 METHOD 8 : Regional direct statistical analysis

Source This handbook.

Computer programs **REGDATA** and **REGFLOOD**

Only computer implementations are practicable for regional direct statistical analyses.

The following distributions are included. The output is in the form of tables with screen and plotter graphics options.

Distribution		Moment estimators
	Untransformed data :	
EV1/MM	Extreme value Type 1	conventional
GEV/MM	General extreme value	conventional
GEV/PWM	General extreme value	probability weighted
WAK/PWM	Wakeby	probability weighted
	Log(10) transformed data :	
LN/MM	Log normal	conventional
LP3/MM	Log Pearson Type 3	conventional
LEV1/MM	Log EV1	conventional

The program REGFLOOD includes the new GEV/PWM and WAK/PWM distributions which were specifically developed for regional statistical analyses. The graphical presentations include linear vertical scales for those distributions which are applied to the untransformed data.

Two versions are provided - one is a compiled version using Borland's TurboBasic compiler that can be run directly, and the other is an ASCII file that can be loaded and run from TurboBasic and possibly other BASIC compilers.

It is not necessary to install TurboBasic to run the compiled version as it will run on its own. If changes in the program are required, the ASCII version can be changed and a new compiled version produced using a suitable BASIC compiler.

Program input
The program makes use of sequential data files created by the program REGDATA details of which are given in the previous section.

Program output
The program output is in the form of tables of the available stations, flow data for each station, individual and regional statistical properties, and estimated maxima for a range of return periods for each site.

13.8 CALCULATION PROCEDURES

13.8.1 METHODS 1 TO 3 : UNIT HYDROGRAPH METHODS

The calculation procedure is the same for all three of these methods. Only the algorithms differ.

The computer versions use equations derived from the HRU diagrams so it is not necessary to read information from the diagrams. There will be small discrepancies between the hand and computer calculations.

Input

Generalized veld type zone	
Area of catchment	
Length of longest watercourse	
Height difference along equal area slope	
Distance to centroid of catchment	
Mean annual precipitation	[method 1 only]
Whether coastal or interior rainfall region	[method 1 only]
Rainfall data from TR 102	[method 2 only]

Calculation procedure

Calculate basin lag T_l

This index represents the weighted area of the catchment, by taking both shape and slope into account, and has the dimensions of area.

Derive S-curve

For half-hour increments of time T measured in hours determine the corresponding values of T/T_l and interpolate the unit discharge values Q/Q_p for the given veld type zone. The tabulated values of T/T_l vs Q/Q_p are from Table 13.3.

Note that by definition the S-curve must be the cumulative sum of the hourly spaced Q/Q_p values. As half-hour values are calculated, alternate values in the column are summed separately to derive the S-curve.

Calculate catchment precipitation

Use the equations on the coaxial diagram Fig 13.2 rather than reading the values off the graph. The answers may be a little different due to the curve fitting procedures used to derive the equations, but the results will be more consistent because of the coarseness of the diagram.

###

Alternative algorithm for catchment precipitation

The alternative algorithm is a more accurate estimate of catchment precipitation for storm durations of four hours or less and requires the following alternative input:

Two-year return period maximum one-day precipitation from Fig 13.5.

Mean lightning ground flash density (flashes/km²/year) from Fig 13.3.

For longer durations see the alternative algorithm used in the rational method.

###

Main calculation loop
For the required return period calculate a complete hydrograph for each duration. Increment the duration in selected steps of multiples of an hour until the duration which produces the highest peak is identified, ie proceed with the calculation until the maximum peak for that duration is less than that for the previous duration. The design hydrograph (or design peak if only the peak is of interest) is therefore that associated with the second-last duration.

The program allows the user to set the initial duration as well as the duration steps, both in multiples of one hour. This is useful for large catchments but must be used with caution because the program will assume that the initial duration is the critical duration if the initial duration exceeds the critical value. If this happens RUN the program again from the start. Note also that the peak may be missed if the chosen increments are too large.

Similarly the discharge *vs* time increments of the hydrographs may be set at either half an hour or one hour, the former being useful for small catchments.

For the first run while the output is to the screen only, a useful combination would be an initial duration of about half the calculated basin lag; hourly duration steps; and hourly flow vs time increments.

Calculation for each duration

Calculate storm precipitation (mm)

The program uses equations derived from Fig 13.2.

Calculate area reduction factor (%)
For catchment areas less than 500 km² use Fig 13.7(a) and for larger areas use Fig 13.7(b). The program uses equations derived from these figures.

##

Alternative precipitation and ARF calculations

Alternative algorithms are those used in the rational method.
Calibration algorithms are included in the alternative implementation.

##

Calculate catchment precipitation
The ARF determined in one of the above methods is the percentage of the point rainfall that is assumed to be uniformly distributed over the catchment.

Calculate effective precipitation
The percentage storm losses are a function of the area of the catchment and the veld zone. Use the equations shown on Fig 13.9 for the PMF, and Fig 13.10 for the return periods:

Determine the runoff factor
(a) for the PMF,or
(b) for return periods where

Zone	a	b
Zone 2	66.75	0.06
Zones 4 to 7	2.35	0.60
Zones 1, 3, 8, and 9	1.45	0.60

Then the effective precipitation which is the depth of precipitation that reaches the river system is calculated.

Calculate time vs flow hydrograph
The peak flow for the particular hydrograph is determined and the value noted.

After the full hydrograph has been determined the peak for this duration is compared with that for the previous duration. If it is greater than that for the previous duration then the duration is incremented by the previously selected increment and another hydrograph calculated.

The process is continued until the peak is less than the previous peak. The previous peak and its associated hydrograph are then the design peak and hydrograph for the specified return period.

13.8.2 METHODS 4 AND 5 : RATIONAL METHOD

The calculation procedure is the same for both of these versions. Only the algorithms differ.

Input

Area of catchment	
Length of longest watercourse	
Slope of longest watercourse (1085 method)	
Catchment coefficients	
Mean annual precipitation	[method 4 only]
Whether coastal or interior rainfall region	[method 4 only]
Rainfall data from TR 102	[method 5 only]

Method

After more than a century since it was first proposed this universally applied method is the oldest of the calculation methods presented in this chapter. It depends on the simple linear relationship:

$$Q_T = 0{,}278\ C_T I_T A$$

where C_T is the runoff coefficient with a value between zero and one, I_T is the rainfall intensity averaged over the catchment in mm/h for the return period T and A is the area of the catchment in km^2. The factor 0.278 is the conversion factor for the units used.

The essence of the method is the estimation of the values of the rainfall intensity and the runoff coefficient.

Two alternative methods are given for the determination of effective catchment rainfall. The first is the HRU 1/72 method (revised in HRU 2/78) which is detailed in method 1 above.

The alternative implementation is based on the recalibrated Hershfield equation for storm durations up to 6 hours, and the Department of Water Affairs' technical report TR 102 for durations from 1 to 7 days. In the program, the point precipitation for durations of 1, 2, 4, and 6 hours is determined using the Hershfield equation which requires values for the return period, duration, mean lightning ground flash density from Fig 13.3 and 2-year return period one day rainfall from TR 102 . Data for 1, 2, 3 and 7-day rainfall depths for a range of return periods are read from TR 102. This information is then presented in the form of a table. Because two different methods are used to derive these values, the 6-hour value (or more rarely the 4-hour or 2-hour values) may be higher than the 24-hour value. If the time of concentration is less than 24 hours and the Hershfield value is larger than the 24-hour value, then it is reduced to equal the 24-hour value. This assumption is realistic as the storm precipitation mechanisms are such that short duration rainfalls exceeding 4 hours are often close to the 24-hour value.

The program uses linear interpolation between the values derived above to estimate the point precipitation for the previously calculated time of concentration.

Results obtained when applying the commonly used methods for deriving the area reduction factor do not vary greatly. The method given in the alternative implementation is new and is based on the graphical relationship proposed by Alexander (Fig 13.6) which in turn was based on that given in the UK Flood Studies Report. The equation is included in the program.

The estimation of the runoff coefficient is based on the Department of Water Affairs' standard calculation sheet which itself was evolved from earlier proposals by Alexander, Kovàcs and others. The relationships used are logical but their values are empirical and based largely on experience.

Calculation procedure

Calculate the 1085 slope

Calculate the time of concentration

A warning is issued if the calculated time of concentration is less than one hour or greater than 7 days. The accuracy of the equations should be investigated at these extremes.

Estimate catchment characteristics
These are listed in the data sheets in the Annexures.

Calculate catchment rainfall : DWA implementation
The catchment precipitation calculation method is that given in HRU 1/72 and HRU 2/78, and is as follows:-

The program uses equations derived from Fig 13.2.

The program reads the factor FF associated with this return period.

Calculate area reduction factor.
The equations used in the program were derived from the graphs Figs 13.5(a) and 13.5(b) respectively:

Calculate catchment precipitation

##

Alternative implementation

The alternative algorithms make use of the TR 102 rainfall data described above. Having calculated the time of concentration, the corresponding point precipitation is determined by linear interpolation.

Calibration algorithms have been included in the alternative implementation.

##

Common to both implementations

Calculate value of the net runoff coefficient
The net runoff coefficient is a function of the sum of the coefficients for the three catchment characteristics and the return period.

Interpretation of the results
The estimation of the runoff coefficient is somewhat subjective. As the relationship is linear it is a simple matter to adjust the runoff coefficient in the light of special circumstances which may be present in the catchment being examined, and recalculate the peak flow manually.

13.8.3 METHOD 6 : REGIONAL MAXIMUM FLOOD

Calculation procedure

Measure the area of the catchment

Estimate the value of the K-factor
Identify the region in which the site is located (Fig 13.4), and determine the value of the K-factor.

Note that the regions on the map refer to the location of the site and not the catchment. Only if the site is located near a boundary will it be necessary to consider adjusting the K-factor.

The equation has the form:

$$Q_{max} = a \text{ Area}^b$$

where a and b are functions of the Francou-Rodier K-factor and are read from the table below :

Calculate the RMF using the appropriate equation in the Table below.

RMF equations for the eight maximum flood peak regions in southern Africa from Kovács (1988b)

Region	Number of floods	Transition zone		Flood zone	
		Area range km^2	RMF m^3/s	Area range km^2	RMF m^3/s
5,6	25	1-100	100 A ^{0,68}	100 - 10 000	302 A ^{0,44}
5,4	34	1-100	100 A ^{0,62}	100 - 20 000	209 A ^{0,46}
5,2	61	1-100	100 A ^{0,56}	100 - 30 000	145 A ^{0,48}
5,0	155	1-100	100 A ^{0,50}	100 - 100 000	100 A ^{0,50}
4,6	55	1-100	100 A ^{0,38}	100 - 100 000	47,9 A ^{0,54}
4,0	26	1-300	70 A ^{0,34}	300 - 300 000	15,9 A ^{0,60}
3,4	12	1-300	50 A ^{0,265}	300 - 500 000	5,25 A ^{0,66}
2,8	6	1-500	30 A ^{0,262}	500 - 500 000	1,74 A ^{0,72}

13.8.4 METHOD 7 : SINGLE STATION DIRECT STATISTICAL ANALYSIS

MANUAL CALCULATION

All distributions
Determine the values of the mean, standard deviation and skewness coefficient of the raw data as well as the $\log_{10}$ transformed data.

Log normal distribution (LN/MM)
Read the value of W_T for the required return period from the third column in Table G1 in the appendices. Then :

$$Q_T = \text{antilog}\ (\text{mean}_{\text{logs}} + W_T.\ \text{standard deviation}_{\text{logs}})$$

Confidence bands
The displacement of the two-sided 95% confidence band about the estimated value can be read from column 5 of Table G1 where N is the number of observations:

The 95% confidence limits are :

$$Q_{T,95} = \text{antilog}\ [\ \text{mean}_{\text{logs}} + \text{standard deviation}_{\text{logs}}\ (W_T \pm \text{displacement}\ )\ ]$$

Log Pearson Type 3 distribution (LP3/MM)
From the right hand panel of Table G1 determine the value of W_T for the known skewness coefficient of the log-transformed data by linear interpolation. Thereafter the calculation is the same as for the normal distribution. No method is presented for the calculation of the confidence bands for the LP3 distribution.

Log extreme value Type 1 (log Gumbel) distribution (LEV1/MM)
From the central column of Table G2 (g = 1,14) read the value of W_T for the required return period. Then calculate Q_T directly :

$$Q_T = \text{antilog}\ [\text{mean}_{\text{logs}} + \text{standard deviation}_{\text{logs}}\ (\ 0{,}780\ .\ W_T\ -\ 0{,}450\ )]$$

Extreme value distributions using raw data
Calculate Q_T using the moments of the raw data in the EV1/MM and GEV/MM distributions.

Extreme value Type 1 distribution (EV1/MM)
The procedure is the same as for the log Gumbel above except that the statistics of the untransformed data are used :

$$Q_T = \text{mean} + \text{standard deviation}\ (\ 0{,}780\ .\ W_T\ -\ 0{,}450\ )$$

General extreme value distribution (GEV/MM)

For the known value of the skewness coefficient g read the value of W_T from Table G2 and the values of k, E(y) and var(y) from Table G3 by linear interpolation.

$$QT = \text{mean} + \left(\text{standard deviation}^2 / \text{var}(y) \right)^{0,5} \left(1 - E(y) \text{ [illegible] } k \cdot W_T \right)$$

COMPUTER CALCULATION

Program modules

The program STFLOOD has three modules:

(a) Calculation of statistical properties of the data set (required for all subsequent modules). Data can be read from within the program, or entered from the keyboard.

(b) Calculation of the flood peak-frequency relationships, and the presentation of the output in the form of tables.

(c) Present the results in the form of graphs on the screen which can also be dumped on a dot matrix printer. Statistical parameters entered from the keyboard will not be plotted.

Input

The following input options are provided:

(a) A complete data set of observed annual maxima either from the keyboard or by incorporating the values in the program.

(b) The statistical parameters (mean, standard deviation and skewness coefficient) of the untransformed data or log(10)-transformed data.

Output

The expected maxima for return periods in the range of 2 to 10 000 years, for the following frequency distributions:-

(a) Untransformed data.
Normal, Extreme Value Type 1 and General Extreme Value.

(b) Log(10)-transformed data.
Log-Normal, Log-Gumbel, Log-Pearson Type 3 and Log-General Extreme Value.

The output can be sent to the screen only. Use the PRINT SCREEN command to print tables or graphics on the printer. The recommended procedure is to use the screen only for the first run and then repeat the calculation with the output sent to the printer in the form of screen dumps. Use REGFLOOD for customised print-outs and higher resolution graphics.

Calculation procedure

Calculate statistical parameters
(This is bypassed if parameters are entered from the keyboard).

The method of moments is used to determine the values of the three statistical parameters of interest. Alternative algorithms using the probability weighted moments method are included in the regional analysis program REGFLOOD.

Rank the data
The data are ranked from largest to smallest. The year of occurrence is attached to each value and then separated from it again after ranking.

The original as well as the ranked data sets are tabulated followed by tables of the statistical properties of the raw and log-transformed data.

General approach
The nine return periods (T) of hydrological interest included in the program are 2, 10, 20, 50, 100, 200, 500, 1000, and 10 000 years. The reliability of the results for long return periods is questionable but the 10 000 year return period is included because of its possible use as an alternative for the PMF estimate provided by the deterministic methods.

13.8.5 METHOD 8 : REGIONAL DIRECT STATISTICAL ANALYSIS

Emergency exit

If you make an irrecoverable entry or get tied up at any point in the program continue to the next *Press any key ----* message. Press the *S*-key, and run the program again from the start. If all else fails, press the reset button !

Go through the following steps when running this program. The first round of calculations will be based on the moments of the individual stations after which go through the second round where the results are based on the regionally weighted moments.

1. Read and note the first two pages of information presented on the screen.

2. All the files on the disk with the subscript .DTA will be listed. Select and type in the name of the required data file including the suffix .DTA. An error trap will prevent you from accidentally loading a non-data file. If the file is not found the program aborts and has to be run again.

3. The data on the selected file will be loaded into memory.

4. It is not necessary to select the printer options at this stage. You will be returned to this menu later when you have had a chance to select the stations that are to be included in the final analysis.

5. The available stations contained in the data file that you have selected will be listed on the screen. You will be asked how many of these stations are to be included in the regional analysis, and to list the menu numbers of the selected stations in ascending order. There is an error trap that will prevent you from duplicating a station. If you get tied up at this point continue until the next opportunity is offered to exit from the program by pressing the S key.

6. Select the Wakeby option.

7. The program will run through each of the selected stations in turn. For each station you will be presented with a number of options. The data for the first selected station will be ranked and listed on the screen.

8. The time span covered by the station record and the numbers of missing years and years with zero flow are presented. You will be returned to this screen after you have studied the graphs and decided how many low outliers should be omitted. Reply no to further adjustment at this stage.

9. Details of the data that will be used for determining the historically weighted moments and conditional probability calculations are shown.

10. The statistics of the data for this station are listed.

11. The Wakeby procedure begins. The fits for all the distributions other than the Wakeby distribution are unique and require no intervention on the part of the user. However, there is a range of combinations of the five parameters of the Wakeby distribution that can be fitted to the data set. You will be guided through the selection process by the screen instructions.

    The parameter estimation procedure for the Wakeby distribution consists of four successive stages (see Annexure 2F in Chapter 2). If no successful combination is obtained by the end of the fourth stage, it is assumed that no WAK/PWM distribution exists for the data set.

    In the first stage it is assumed that $m=0$ and the unique set of parameter values is sought. If the value of b is less than 0,3 the appropriate error message will be shown on the screen, (this is commonly the case), otherwise other error conditions will be tested.

    The second stage is similar to the first stage except that m need no longer be zero.

    The last two stages require that the user specify the range of values of b to be investigated, and you are prompted to specify the maximum (say 50) and minimum (say 0.3). The program calculates the incremental values.

    The remaining parameter values are uniquely related to b. These are determined for each of the values of b and the combinations tested. If the combination fails, the error condition is listed, otherwise the parameter values are listed together with the calculated values of Q_{10}, Q_{100} and Q_{1000}. You now have the option of selecting a valid set, carrying out another cycle with another range of b_{max} and b_{min}, or proceeding to the next step. The procedure recommended by the authors of the method automatically selects the highest acceptable value of b, but this program gives the user the option of exploring the effect of any successful combination. Select the highest valid value of b until you have become familiar with the method and the underlying theory and assumptions. Otherwise stay within the Wakeby loop until you are satisfied with the value of b that you have selected. The criterion for preferring an alternative value of b would be that it provides a better visual fit on the graph.

12. Once a successful combination of Wakeby parameter values has been identified the set of values will be displayed on the screen, followed by the parameter values for the extreme value and LP3 distributions.

13. The estimated maxima are presented for a range of return periods for all the distributions analysed.

14. Further options menu. You will be returned to this menu several times during the analysis. Note the omission of some options and inclusion of others as the calculation progresses. At this stage select the graphics on screen option.

15. Accept the default combination which should work for all screens with graphics cards. Later on you can try other SCREEN numbers to produce higher resolution graphics, and experiment with the other options to improve the presentation. Note the combinations which best suit your requirements.

16. There are nine graph options. Option number 8 is the most useful for a general overview. If you only get garbage on the screen then you probably selected the wrong option at step 15 or you do not have a graphics card installed.

17. Note the distribution code at the bottom of the screen. Press the space bar for the next fit on the same screen. When the asterisk appears press the PRINT SCREEN key. If you get garbage on the printer then you either forgot to load the graphics command before starting the program, or your printer does not recognise the standard graphics commands.

18. After each graph you will be returned to the menu in step 16.

19. Cycle through the different graphs. Note the visual goodness of fit of the various distributions. Also make a note of the anomalous low outliers that should be excluded from the data when the final analyses are undertaken.

20. When finished, select the option that will return you to the further options menu at step 14.

21. Select option 4 to start the analysis for the next station. You will be taken back to step 6. The calculation for the next station is the same as in steps 6 to 20 above. Repeat the cycle for each station that was selected in step 5.

22. After you have examined the graphs for the last station and are returned to the further options menu select "Determine regional relationships."

23. A table giving the statistics for each station and the regional average is presented. Look for anomalies in the coefficients of variation and skewness and try to reconcile them with the graphs for the stations. Remember that these values will change in the next round of calculations when low outliers are omitted.

24. The parameters and moments for each station are listed in a table headed REGIONAL PARAMETERS. The average values in the bottom line are the weighted averages where the weights are directly proportional to the record lengths of the stations used to derive them.

    This is followed by a table of these regionally weighted moments. Note that it is the moments that are regionalized and not the distribution parameters.

A further weighting takes place for each station. This follows the USA guidelines procedure where for stations with less than 25 years of data the regional moments are used and for more than 100 years of data the station moments alone are used. For record lengths from 25 to 100 years the regional weighting is proportionally adjusted.

For the LP3/MM distribution the regionally weighted skewness coefficient is used directly.

For the GEV/PWM distribution the weighted moments for each station are determined, from which the parameter values a, u, and k are calculated.

In the case of the WAK/PWM distribution the weighted moments for each station are determined. This is followed by a repeat of the fitting process to calculate the parameter values for a, b, c, d and m.

The weighting procedures can be changed by altering the algorithms in the program. All changes should be detailed in the program.

25. Another full cycle for each station is commenced, but this time regionally weighted station moments will be used in the calculations. Only the LP3/MM, GEV/PWM and WAK/PWM distributions are used for the regional analyses.

26. When you have completed this cycle the further options menu in step 14 will be reduced to selections 6 and 7 for further calculations or exit from the program respectively. Select further calculations as you will have completed the exploratory calculations and now have to undertake the final analyses.

27. Go through the whole process again but this time omit the anomalous stations and exclude the low outliers that were noted during the first round of calculations. Add historical information if available.

13.9 INTERPRETATION OF RESULTS - ALL METHODS

This is the most important part of any flood frequency analysis. No two methods in this handbook and suite of programs will give the same answers. The user will have to decide on the basis of his own experience and judgment which values are most appropriate to the problem at hand. The computer programs have been designed with this in mind. The results of all intermediate calculations are displayed on the screen or printer, and it is a simple matter to test the sensitivity of the final result to the various input data by running the programs with different input values. The user can see the calculations in progress and therefore develop a much better appreciation of the effect of the different algorithms on the results. Visual examination of the plots on the graphs is by far the best means of determining which distribution best fits the data, as well as indicating the presence of outliers or anomalies in the data and their effect on the calculations.

Experienced judgment is required for the interpretation of the results of all methods. In the case of the regional statistical analysis methods specific decisions that have to be made include :

1. Which of the regional stations are relevant to the analysis and which should be omitted?

2. How many low outliers should be omitted from each station's data ?

3. Which distributions perform the best for the region ?

4. Should the results produced by one of the distributions be accepted, or the most severe results, or an average of the results from more than one distribution? Is it rational to assume that one distribution is better for some stations within a region and another distribution for the remaining stations?

Where direct statistical analyses are carried out it would be instructive if the deterministic methods were also applied at the site and the results plotted on the statistical analysis plot.

13.10 LIST OF ANNEXURES, FIGURES AND TABLES

LIST OF ANNEXURES

LIST OF FIGURES

LIST OF TABLES

CALCULATION

SHEETS

ANNEXURE 13A

13.10.1 DATA SHEET FOR THE DETERMINISTIC METHODS

Name of river :
Site :
Your name :
Today's date :

Catchment characteristics:
Area of catchment (km²) :
Length of longest watercourse (km) :
Height difference along equal area slope (m) :
Height difference along 1085 slope (m) :
Distance to catchment centroid (km) :

Unit hydrograph method data:
Veld type zone (from Fig. 13.1) :
Catchment MAP from HRU quaternary zone data ..:
Coastal (1) or inland (2) region :

Rational method catchment coefficient categories (indicate which category number for each of the three criteria)						
Category	Catchment slope		Permeability		Vegetal cover	
1	< 3%		very permeable		dense	
2	3% - 10%		permeable		cultivated	
3	10% - 30%		semi-permeable		grassland	
4	> 30%		impermeable		bare	

Lightning ground flash density (Fig13.3) :
RMF *K*-value (Fig 13.4) :

Rainfall data from DWA publication TR102 :

Weather Bureau station number :
(write "average" if more than one station)
Location :
Mean annual rainfall (TR102) :
One-day 2 year RP rainfall from Fig 13.5 :
Enter the TR102 values in the table below. The program will adjust these in the ratio of the catchment MAP to the TR102 MAP.

DURATION	RETURN PERIOD (YEARS)						
	2	5	10	20	50	100	200
1 day							
2 days							
3 days							
7 days							

Probable maximum precipitation.
The program default values are from Fig 13.6 (RSA envelope). If other values are required enter them below and key them in when requested in the program. *Do not omit any values.*

Duration	(hours)	0,25	0,50	1	2	4	8	12
PMP	(mm)							

Duration	(days)	1	2	3	7
PMP	(mm)				

Comment:
Catchment topography:
Site RL .... Watershed RL .... Range .... :
Drainage channel density (dense, average, sparse) :
Drainage system shape (avr, elongated, squat, fan, parallel) :
Major upstream dams :
Approx number of farm dams / 10km² :

Information from HRU report series on RSA resources:
Quaternary catchment nrs :
Total area :
Total catchment MAP :
Total catchment MAR :
% runoff :
% afforestation :

Other comment:

13.10.2 CALCULATION SHEET FOR THE DWA RATIONAL METHOD

RATIONAL METHOD (DWA CALCULATION SHEET)

Site :
River :
Description :
Analysis by :
Date :

Topography and geology:		Area weighting factors:	
catchment area (km^2)		rural	
longest watercourse (km)		urban	
height difference (m)		lakes	
dolomitic area (%)		Total	1,00

PHYSICAL CHARACTERISTICS AS A PERCENTAGE OF THE AREA OF THE CATCHMENT

RURAL						URBAN	
Steepness		Permeability		Vegetation		Occupation	
< 1%		very permeable		forest		lawns, parks	
1 to 3%		permeable		dense bush		residential	
3 to 10%		semi-permeable		thin bush		industrial	
10 to 30%		impermeable		cultivated		downtown	
30 to 50%				grassland		streets	
> 50%				bare			
		Note: use very permeable in afforested areas					
Total	100	Total	100	Total	100	Total	100

RECOMMENDED VALUES OF RUNOFF COEFFICIENT

RURAL

Component	Category	MAP(mm) < 600	MAP(mm) 600-900	MAP(mm) >900
Steepness Cy =	< 3% 3 to 10% 10 to 30% 30 to 50% > 50%	0,01 0,06 0,12 0,22 0,26	0,03 0,08 0,16 0,26 0,30	0,05 0,11 0,20 0,30 0,34
Permeability of soil Cp =	very permeable permeable semipermeable impermeable	0,03 0,06 0,12 0,21	0,04 0,08 0,15 0,26	0,05 0,10 0,20 0,30
Vegetal cover Cv =	dense bush, forest cultivated land, scrub grassland bare surface	0,03 0,07 0,17 0,26	0,04 0,11 0,21 0,28	0,05 0,15 0,25 0,30
Interpolate between columns if MAP is between 570 and 630, or between 860 and 950mm.				

Continued :-

URBAN		
Occupation	**Runoff coefficient**	**Comment**
Lawns:		Use linear interpolation for intermediate values.
sandy, flat <2%	0,05 to 0,10	
sandy, steep >7%	0,15 to 0,20	
heavy soil, flat <2%	0,13 to 0,17	
heavy soil, steep >7%	0,25 to 0,35	
Residential:		
single dwelling area	0,30 to 0,50	
flats	0,50 to 0,70	
Industrial:		
light industries	0,50 to 0,80	
heavy industries	0,50 to 0,90	
Business:		
downtown	0,70 to 0,95	
neighbourhood	0,50 to 0,70	
Streets	0,70 to 0,95	

RETURN PERIOD ADJUSTMENT FACTOR TO BE APPLIED TO CALCULATED RUNOFF COEFFICIENT

RETURN PERIOD	RURAL	URBAN
T (years)	f_T	
2	0,50	For return periods equal to or greater than 50 years
5	0,55	
10	0,60	C_T = 1,00
20	0,67	
50	0,83	
100	1,00	
where $C_T = f_T(Cy + Cp + Cv)$		
For PMF $C_T = Cy + Cp(max) + Cv(max)$		

RAINFALL

Mean annual rainfall (MAP) =

Point rainfall for time of concentration t_c	Return period (years)						
	5	10	20	50	100	200	Max
Point rainfall depth (mm)							
Point intensity (mm/h)							
Area reduction factor							
Average intensity (mm/h) I_T =							

AREA WEIGHTED RUNOFF COEFFICIENT

Rural							
Urban							
Lakes							
Area weighted average C_T =							

PEAK DISCHARGE

$Q_T = 0,278\ C_T I_T A$							
Corrected peak							

13.10.3 LIST OF FIGURES

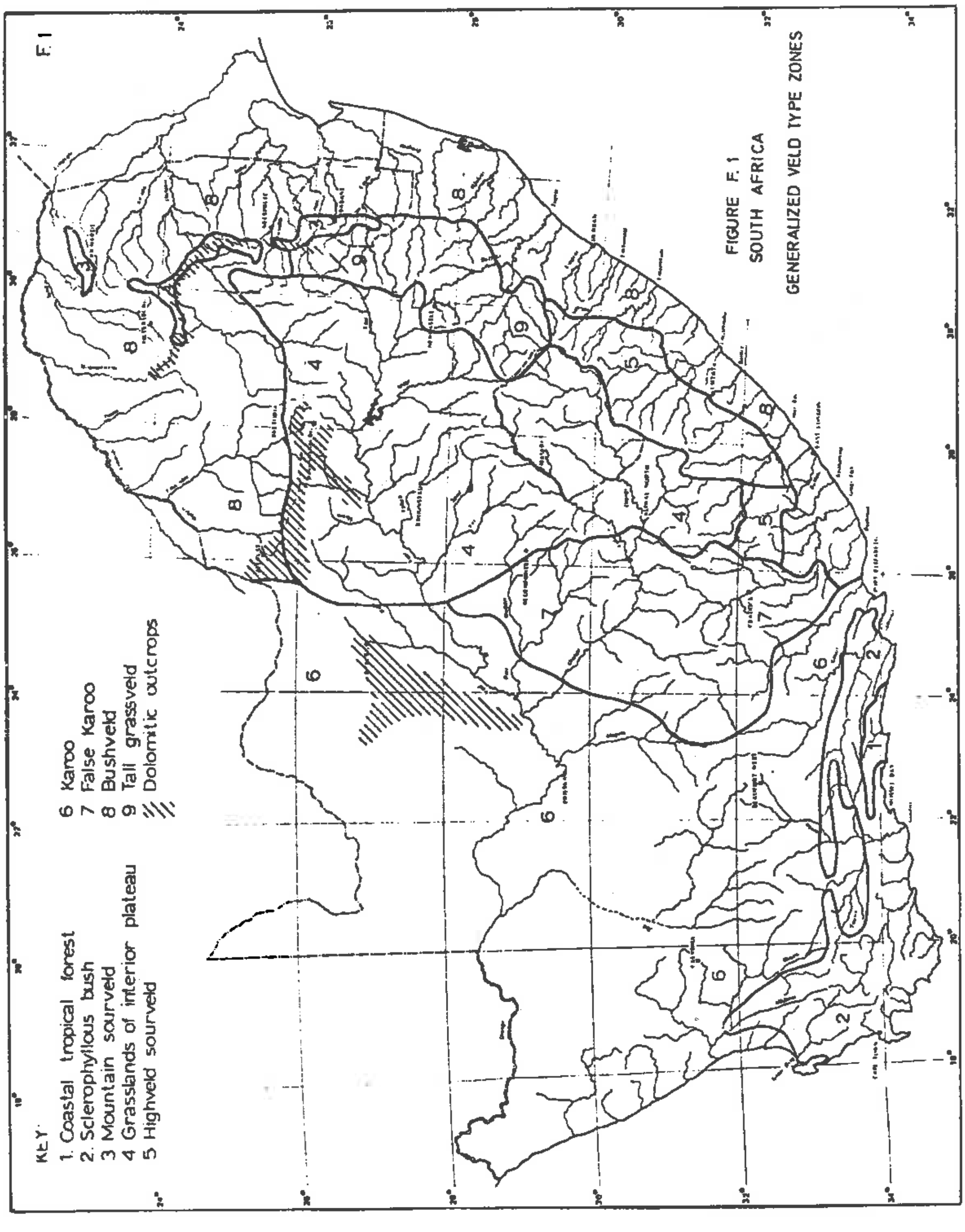


FIGURE 13.1 Generalized veld type zones from HRU 1/72 Fig F.1

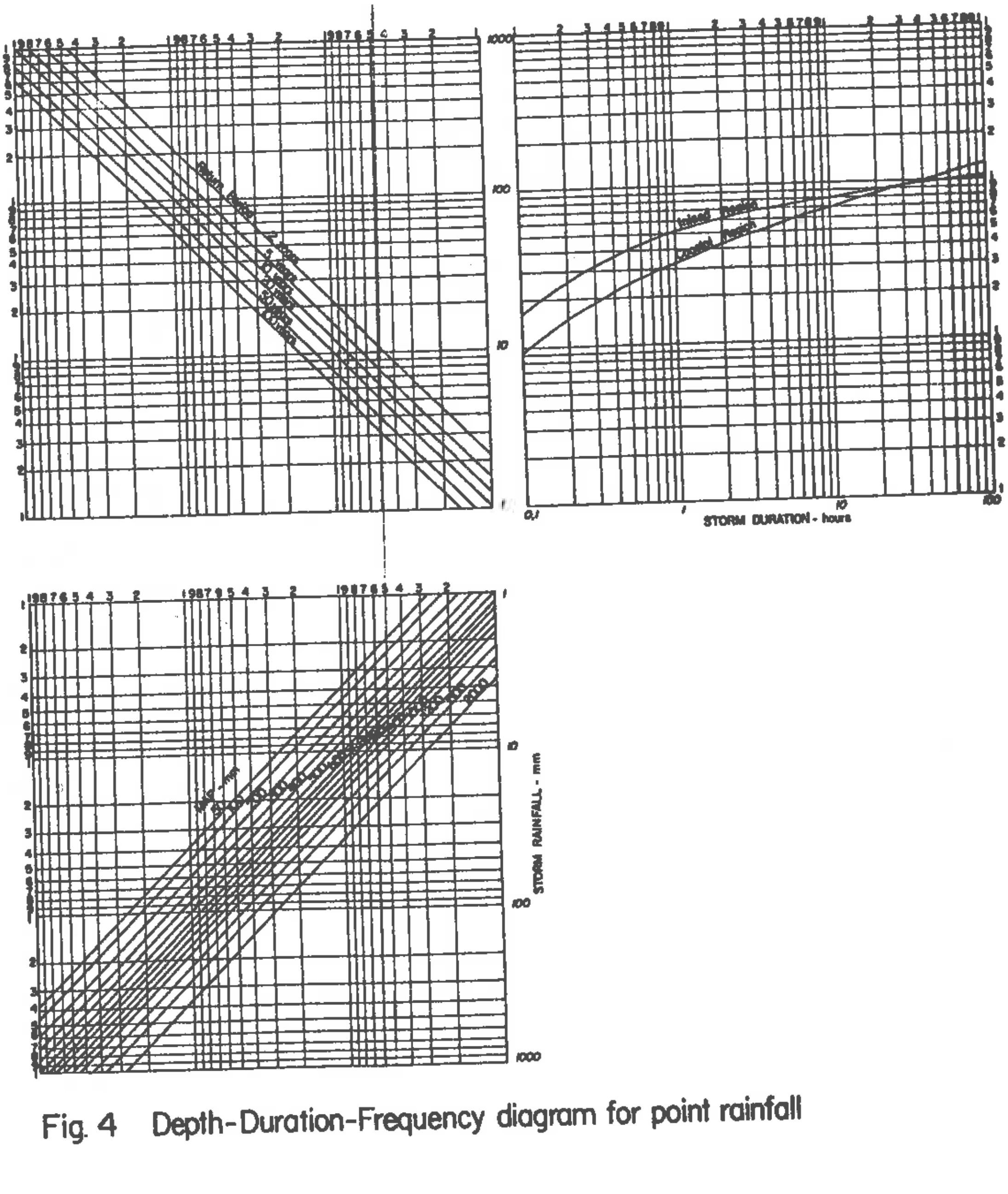


FIGURE 13.2 Depth-duration-frequency diagram for point rainfall from HRU 2/78 Fig 4

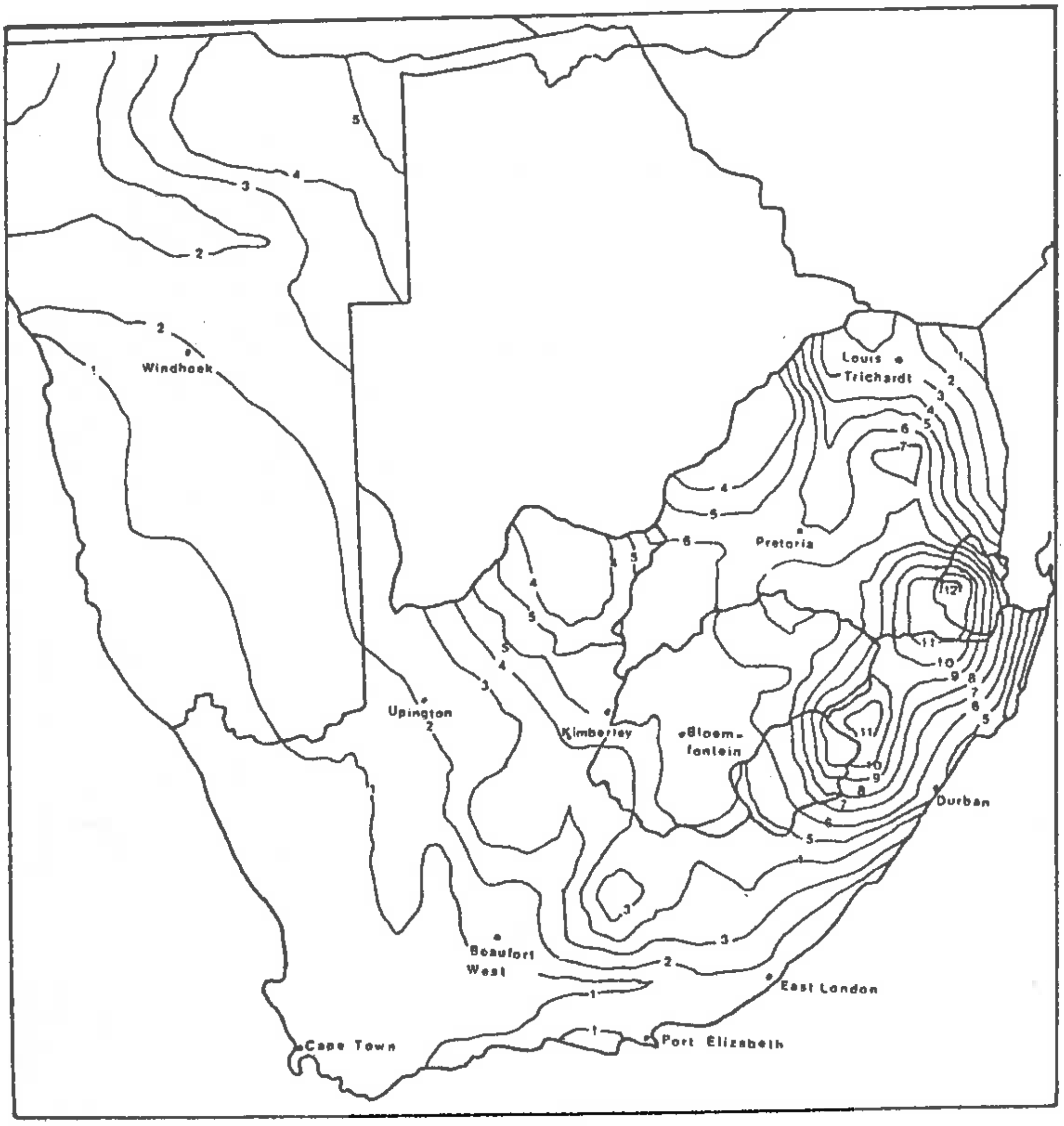


FIGURE 13.3 Mean annual lightning ground flash density from Kröninger *et al* CSIR

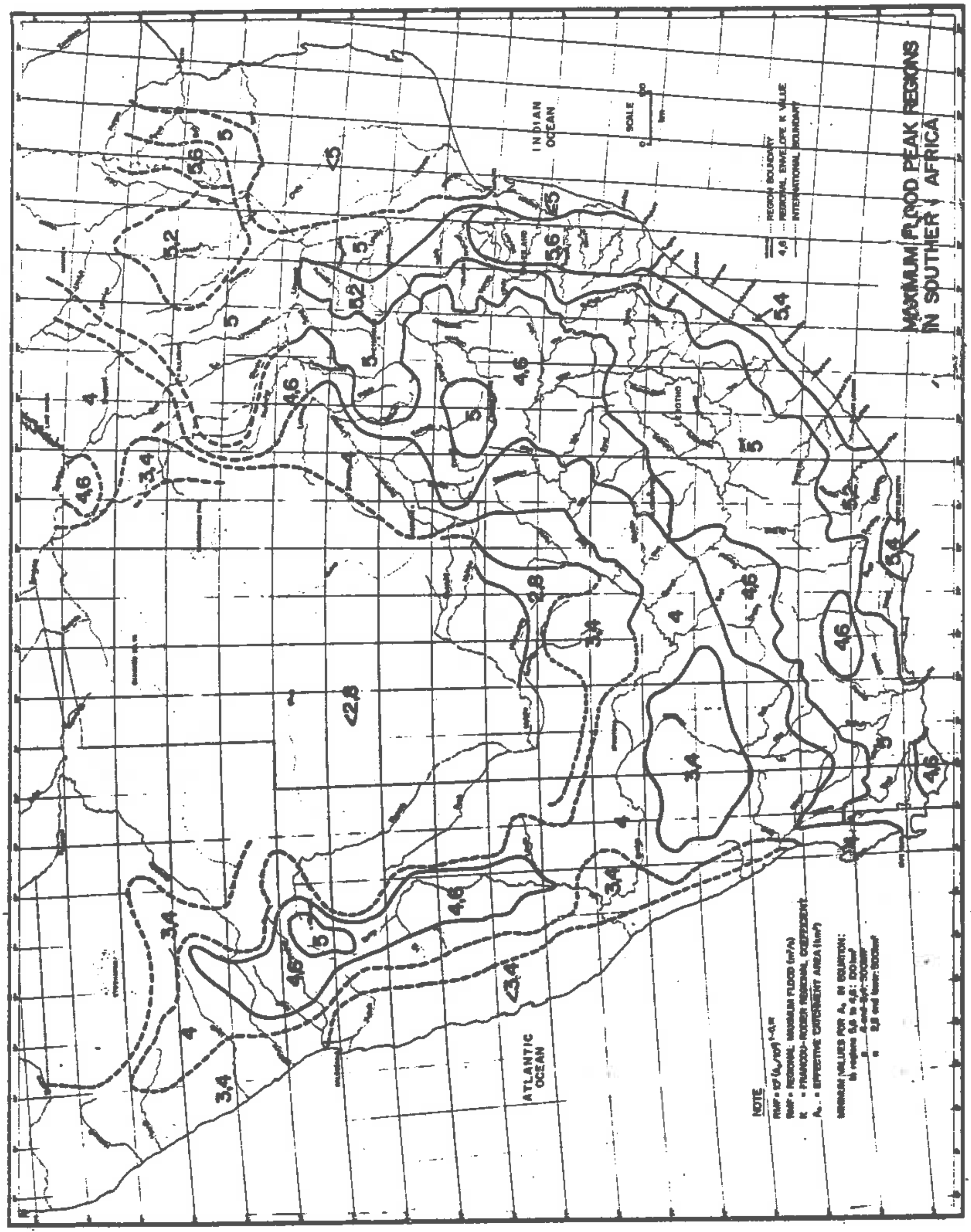


FIGURE 13.4 Maximum flood peak regions in southern Africa from Kovàcs 1989

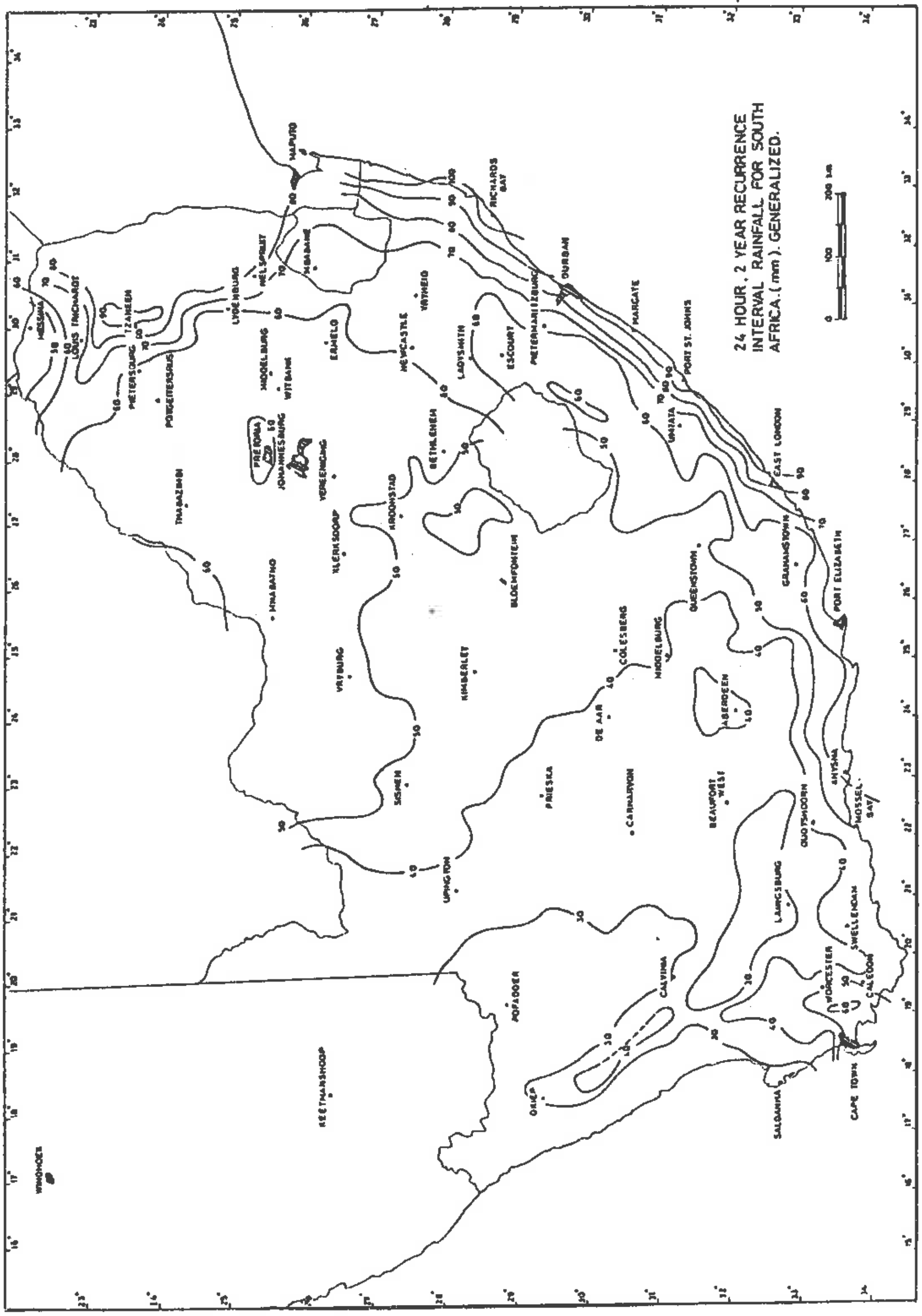


FIGURE 13.5 One-day 2 year return period rainfall for South Africa from TR 102

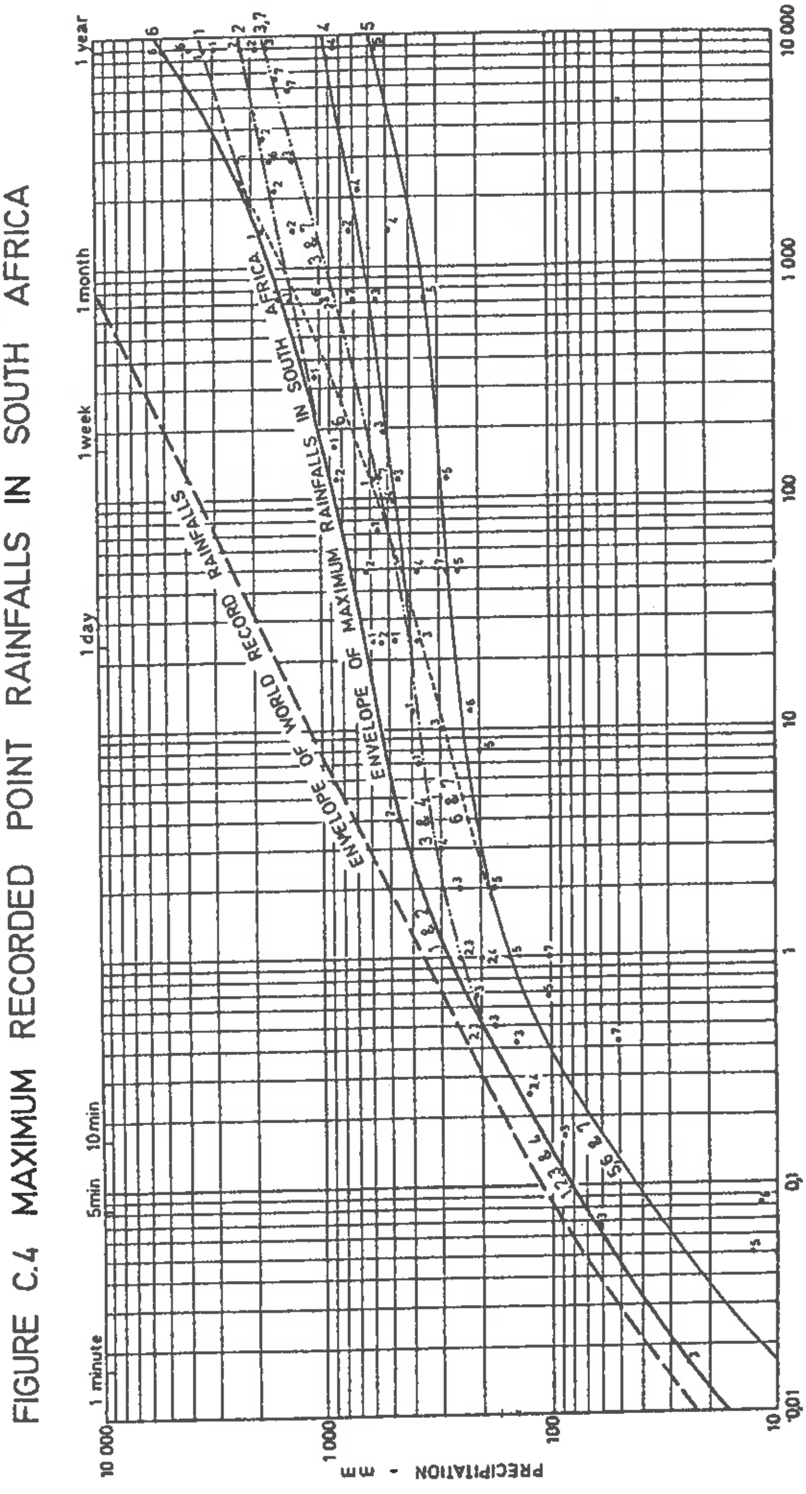
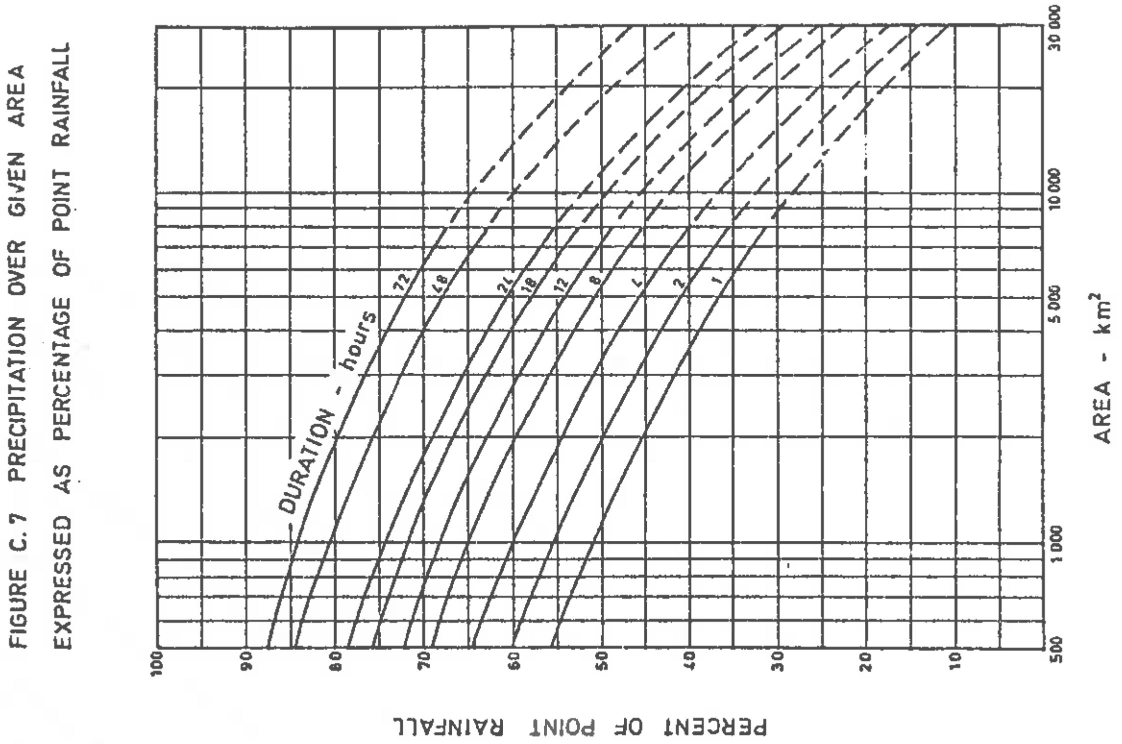


FIGURE 13.6 Maximum recorded point rainfalls in South Africa from HRU 1/72 Fig C.4



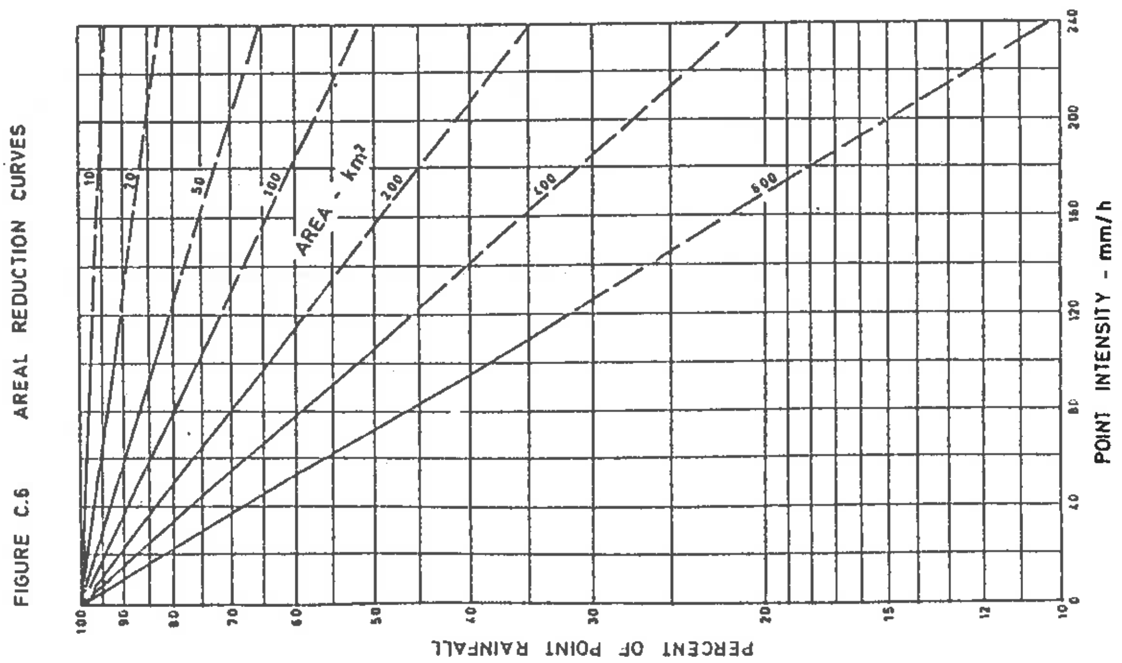


FIGURE 13.7 Area reduction curves from HRU 1/72 Figs C.6 and C.7

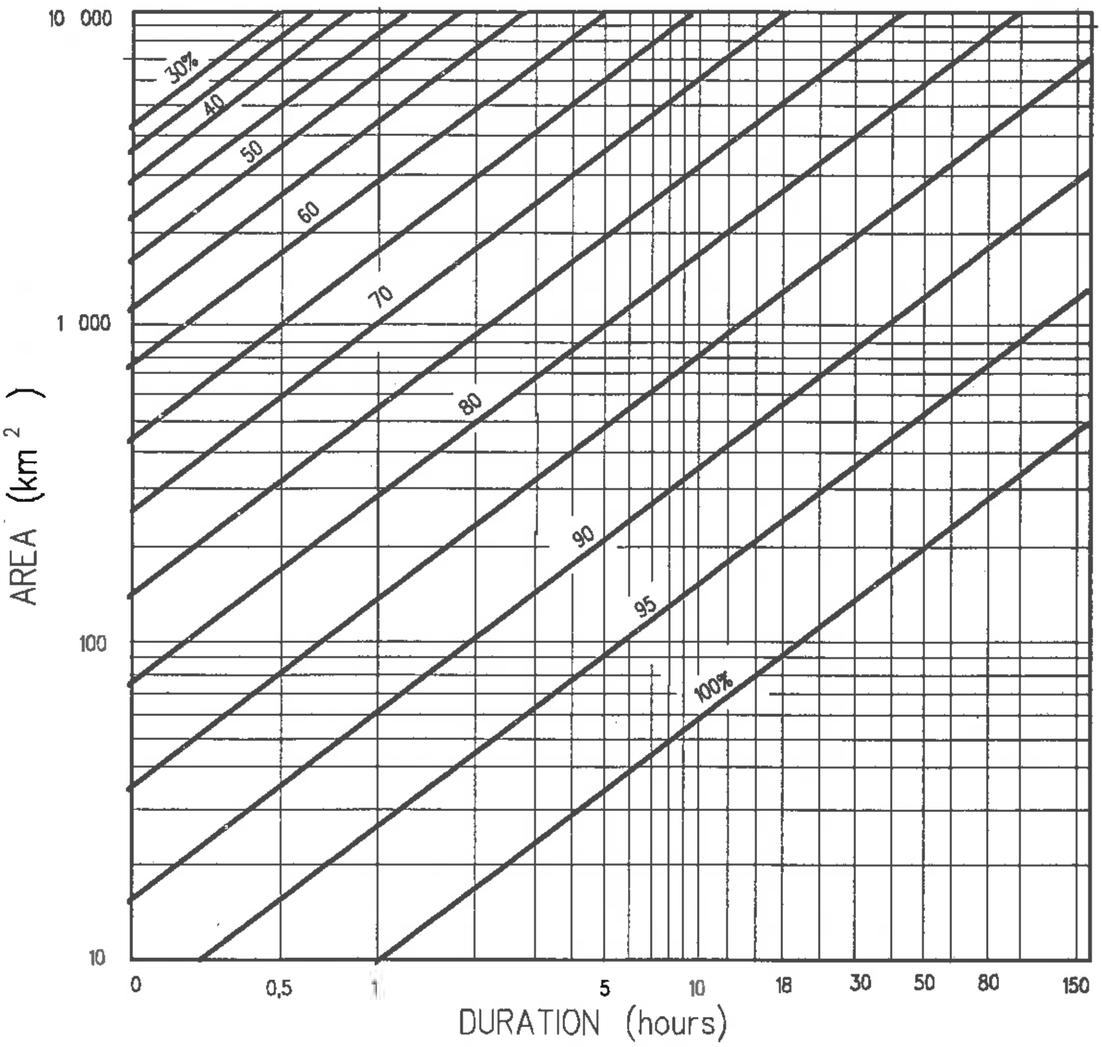


FIGURE 13.8 Area reduction factor (from this handbook)

FIGURE G. 1 MINIMUM STORM LOSSES

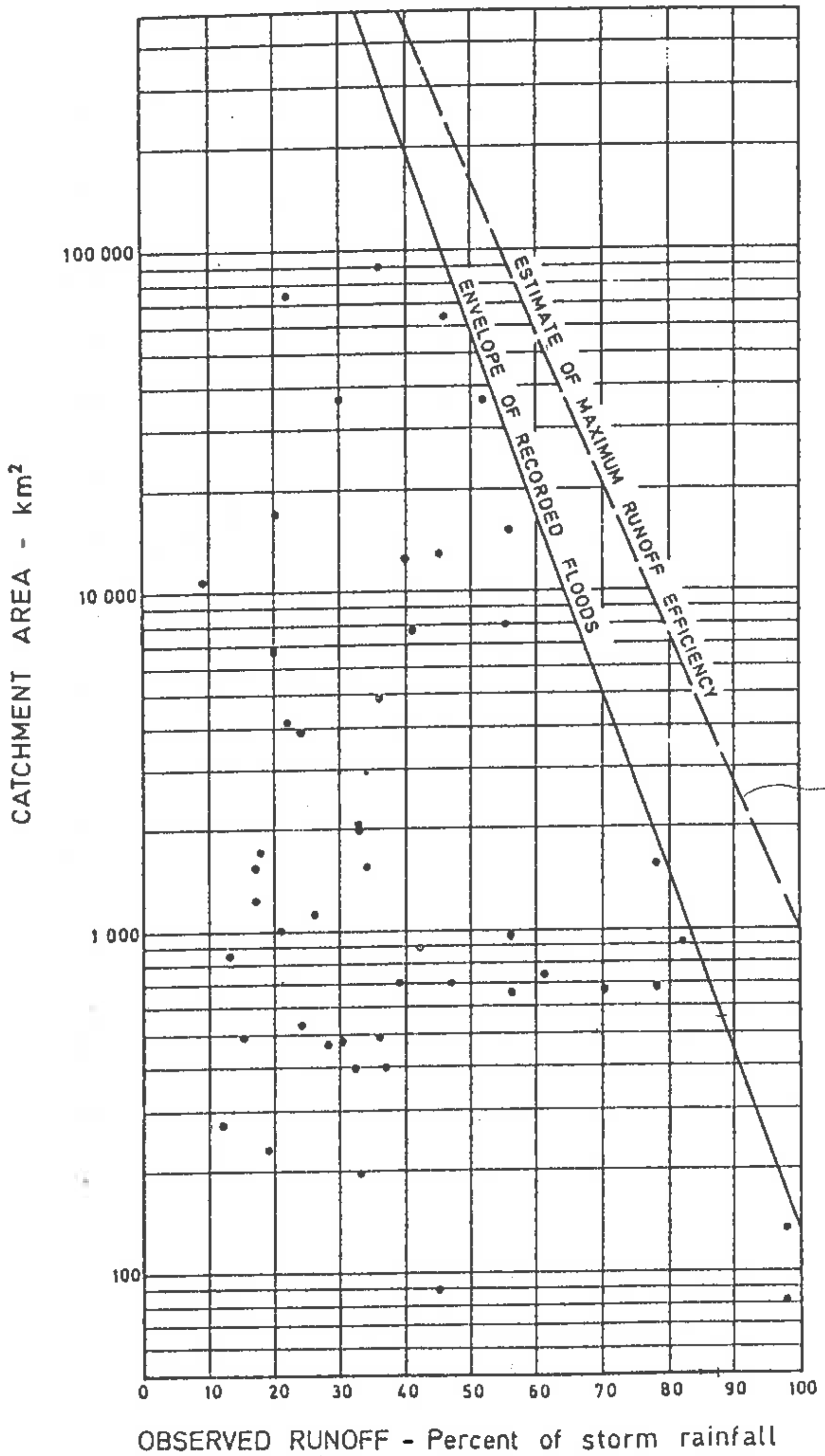


FIGURE 13.9 Minimum storm losses from HRU 1/72 Fig G.1

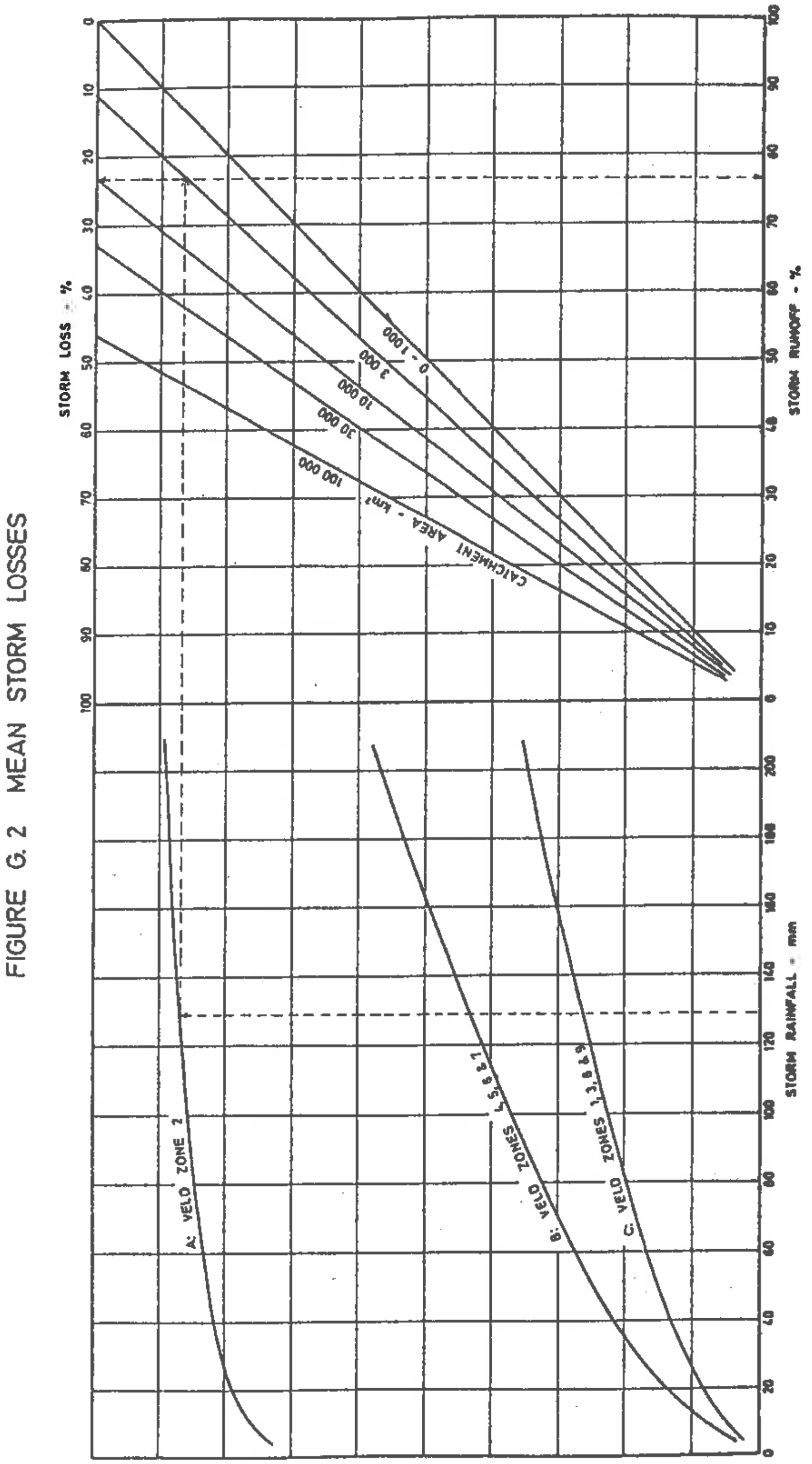


FIGURE 13.10 Mean storm losses from HRU 1/72 Fig G.2

13.10.4 LIST OF TABLES

Table 13.1 Generalized veld type zones with corresponding lag coefficients C_t from HRU 1/72 Table F.2

Zone No (Fig 1)	Generalized veld type	C_t
1	Coastal tropical forest	0,99
2	Schlerophyllous bush	0,62
3	Mountain sourveld	0,35
4	Grasslands of interior plateau	0,32
5	Highlands sourveld and Dohne sourveld	0,21
5a	As for 5 - but soils weakly developed	0,53
6	Karoo	0,19
7	False Karoo	0,19
8	Bushveld	0,19
9	Tall sourveld	0,13

Table 13.2 Values of K_u in the expression $Q_p = K_u\ A/T_l$ from HRU 1/72 Table F.4

Zone No (Fig 1)	Factor K_u
1	0,261
2	0,306
3	0,277
4	0,386
5	0,351
5a	0,488
6	0,265
7	0,315
8	0,367
9	0,321

TABLE 13.3 Regionally generalized dimensionless one-hour unitgraphs from HRU 1/72 Table F.3

Table F.3 : Regionally generalized dimensionless one-hour unitgraphs

Time expressed as ratio T/T_1	Discharge expressed as ratio Q/Q_p for zones									
	1	2	3	4	5	5A	6	7	8	9
0	0	0	0	0	0	0	0	0	0	0
0.05	.035	.012	.010	.011	.018	.004	.024	.006	.006	.011
0.10	.070	.024	.023	.024	.038	.011	.052	.014	.014	.027
0.15	.112	.036	.039	.038	.063	.019	.087	.024	.025	.043
0.20	.163	.052	.057	.041	.095	.027	.140	.032	.035	.065
0.25	.228	.072	.074	.070	.142	.037	.260	.044	.050	.093
0.30	.306	.091	.106	.089	.220	.050	.700	.058	.069	.142
0.35	.414	.121	.139	.111	.315	.064	.983	.074	.100	.225
0.40	.524	.152	.184	.138	.500	.083	1,000	.095	.150	.350
0.45	.709	.198	.261	.175	.685	.107	.970	.121	.245	.570
0.50	.921	.258	.376	.220	.810	.140	.915	.160	.655	.772
0.55	.983	.342	.518	.350	.936	.210	.848	.275	.905	.930
0.60	.996	.472	.670	.700	.985	.425	.795	.480	.980	.982
0.65	.998	.676	.809	.980	1,000	.885	.754	.700	.994	1,000
0.70	.964	.940	.970	1,000	.960	.958	.714	.950	.991	.985
0.75	.893	.991	1,000	.987	.800	.993	.678	.975	.966	.945
0.80	.826	.995	.990	.885	.675	.991	.641	.993	.860	.900
0.85	.758	.973	.935	.760	.588	.955	.605	1,000	.755	.814
0.90	.700	.888	.840	.670	.524	.740	.572	.995	.655	.750
0.95	.652	.807	.755	.580	.473	.535	.540	.980	.565	.670
1.00	.605	.741	.675	.530	.432	.440	.514	.900	.500	.600
1.05	.563	.678	.612	.470	.397	.385	.488	.805	.440	.530
1.10	.525	.622	.546	.430	.365	.340	.465	.730	.392	.472
1.15	.491	.567	.500	.393	.340	.300	.443	.655	.355	.413
1.20	.463	.513	.460	.364	.315	.265	.422	.590	.322	.364
1.25	.437	.467	.424	.336	.295	.235	.402	.530	.294	.318
1.30	.411	.425	.395	.310	.276	.209	.382	.477	.270	.280
1.35	.387	.394	.368	.288	.260	.187	.365	.432	.250	.260
1.40	.362	.364	.347	.271	.242	.169	.347	.388	.231	.241
1.45	.341	.338	.325	.252	.228	.152	.330	.350	.215	.225
1.50	.321	.313	.305	.235	.214	.140	.315	.308	.200	.210
1.55	.302	.291	.290	.218	.200	.128	.300	.280	.186	.198
1.60	.283	.272	.276	.201	.187	.116	.287	.255	.174	.188
1.65	.265	.253	.264	.187	.174	.105	.274	.232	.164	.176
1.70	.252	.236	.252	.172	.163	.097	.260	.211	.155	.168
1.75	.238	.220	.238	.159	.152	.088	.249	.194	.146	.158
1.80	.226	.206	.228	.147	.143	.081	.237	.177	.137	.151
1.85	.215	.192	.216	.136	.134	.074	.225	.164	.130	.144
1.90	.204	.181	.208	.125	.126	.067	.214	.152	.122	.137
1.95	.194	.171	.200	.115	.120	.061	.203	.140	.115	.131
2.00	.183	.160	.194	.108	.112	.055	.193	.130	.110	.124
2.05	.174	.152	.186	.098	.106	.050	.183	.120	.103	.119
2.10	.165	.143	.178	.089	.100	.046	.173	.111	.098	.113
2.15	.157	.136	.171	.081	.094	.041	.164	.102	.091	.108
2.20	.149	.130	.165	.074	.088	.038	.155	.094	.086	.103
2.25	.142	.123	.158	.068	.084	.034	.147	.087	.081	.097
2.30	.135	.118	.152	.062	.079	.031	.138	.081	.075	.093
2.35	.128	.114	.147	.056	.074	.028	.130	.075	.070	.087
2.40	.121	.108	.142	.052	.070	.025	.122	.069	.066	.085
2.45	.116	.104	.139	.047	.066	.023	.115	.063	.062	.079
2.50	.110	.100	.132	.043	.062	.021	.109	.058	.058	.075
2.55	.105	.096	.128	.039	.058	.019	.102	.053	.054	.071
2.60	.100	.093	.124	.035	.055	.017	.097	.049	.050	.070
2.65	.096	.089	.120	.032	.051	.015	.090	.045	.047	.063
2.70	.091	.085	.114	.029	.048	.013	.085	.041	.044	.061
2.75	.087	.081	.111	.026	.045	.012	.080	.039	.041	.055
2.80	.082	.078	.107	.023	.042	.011	.075	.036	.038	.053
2.85	.078	.074	.103	.021	.039	.010	.069	.033	.035	.049
2.90	.074	.070	.099	.019	.036	.009	.064	.030	.032	.045
2.95	.070	.066	.095	.017	.033	.008	.059	.029	.029	.041
3.00	.066	.063	.091	.016	.030	.006	.054	.026	.026	.038
3.05	.062	.060	.087	.012	.027	.004	.049	.023	.024	.035
3.10	.057	.056	.084	.011	.025	.003	.044	.021	.022	.030
3.15	.054	.053	.081	.009	.022	.002	.040	.019	.020	.027
3.20	.050	.050	.078	.008	.020	.001	.036	.017	.019	.022
3.25	.047	.047	.075	.006	.018	.000	.031	.015	.017	.018
3.30	.043	.044	.071	.004	.016		.027	.013	.015	.014
3.35	.039	.040	.068	.003	.013		.022	.011	.013	.010
3.40	.036	.037	.064	.002	.011		.018	.010	.011	.007
3.45	.032	.034	.062	.001	.010		.013	.008	.009	.004
3.50	.029	.031	.059	.000	.008		.010	.006	.007	.002
3.55	.025	.027	.056		.006		.005	.005	.005	.000
3.60	.022	.024	.051		.004		.000	.004	.004	
3.65	.019	.021	.048		.002			.002	.002	
3.70	.016	.018	.046		.001			.001	.001	
3.75	.012	.015	.043		.000			.000	.000	
3.80	.009	.011	.040							
3.85	.005	.008	.037							
3.90	.003	.005	.035							
3.95	.000	.002	.032							
4.00		.000	.029							
4.05			.027							
4.10			.024							
4.15			.021							
4.20			.019							
4.25			.016							
4.30			.013							
4.35			.011							
4.40			.008							
4.45			.006							
4.50			.003							
4.55			.000							
4.60										
4.65										
4.70										
4.75										
Dimensionless time to peak	0.62	0.78	0.75	0.70	0.65	0.78	0.40	0.85	0.68	0.85

TABLES G1 and G2 Standardized variates for several distributions (this handbook)

WT

RETURN PERIOD T IN YEARS	NON-EXCEEDANCE PROBABILITY	NORMAL DISTRIBUTION	NORMAL CONFIDENCE LIMITS 75%	NORMAL CONFIDENCE LIMITS 95%	EXPONENTIAL DISTRIBUTION	PEARSON TYPE III DISTRIBUTION [illegible] - 1,0	- 0,8	- 0,6	- 0,4	- 0,2	0	0,2	0,4	0,6	0,8	1,0
2	0,50	0,00	1,63 / $\sqrt{2N}$	2,77 / $\sqrt{2N}$	0,69	0,16	0,13	0,10	0,07	0,03	0,00	-0,03	-0,07	-0,10	-0,13	-0,16
5	0,80	0,84	1,89 -"-	3,23 -"-	1,61	0,85	0,86	0,86	0,86	0,85	0,84	0,83	0,82	0,80	0,78	0,76
10	0,90	1,28	2,20 -"-	3,74 -"-	2,30	1,13	1,17	1,20	1,23	1,26	1,28	1,30	1,32	1,33	1,34	1,34
20	0,95	1,64	2,49 -"-	4,25 -"-	3,00	1,32	1,39	1,46	1,52	1,59	1,64	1,70	1,75	1,80	1,84	1,88
50	0,98	2,05	2,87 -"-	4,89 -"-	3,91	1,49	1,61	1,72	1,83	1,94	2,05	2,16	2,26	2,36	2,45	2,54
100	0,99	2,33	3,13 -"-	5,34 -"-	4,61	1,59	1,73	1,88	2,03	2,18	2,33	2,47	2,62	2,76	2,89	3,02
200	0,995	2,58	3,38 -"-	5,76 -"-	5,30	1,66	1,84	2,02	2,20	2,39	2,58	2,76	2,95	3,13	3,31	3,49
500	0,998	2,88	3,69 -"-	6,27 -"-	6,21						2,88					
1 000	0,999	3,09	3,91 -"-	6,66 -"-	6,91	1,79	2,02	2,27	2,53	2,81	3,09	3,38	3,67	3,96	4,24	4,53
10 000	0,9999	3,72	4,58 -"-	7,80 -"-	9,21	1,88	2,18	2,53	2,90	3,30	3,72	4,15	4,60	5,05	5,50	5,96

TABLES G1 (above) and G2 (below) : Values of the standardised variate W_T

GENERAL EXTREME VALUE DISTRIBUTION ([illegible])

RETURN PERIOD T IN YEARS	-1,0 (EV 3)	-0,8 (EV 3)	-0,6 (EV 3)	-0,4 (EV 3)	-0,2 (EV 3)	0 (EV 3)	0,2 (EV 3)	0,4 (EV 3)	0,6 (EV 3)	0,8 (EV 3)	1,0 (EV 3)	1,14 (EV 1)
2		0,33	0,34	0,34	0,34	0,35	0,35	0,36	0,36	0,36	0,365	0,37
5		1,01	1,06	1,12	1,17	1,23	1,28	1,33	1,38	1,43	1,47	1,50
10		1,28	1,37	1,46	1,57	1,67	1,78	1,89	2,00	2,10	2,19	2,25
20		1,44	1,57	1,71	1,86	2,02	2,19	2,37	2,54	2,71	2,86	2,97
50		1,58	1,74	1,93	2,15	2,38	2,64	2,90	3,18	3,45	3,72	3,90
100		1,64	1,83	2,05	2,31	2,60	2,92	3,26	3,62	3,99	4,35	4,60
200		1,68	1,89	2,14	2,44	2,77	3,16	3,58	4,02	4,49	4,97	5,30
500		1,72	1,95	2,23	2,56	2,96	3,42	3,94	4,51	5,13	5,76	6,21
1 000		1,74	1,98	2,27	2,64	3,07	3,59	4,19	4,86	5,58	6,35	6,91
10 000		1,76	2,02	2,36	2,78	3,32	4,00	4,81	5,81	6,96	8,24	9,21

RETURN PERIOD T IN YEARS	1,2 (EV 2)	1,4 (EV 2)	1,6 (EV 2)	1,8 (EV 2)	2,0 (EV 2)	2,5 (EV 2)	3,0 (EV 2)	3,5 (EV 2)	4,0 (EV 2)	4,5 (EV 2)	5,0 (EV 2)	5,5 (EV 2)	6,0 (EV 2)
2	0,37	0,37	0,37	0,37	0,37	0,38	0,38	0,38	0,38	0,38	0,38	0,38	0,38
5	1,51	1,55	1,58	1,60	1,63	1,68	1,72	1,75	1,77	1,79	1,80	1,82	1,83
10	2,28	2,35	2,43	2,49	2,55	2,67	2,76	2,84	2,90	2,94	2,98	3,01	3,04
20	3,01	3,15	3,28	3,40	3,50	3,73	3,91	4,05	4,16	4,25	4,33	4,39	4,45
50	3,97	4,22	4,45	4,66	4,86	5,28	5,62	5,89	6,12	6,30	6,46	6,59	6,70
100	4,71	5,05	5,38	5,68	5,97	6,59	7,10	7,52	7,86	8,15	8,39	8,59	8,77
200	5,44	5,90	6,34	6,76	7,16	8,04	8,77	9,38	9,89	10,31	10,67	10,97	11,24
500	6,41	7,05	7,68	8,29	8,87	10,19	11,32	12,26	13,06	13,74	14,32	14,81	15,24
1 000	7,15	7,95	8,75	9,53	10,29	12,02	13,53	14,82	15,92	16,86	17,66	18,36	18,96
10 000	9,64	11,11	12,68	14,25	15,82	19,65	23,19	26,34	29,13	31,50	33,73	35,61	37,31

TABLE G3 PARAMETERS OF THE STANDARDIZED GEV DISTRIBUTION

g	k	$E(y)$	$var(y)$
-2.000	1.406	-1.247	3.204
-1.900	1.321	-1.182	2.505
-1.800	1.240	-1.127	1.984
-1.700	1.163	-1.080	1.590
-1.600	1.089	-1.041	1.287
-1.500	1.018	-1.008	1.052
-1.400	0.950	-0.980	0.868
-1.300	0.885	-0.957	0.721
-1.200	0.824	-0.938	0.602
-1.100	0.765	-0.922	0.507
-1.000	0.708	-0.910	0.428
-0.900	0.655	-0.901	0.362
-0.800	0.604	-0.894	0.307
-0.700	0.555	-0.889	0.261
-0.600	0.509	-0.887	0.222
-0.500	0.465	-0.886	0.188
-0.400	0.424	-0.886	0.159
-0.300	0.384	-0.888	0.134
-0.200	0.346	-0.892	0.112
-0.100	0.311	-0.896	0.094
0.000	0.277	-0.901	0.077
0.100	0.245	-0.907	0.063
0.200	0.215	-0.914	0.050
0.300	0.187	-0.922	0.039
0.400	0.160	-0.930	0.030
0.500	0.134	-0.938	0.022
0.600	0.110	-0.947	0.016
0.700	0.088	-0.956	0.010
0.800	0.067	-0.966	0.006
0.900	0.047	-0.975	0.003
1.000	0.028	-0.985	0.001
1.100	0.010	-0.994	0.000
1.200	-0.006	1.004	0.000
1.300	-0.022	1.013	0.001
1.400	-0.037	1.023	0.002
1.500	-0.050	1.032	0.005
1.600	-0.063	1.041	0.008
1.700	-0.075	1.049	0.011
1.800	-0.086	1.058	0.016
1.900	-0.097	1.066	0.021
2.000	-0.107	1.074	0.026
2.100	-0.116	1.082	0.032
2.200	-0.125	1.089	0.038
2.300	-0.133	1.097	0.044
2.400	-0.140	1.104	0.051
2.500	-0.148	1.110	0.058
2.600	-0.154	1.116	0.065
2.700	-0.160	1.123	0.072
2.800	-0.166	1.128	0.080
2.900	-0.172	1.134	0.087
3.000	-0.177	1.139	0.094
3.100	-0.182	1.145	0.102
3.200	-0.187	1.150	0.110
3.300	-0.191	1.154	0.117
3.400	-0.195	1.159	0.125
3.500	-0.199	1.163	0.132
3.600	-0.203	1.168	0.140
3.700	-0.207	1.172	0.148
3.800	-0.210	1.176	0.155
3.900	-0.213	1.180	0.163
4.000	-0.217	1.183	0.170
4.100	-0.220	1.187	0.178
4.200	-0.223	1.191	0.186
4.300	-0.225	1.194	0.193
4.400	-0.228	1.197	0.201
4.500	-0.231	1.201	0.208
4.600	-0.233	1.204	0.215
4.700	-0.236	1.207	0.223
4.800	-0.238	1.210	0.230
4.900	-0.240	1.213	0.237
5.000	-0.242	1.215	0.244

TABLE G4 PROPERTIES OF THE STANDARDIZED NORMAL DISTRIBUTION

Table A

y	G(y)%	y	G(y)%
0.00	50.00	0.00	50.00
0.05	51.99	-0.05	48.01
0.10	53.98	-0.10	46.02
0.15	55.96	-0.15	44.04
0.20	57.93	-0.20	42.07
0.25	59.87	-0.25	40.13
0.30	61.79	-0.30	38.21
0.35	63.68	-0.35	36.32
0.40	65.54	-0.40	34.46
0.45	67.36	-0.45	32.64
0.50	69.14	-0.50	30.86
0.55	70.88	-0.55	29.12
0.60	72.57	-0.60	27.43
0.65	74.22	-0.65	25.78
0.70	75.81	-0.70	24.19
0.75	77.34	-0.75	22.66
0.80	78.81	-0.80	21.19
0.85	80.24	-0.85	19.76
0.90	81.59	-0.90	18.41
0.95	82.89	-0.95	17.11
1.00	84.13	-1.00	15.87
1.05	85.31	-1.05	14.69
1.10	86.43	-1.10	13.57
1.15	87.49	-1.15	12.51
1.20	88.49	-1.20	11.51
1.25	89.44	-1.25	10.56
1.30	90.32	-1.30	9.68
1.35	91.15	-1.35	8.85
1.40	91.92	-1.40	8.08
1.45	92.65	-1.45	7.35
1.50	93.32	-1.50	6.68
1.55	93.94	-1.55	6.06
1.60	94.52	-1.60	5.48
1.65	95.05	-1.65	4.95
1.70	95.54	-1.70	4.46
1.75	95.99	-1.75	4.01
1.80	96.41	-1.80	3.59
1.85	96.78	-1.85	3.22
1.90	97.13	-1.90	2.87
1.95	97.44	-1.95	2.56
2.00	97.72	-2.00	2.28
2.05	97.98	-2.05	2.02
2.10	98.21	-2.10	1.79
2.15	98.43	-2.15	1.57
2.20	98.61	-2.20	1.39
2.25	98.78	-2.25	1.22
2.30	98.93	-2.30	1.07
2.35	99.06	-2.35	0.94
2.40	99.18	-2.40	0.82
2.45	99.29	-2.45	0.71
2.50	99.38	-2.50	0.62
2.55	99.46	-2.55	0.54
2.60	99.53	-2.60	0.47
2.65	99.60	-2.65	0.40
2.70	99.65	-2.70	0.35
2.75	99.70	-2.75	0.30
2.80	99.74	-2.80	0.26
2.85	99.78	-2.85	0.22
2.90	99.81	-2.90	0.19
2.95	99.84	-2.95	0.16
3.00	99.86	-3.00	0.14
3.05	99.88	-3.05	0.16
3.10	99.90	-3.10	0.10
3.15	99.92	-3.15	0.08
3.20	99.93	-3.20	0.07
3.25	99.94	-3.25	0.06
3.30	99.95	-3.30	0.05
3.35	99.96	-3.35	0.04
3.40	99.97	-3.40	0.03
3.45	99.97	-3.45	0.03

Table B

T	G(y)%	W_T
1000	0.1	-3.09
500	0.2	-2.88
200	0.5	-2.58
100	1	-2.33
50	2	-2.05
20	5	-1.64
10	10	-1.28
5	20	-0.84
2	50	0.00
5	80	0.84
10	90	1.28
20	95	1.64
50	98	2.05
100	99	2.33
200	99.5	2.58
500	99.8	2.88
1 000	99.9	3.09
5 000	99.98	3.55
10 000	99.99	3.72

3.50	99.98	-3.50	0.02
3.55	99.98	-3.55	0.02
3.60	99.98	-3.60	0.02
3.65	99.99	-3.65	0.01
3.70	99.99	-3.70	0.01
3.75	99.99	-3.75	0.01

Chapter 14

CASE STUDIES

Contents

14.1 INTRODUCTION

In the preface to this handbook you were urged to carry out the first three case studies in this chapter as these require the application of the principal flood magnitude determination methods detailed in the handbook. The purpose of this suggestion was to demonstrate the wide range of solutions that the different methods produce. The next question is which of the solutions should be accepted for the problem. To answer this you will have to go back into the handbook and read through the relevant sections. However, as has been stressed throughout the handbook you will ultimately have to apply your own knowledge and experience to some of the more complex problems.

The examples have been chosen to demonstrate the application of the methods to various problems that may be encountered in practice. Emphasis is on the *interpretation of the results* rather than the mechanics of the calculations.

Go through the examples slowly and conscientiously. If there is anything that is not clear make a note of it and carry on. Do not make use of the printer options until you understand the screen presentations, then go through the calculations again, this time printing the results and the graphs. Keep these in a file for future reference.

Where opinions are expressed in these notes question them and form your own. Note the difference between your answers and those presented. These will give you an indication of the likely range of results that different users will obtain when using these methods.

If you should get despondent at any stage think of the alternative - carrying out the calculations by hand !

14.2 SAMPLE DATA

The following sample data sets are used in several of the case studies. These plus a number of additional data sets are also included in the computer programs.

CHARACTERISTIC	UNIT	MORETELE	GROOTDRAAI
Area of catchment	km^2	156	8254
Length of longest feeder	km	28	230
Height difference (equal area)	m	213	195
Height difference (1085 method)	m	175	170
Mean annual precipitation	mm	701	698
Coefficient categories for the rational method:			
MAP	mm	2	2
Slope	-	2	1
Permeability	-	3	3
Vegetation	-	2	3

Representative rainfall data from TR 102
Moretele catchment (Bryntirion)
Grootdraai catchment (Kranspan)
Extracts from TR 102 :

WEATHER BUREAU NUMBER.	STATION	LAT.	LONG.	YEARS OF RECORD	MEAN ANNUAL RAINFALL	DURATION	RECURRENCE INTERVAL. - YEARS - 2	5	10	20	50	100	200
513404	PRETORIA (BRYNTIRION)	25°44'	28°14'	71	701	1 DAY.	62	88	108	130	163	191	223
						2 DAY.	78	112	138	167	210	246	287
						3 DAY.	88	128	158	191	241	283	330
						7 DAY.	111	156	190	226	278	321	368
480267	KRANSPAN	26°27'	30°09'	44	698	1 DAY.	62	82	97	112	135	153	173
						2 DAY.	77	102	120	140	167	189	213
						3 DAY.	86	115	136	158	188	213	240
						7 DAY.	113	151	179	207	246	278	312

Lightning ground flashes		6	9
Length to centroid of catchment	km	18	52
Veld type zone	-	4	4

Probable maximum precipitation - see Fig 13.6 Chapter 13.
Regional maximum flood k-factor - see Fig 13.4 in Chapter 13.

DATA SETS OF RECORDED ANNUAL MAXIMA

Year starting October	Bryntirion rainfall	Vaal @ Grootdraai	Vaal @ Vereeniging	Vaal @ Riverton	Willem Nels @ Langevalley	Buffels @ Laingsburg	Pongola @ Grootdraai
	mm	m^3/s	m^3/s	m^3/s	m^3/s	m^3/s	m^3/s
1900			4400				
1902			-				
1902			-				
1903			-				
1904			-				
1905		264	-				
1906	58	1260	-				
1907	52	558	-				
1908	33	1417	-	3156			
1909	156	1144	-	1821			
1910	95	2268	-	1767			
1911	34	240	-	637			
1912	37	113	-	603			
1913	42	117	-	508			
1914	69	396	-	2998			
1915	77	486	3000	2263			
1916	39	321	-	385			
1917	111	1417	4100	4354			
1918	69	452	-	2678			
1919	48	90	-	542			
1920	69	767	-	850		392	
1921	62	1171	-	1762		168	
1922	75	2059	-	1985		86	
1923	44	120	635	562		34	
1924	56	700	1850	2710		742	
1925	52	100	299	311		1	
1926	81	194	627	529		65	
1927	46	154	245	508		7	
1928	169	368	682	726		365	
1929	68	761	958	1141		9	
1930	90	122	369	554		15	
1931	63	139	225	566		19	
1932	46	55	172	227		68	
1933	77	864	1564	3653		513	
1934	67	229	1492	2520		445	
1935	45	738	1589	1573		129	
1936	93	750	2198	2670		188	
1937	139	475	652	244		738	
1938	37	1460	1770	1999		533	
1939	98	597	1088	546		73	
1940	65	706	1222	801		206	
1941	70	200	460	221		86	
1942	68	291	1723	952		40	
1943	49	1546	4100	3189*		47	
1944	69	211	988	462		95	
1945	68	416	657	395		16	
1946	60	238	514	277		14	
1947	66	331	391	476		17	
1948	116	120	413	176		183	
1949	42	523	633	469		1	

* **Exceeds capacity of gauging weir.**

DATA SETS OF RECORDED ANNUAL MAXIMA							
Year starting October	Bryntirion rainfall	Vaal @ Grootdraai	Vaal @ Vereeniging	Vaal @ Riverton	Willem Nels @ Langevalley	Buffels @ Laingsburg	Pongola @ Grootdraai
	mm	m^3/s	m^3/s	m^3/s	m^3/s	m^3/s	m^3/s
1950	79	197	497	176	40	95	355
1951	47	335	605	150	10	660	215
1952	55	680	1482	991	14	207	384
1953	58	154	324	184	14	82	412
1954	44	1823	2591	1685	11	41	847
1955	46	937	728	289	2	-	1067
1956	38	1020	3886	1537	10	-	555
1957	52	600	1716	3132	11	-	3016
1958	46	547	607	96	5	-	650
1959	102	195	579	238	15	-	301
1960	39	613	1010	650	10	-	1348
1961	72	421	462	253	40	-	370
1962	64	271	615	236	9	-	3404
1963	60	285	602	169	19	-	239
1964	73	681	1664	607	11	-	2373
1965	69	38	326	612	17	-	982
1966	50	984	1646	2076	24	-	379
1967	68	169	668	45	3	-	384
1968	40	144	190	27	11	-	334
1969	68	331	310	13	6	-	348
1970	70	137	803	17	24	-	1004
1971	73	1150	1027	20	10	-	844
1972	53	23	329	84	6	-	1435
1973	37	383	1030	760	25	-	596
1974	62	1790	3380	2927	5	-	493
1975	60	641	1503	3188	11	-	431
1976	104	961	1573	1881	5	-	533
1977	42	240	1852	700	7	-	461
1978	245	140	370	28	17	-	270
1979		215	516	26	4	-	424
1980		192	560		551	-	261
1981		262				5680	9200
1982							1710
1983							
1984							
1985							
1986							
1987							
1988							
1989							
1990							
1991							
1992							
1993							
1994							
1995							
1996							
1997							
1998							
1999							

Case study

14.3 BRYNTIRION @ PRETORIA : DAILY RAINFALL MAXIMA

14.3.1 Problem definition

Determine the maximum 1-day storm rainfall-frequency relationship for the Weather Bureau station No. 513 404 Pretoria (Bryntirion).

This station was opened in 1906. In February, 1978 the maximum daily rainfall was 245 mm compared with the previous maximum of 169 mm. In this example the hand calculation method of analysis is illustrated.

Calculate P_2 and P_{200} using the moments of the raw data in the GEV and Gumbel distributions, and the moments of the log transformed data in the LN and LP3 distributions. Calculate the 95% confidence bands for the LN distribution.

			Raw data	Log-transformed data
Mean	P	=	68,2	1,7973
Standard deviation	s	=	33,8	0,1688
Skewness coefficient	g	=	2,76	0,931
	N	=	73	

14.3.2 Statistical analysis : Hand calculation : Determine P_2 and P_{200}

<u>*Log-normal with 95% confidence bands*</u>

$\log P_2$	$= 1,7973 + 0,1688 \,.\, 0$	$= 1,7973$
$\log P_{200}$	$= 1,7973 + 0,1688 \,.\, 2,58$	$= 2,2328$
P_2	$= 62,7$	
P_{200}	$= 170,9$	
$\log P_{2/95\%}$	$= 1,7973 + 0,1688\,(0 \pm 2,77 / 146)^{0,5}$	
	$= 1,7973 + 0,0387$	$= 1,8360$
	$\&\ 1,7973 - 0,0387$	$= 1,7586$
$P_{2/95\%}$	$= 68,5\ \&\ 57,4$	
$\log P_{200/95\%}$	$= 1,7973 + 0,1688\,(2,58 \pm 5,76 / 100)^{0,5}$	
	$= 1,7973 + 0,5160$	$= 2,3133$
	$\&\ 1,7973 + 0,3550$	$= 2,1523$
$P_{200/95\%}$	$= 206\ \&\ 142$	

Log-Pearson Type 3

Log P_2	$= 1,7973 + 0,1688\ (-\ 0,15)$	$= 1,7720$
Log P_{200}	$= 1,7973 + 0,1688\ (3,43)$	$= 2,3763$
P_2	$= 59,2$	
P_{200}	$= 237,8$	

EV1

P_2	$= 68,2 + 33,8\ (0,780\ .\ 0,37 - 0,450)$	$= 62,7$
P_{200}	$= 68,2 + 33,8\ (0,780\ .\ 5,30 - 0,450)$	$= 192,7$

GEV

for g	$= 2,76$		
k	$= -0,164$	From Table G3 for g	$= 2,76$
$E.y_2$	$= 1,126$	" " "	
$Var.y_2$	$= 0,0768$	" " "	
W_2	$= 0,38$	From Table G2 for g	$= 2,76$
W_{200}	$= 8,42$		

P_2 $= 68,2 + [\ (\ 33,8^2\ /\ 0,0768\ )\ ]^{0,5}\ .\ (\ 1 - 1,126 + 0,164\ .\ 0,38\ )$
$= 60$

P_{200} $= 68,2 + [\ (\ 33,8^2\ /\ 0,0768\ )\ ]^{0,5}\ .\ (\ 1 - 1,126 + 0,164\ .\ 8,42\ )$
$= 221$

Summary

	LN/MM	LP3/MM	EV1/MM	EV2/MM	95 % confidence limits for LN/MM
P_2	63	59	63	60	57 - 68
P_{200}	171	238	193	221	142 - 206

14.3.3 Interpretation

The first step in the evaluation of the relative merits of the four distributions is to examine the skewness coefficient which has a value of 2.76 for the raw data and 0.931 for the log transformed data. The EV1 requires a skewness coefficient of 1.14 and is therefore inappropriate when applied to the raw data. Similarly the log normal distribution is a doubtful contender because of the assumption of zero skewness of the log transformed data.

This leaves the two three-parameter distributions LP3 and EV2. However, there is another possibility, and that is the log EV1 which has not been calculated.

These results can also be compared with results from TR 102 given in the sample data set above, where $P_2 = 62$ mm and $P_{200} = 223$ mm.

14.3.4 Further exercises

1. Calculate P_2 and P_{200} using the log EV1 distribution. Would you consider this to be more appropriate than the other distributions used so far?

2. Use the program STFLOOD to produce graphical presentations (the data set is included in the program).

3. Look for the presence of outliers in the graphical presentations (they are there !). Can they be explained?

4. Why does the LN value for P_{200} differ from that in TR 102?

5. What value would you recommend for P_{200}?

6. Now use the program REGFLOOD for another set of analyses. Do you wish to change your mind about your recommendation?

Case study

14.4 MORETELE RIVER @ PRETORIA : SMALL CATCHMENT

14.4.1 Problem definition

Use all available methods to obtain the best possible estimate of the 100-year return period flood peak at this site.

This catchment is within the Pretoria municipal area, and experienced the Pretoria storm described by Kovàcs (1978) and which produced the highest rainfall at Bryntirion in the first case study. It is the largest of the catchments included in that study (156 km^2) and is numbered 12 in the figure below. Large parts of the catchment have been urbanised in recent years.

14.4.2 Procedure

Use the sample data listed at the beginning of this chapter and which is also included in the program DETFLOOD to calculate the flood peak-frequency relationship for this site using the original as well as the alternative implementations. This is the easiest part of the problem !

The next step would be to use some of the other flood frequency estimation methods that are possibly more applicable to small urban catchments. They will doubtless each produce different answers. A study of the literature will not be very helpful because each method has its supporters. Readers of this handbook should be particularly wary of research publications which attempt to demonstrate that one method is superior to other contenders. Such a conclusion requires an analysis of recorded data from a large number of catchments each with long, accurate flow records and cannot be based on theoretical grounds or simultaneous observations of storm rainfall and runoff from a few small catchments.

A far more productive approach would be to go through each method and review the assumptions used in the method to see whether these could be improved when applied to the specific site. For example examine the statistical properties of storm rainfall in the light of local data. In this case examine the daily rainfall-frequency relationship developed for Bryntirion in the first case study. This is the station which registered 245mm and is located close to the centre of the storm in the figure below.

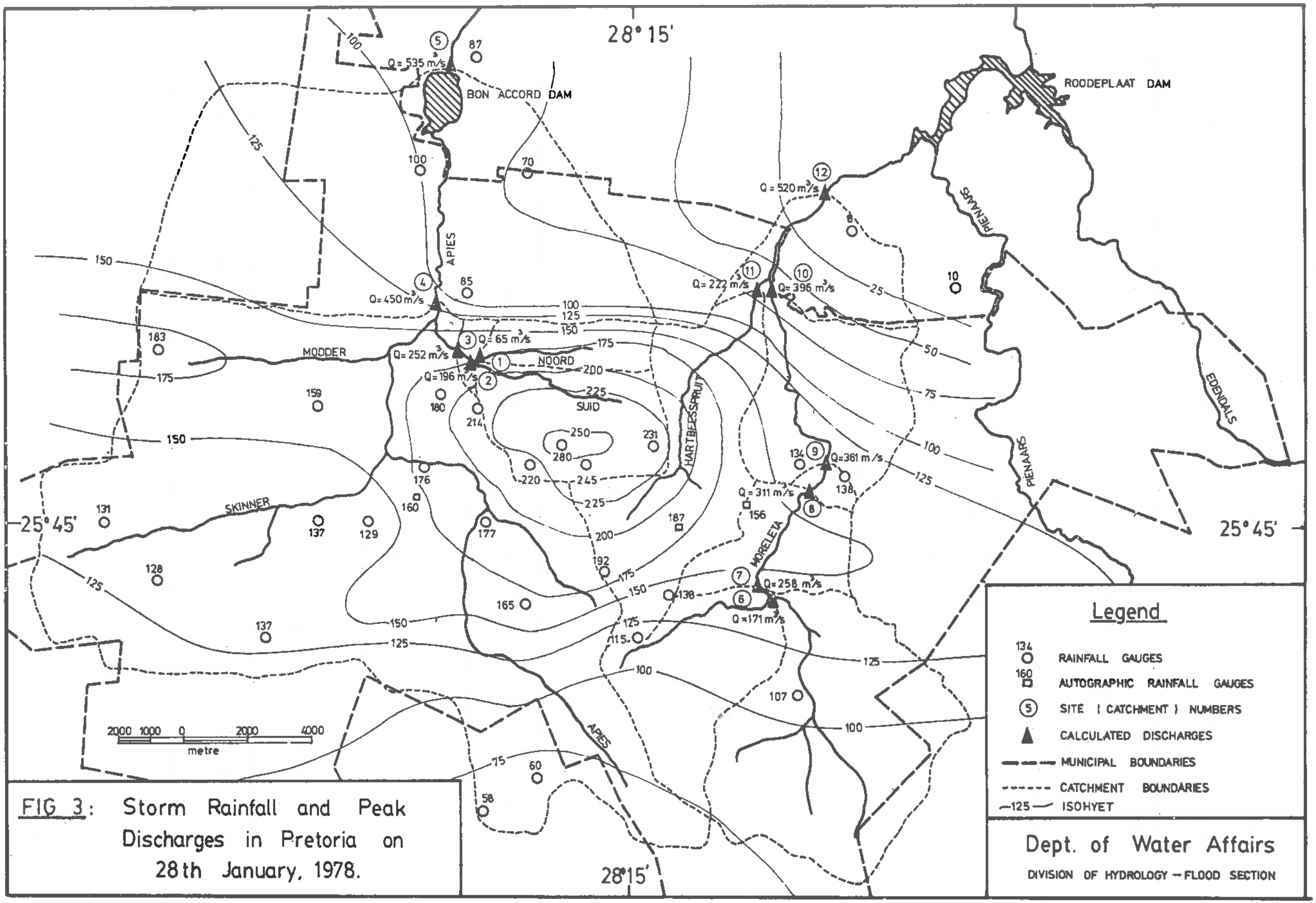


FIG 3: Storm Rainfall and Peak Discharges in Pretoria on 28th January, 1978.

Examine the various methods described in Chapter 3 that are used to determine shorter duration storm rainfall and evaluate their appropriateness in the Pretoria area.

Examine the depth-duration-frequency relationships of storm rainfall in this area.

Note the areal distribution of the rainfall shown in the figure. How does this influence the assumptions of uniformly distributed rainfall in most methods. Would you revise the area reduction factor on the basis of this information?

The observed runoff coefficients for the twelve sites identified on the figure above were as follows:

Site	Runoff coefficient
1	1,05
2	0,95
3	0,90
4	0,28
5	0,26
6	0,76
7	0,80
8	0,75
9	0,84
10	0,75
11	0,87
12	0,64

How would you adjust the runoff coefficient in the rational method based on this information (site 12) bearing in mind the difference between C and C_T?

Go through a similar procedure for all methods used in the analysis.

If you were satisfied that an alternative relationship for determining (say) the ARF was better than the relationship contained in a particular method would you use it? (If you answered yes, please think more deeply !)

14.4.3 Interpretation of the results

In most studies the uncertainties in the accuracy of the results arise from the lack of data and the appropriateness of the available methods. This case study has been chosen because of the wealth of rainfall data as well as the availability of several other flood peak-frequency methods applicable to urban areas which could be applied to this problem.

Case study

14.5 VAAL RIVER @ GROOTDRAAI DAM : ALL METHODS

14.5.1 Problem definition

Determine the flood peak-frequency relationships for the Grootdraai Dam on the Vaal River near Standerton.

A flow gauging station C1M01A was established in 1905 in the Vaal River just downstream of the present Grootdraai Dam site. Records are available from 1905 to 1982 when storage commenced at the dam. The station was well calibrated using a cable suspended across the river at high stages when the gauging weir was drowned. This is one of the longest and most reliable records available in South Africa.

14.5.2 Statistical analysis : hand calculation

Carry out the calculations by hand using the same method as that in the Bryntirion case study using the record and parameters given in Table 2.1 in Chapter 2. Note that this record extends only to 1975.

Based on this information alone, which distribution would you recommend?

The skewness coefficient of the logarithms is -0,35. The log normal distribution assumes a zero skewness coefficient while the LEV1/MM distribution assumes a positive skewness coefficient of 1,14. On the basis of this coefficient alone, the results using the log normal distribution should be treated with caution, while those from the LEV1/MM distribution would be highly suspect. Make a note to check this when examining the LEV1/MM graph and the table of estimated maxima.

The next question to be answered is whether to use the two parameter log normal distribution or one of the two three parameter distributions,

An examination of the ranked data in Fig 2.1 in Chapter 2 shows that the four highest values occurred in 1910, 1922, 1974 and 1954, while the four lowest values occurred in 1919, 1932, 1965 and 1972. These years are well distributed within the data set and support the assumption of no trends in the data. A good rule of thumb is that if there are no visible trends in the data it is unlikely that they will be detected by a statistical analysis.

There are no apparent high outliers in the table of the ranked values in Table 2.1 nor in Fig 2.1.(d), although there is some evidence of possible low outliers in the table and Fig 2.1(e).

The final question is whether the apparent skewness is the result of the relatively small sample, although 70 years is a long record by hydrological standards.

The standard error of the estimate (S.E.E) of the skewness coefficient is given by:

$$SEE\ (g) = [\ 6N(N-1)\ /\ (N-2)(N+1)(N+3)\ ]^{0,5}$$

For $N = 70$, $SEE(g) = 0{,}286$

The skewness coefficient of -0,349 is therefore located at 1,22 standard deviations from a value of zero. Some hydrologists assume that the true value of the skewness coefficient is zero unless the value of g differs from zero by more than twice the standard error of estimate, or if consistent non-zero values are obtained from other stations with long records in the region. In this case g differs from zero by 1,22 standard deviations and following this criterion it should be made equal to zero, ie the log normal distribution would be more appropriate.

Because of their increased flexibility, three parameter distributions will provide better visual fits than the two parameter log normal distribution, but this does not necessarily mean that they are more appropriate. This can only be ascertained by studying the data from other catchments in the region and carrying out regional analyses.

14.5.3 Statistical analysis : microcomputer application (Program STFLOOD)

An additional seven years of data have become available since the decision to build the dam and its completion which invalidated subsequent measurements at the site which is just downstream of the dam. These are:-

75/76	641 m^3/s
76/77	961
77/78	240
78/79	140
79/80	215
80/81	192
81/82	262

Three of the earlier maxima also have to be corrected. These are:-

59/60	195
73/74	383
74/75	1790

These changes have been incorporated in the program STFLOOD.

Run the program and carry out a further set of analyses. This time you have the opportunity to examine the graphical presentations.

1. Has the additional data changed the results significantly?

2. Have the graphs given you more insight than the calculations alone?

14.5.4 Rational method : microcomputer application

Run the program DETFLOOD using both the original and alternative implementations of the rational method.

The assumption of uniformly distributed storm rainfall over a catchment of this size is questionable. However, some interesting comparisons can be made between these results and the statistical analyses from this well-gauged catchment.

The alternative implementation of the rational method provides you with the means of calibrating the method by adjusting the runoff coefficient. Note the factors that the 2-year and 100-year rational results have to be multiplied by to reduce them to the accepted statistical analysis results. Run the program DETFLOOD again and enter these calibration factors when requested to do so. Compare the calibrated results for all return periods with the statistical analysis results.

Do you agree that it would be reasonable to use these calibration factors when applying the rational method to ungauged sites in the region?

What additional checks should be carried out to confirm this calibration procedure?

14.5.5 Unit hydrograph method : microcomputer application

Run the program DETFLOOD using both the original and alternative algorithms of the unit hydrograph method.

The catchment area at the site is 8 254 km² which is well beyond the limit of 5 000 km² recommended by the authors of the method. They suggested that catchments larger than 5 000 km² could be divided into smaller sub-catchments and the resulting hydrographs be routed to the site being investigated.

Despite this caution, what are your views on the acceptability of the results in this case?

Carry out the calibration procedure in the same way as described for the rational method above, and answer the same questions.

14.5.6 Summary of calculations (all methods)

Summarise your calculations and conclusions. This data set is very useful for comparing the merits of all methods detailed in this handbook. (You will have the opportunity of using this data set in the program REGFLOOD later.)

Time spent on careful analysis followed by a comparison of the results will be well spent.

This type of comparison based on a long, reliable record is the only way of determining which method is likely to give more acceptable results in similar situations.

Case study

14.6 VAAL @ VEREENIGING : INCLUSION OF HISTORICAL MAXIMA

14.6.1 Problem definition

Determine the flood peak-frequency relationship in the Vaal River at Vereeniging by making use of the records at Vaal Dam and Engelbrechtsdrif gauging weir just downstream of the dam, and historical records of maximum peaks observed at Vereeniging prior to the establishment of the gauging station at Engelbrechtsdrif.

For the purposes of this exercise assume that the flood peak attenuation effect of Vaal Dam will be counterbalanced by concurrent flows in the tributaries which enter the Vaal River between Vaal Dam and Vereeniging. In practice this assumption would have to be tested.

Records are available since 1923 at Engelbrechtsdrif and subsequently at Vaal Dam after its completion. High flood levels were observed at Vereeniging in 1894, 1915, and 1917 at the old railway bridge.

Good surveys of the river channel between the Vaal Barrage and Vaal Dam are available, and the flood peaks associated with the 1894 and 1917 observed levels can be calculated with an acceptable degree of accuracy. The 1915 flood level and the associated flood peak are approximate values. The seven highest observed flows during the 87 year period 1894 to 1981 were as follows. The levels refer to the Rand Water Board's No. 1 intake in the Vaal Barrage.

Rank	Year	R L	Estimated peak flow (m^3/s)
1	1894	1429.12	4 400
2	1917	1428.63	4 100
3	1975	1428.34	3 900
4	1944	1427.54	3 500
5	1957	1427.38	3 400
6	1915	?	3 000 (approximate)
7	1967	1424.79	2 200

(The reduced levels (RL) for 1894 and 1917 relate to observations at the old railway bridge. A correction was made to allow for the estimated water profile slope between the two points. The other levels are observed values.)

A further assumption is made that all other peak flows at Vereeniging for the period 1894 - 1922 were less than the 1915 peak of 3 000 m3/s.

14.6.2 Hand calculation method

The hand calculation method detailed below follows that described by Reich (1976) and is a useful introduction to the slightly different procedures used in the program REGFLOOD.

The earlier historical floods cannot simply be added into the continuous record from 1923 because the total length of the record is from 1894 to 1981 (88 years) and not 1923 to 1981 (69 plus three years). Conventional analytical procedures cannot be used because the values of the remaining peaks between 1894 and 1923 are not known, and therefore the mean, standard deviation and skewness coefficient of the full data set cannot be calculated.

The problem can be solved graphically by adjusting the plotting positions. The Weibull plotting position which is applicable to the log normal and log Pearson Type 3 distributions is:

$$T = \frac{N+1}{m}$$

where T is the plotting position in years and N is the total number of observations .

The adjusted rank m_a is given by:

$$ma = m_b + \frac{t - m_b}{N - m_b} \quad (m - m_b)$$

and

$$T_a = \frac{t+1}{m_a}$$

where :

m	is the rank without regard to the missing years
m_b	is the rank of the lowest historical flood
N	is the total number of continuous floods plus historical observed floods
t	is the total span of years since the first historic flood, including missing years.

In this example these values are:-

N	= 59+ 3	= 62	
t	= 29+ 59	= 88	
m_b	= 6		(from the table above)

The adjusted plotting positions for the ranked sequence up to and including the lowest historical peak remain unchanged, but those for lower ranked peaks have to be modified.

For $m < m_b$, $m_a = m$
otherwise
$m_a = 6 + [\ (88-6)/(62-6)\ ]\ .\ (m-6)$

The modified plotting positions T_a of the ten highest values are :-

Rank m	Year	Peak m^3/s	Initial plotting position T	Adjusted rank m_a	Adjusted plotting position T_a
1	1894	4 400	-	1	89 / 1
2	1917	4 100	-	2	89 / 2
3	1975	3 900	60/1	3	89 / 3
4	1944	3 500	60/2	4	89 / 4
5	1957	3 400	60/3	5	89 / 5
6	1915	3 000	-	6	89 / 6
7	1967	2 200	60/4	7,46	89 / 7,46
8			60/5	8,93	89 / 8,93
9			60/6	10,39	89 / 10,39
10			60/7	11,86	89 / 11,86
etc	etc	etc	etc	etc	etc

The same procedure can be used to adjust other plotting position formulae.

14.6.3 Microcomputer application

Carry out the calculation using the program REGFLOOD, firstly without the historical data, and then including the additional information. Dump the log-probability graphs (option 8 in the graph menu).

14.6.4 Interpretation of results

The UK, USA, Canadian and Australian guidelines all suggest procedures for using historical data to improve flood peak-frequency relationships. Some subsequent researchers have criticized these procedures maintaining that they do not add further insight. However, still more recently other researchers have been studying the possibility of using pre-historical data such as tree rings and old sediment deposits to extend historical data so the position is still fluid. (See Stedinger and Baker, 1987).

What is your conclusion in this case?

Case study

14.7 VAAL RIVER @ RIVERTON : ADJUSTMENTS FOR UPSTREAM DEVELOPMENT

14.7.1 Problem definition

The gauging station at Riverton was established in 1908 in association with the water abstraction works to supply Kimberley. Over the years there has been substantial development within the catchment, and all of the major upstream rivers now have storage dams on them. For most of the year the flow in the Vaal River at this site now consists of controlled releases from Bloemhof Dam. The determination of the flood peak-frequency relationship at this site provides an interesting challenge with applications in most of our major rivers where upstream development has taken place.

14.7.2 Alternative solution methods

Probably the best method in this situation is to examine the flood peak-frequency relationship for the whole length of the Vaal River. This is the problem discussed in the next case study. The following are some alternative methods that could be applied to single site analyses.

14.7.3 Visual examination of the data

Run the program REGFLOOD and examine the tables of the recorded and ranked discharges. Is there a preponderance of high values at the beginning of the record and low values at the end of the record?

Examine the log normal plot. Is there a clear break point above which the data plot appears usable while showing distinct downward curvature below this value? If so, make use of the conditional probability approach described in Chapter 5.

14.7.4 Wakeby distribution

One of the claimed advantages of the five-parameter Wakeby distribution is that the upper end of the flood peak-frequency plot is independent of the effect of low outliers. Run the REGFLOOD program and study the results. Can the Wakeby distribution be applied to the whole record? If not, will the removal of low outliers help?

For each distribution, how many low outliers have to be removed before a good visual fit can be obtained? Do fewer low outliers have to be removed for the Wakeby distribution than for other distributions?

Is the Wakeby a noticeably better performer than the other distributions?

Case study

14.8 VAAL RIVER BETWEEN VAAL DAM AND THE ORANGE RIVER CONFLUENCE

14.8.1 Problem definition

Determine the flood peak-frequency relationship and the regional maximum flood (RMF) for the Vaal River between Vaal Dam and the confluence with the Orange River at Douglas. Assume that this information is required for determining 20- to 200-year flood lines, as well as for the possible construction of a system of weirs in the river.

This example has been chosen as it illustrates the need to look beyond the conventional flood frequency analyses in a complex system. The assumptions made in this analysis as well as the data on which the calculations are based may be challenged - as indeed they should be in the case of a thorough study - but the important point is that these interrelationships should not be overlooked.

14.8.2 Procedure

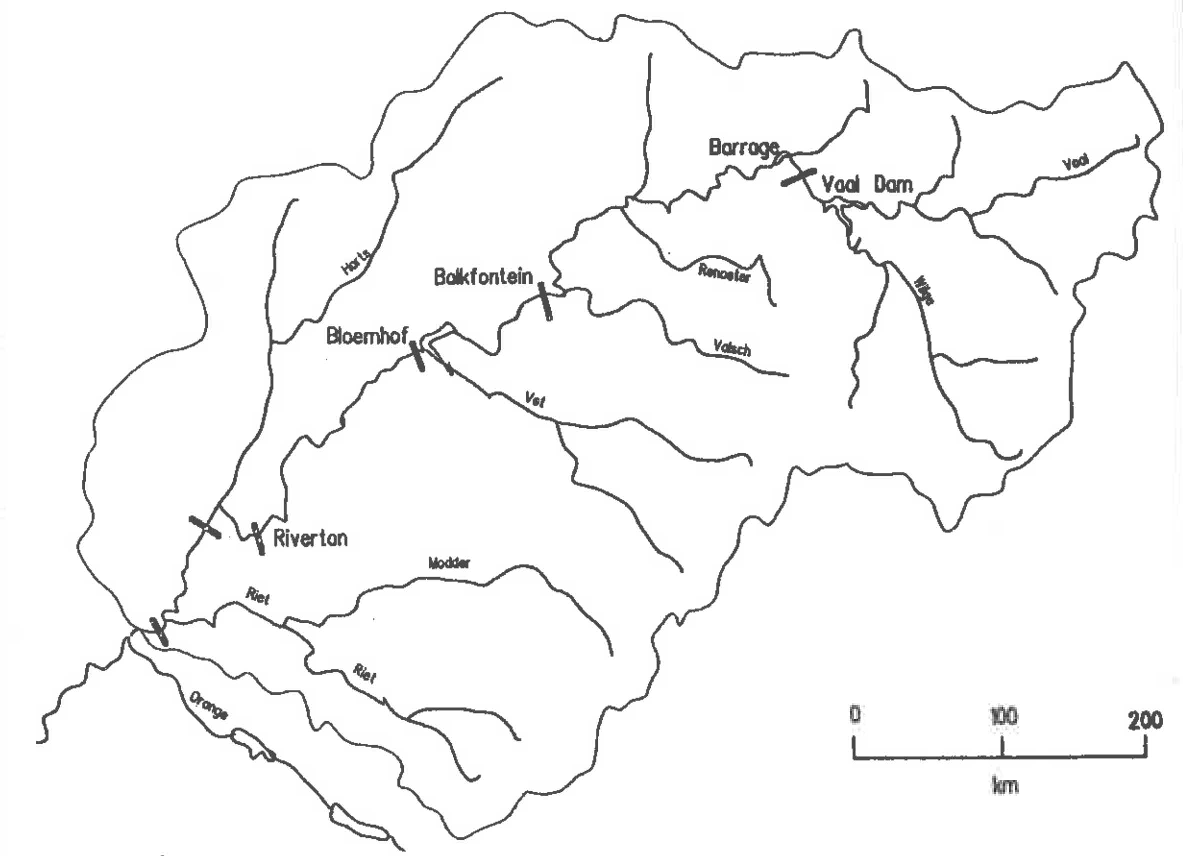
Kovåcs (1988) in the Department of Water Affairs' report TR137 recommends RMF *K*-factors of 4,6 at Vaal Dam, 4,0 from Vaal Dam to Bloemhof Dam, and 3,4 from Bloemhof Dam to the Orange River confluence. Note the reduction in the *K*-value in a downstream direction.

The reduction in peak flow per unit area of catchment (and therefore decrease in the *K*-factor) in a downstream direction along the Vaal River is a result of a decrease in catchment rainfall and an increase in the proportion of ineffective runoff areas (mostly enclosed drainage basins) in a downstream direction. These factors plus the effect of channel and flood plain storage on flood peak attenuation at high flows have a greater effect in reducing flood peaks in a downstream direction than the increases due to downstream tributary inflows.

However, this still postulates increases in the RMF itself within each *K*-region due to increase in catchment area, which in turn results in large discontinuities at the boundaries of the *K*-regions. This illustrates the need to interpret generalised relationships with care. The alternative adopted in this study is to select specific sites in distinct reaches of the river and assume a constant flood magnitude-frequency relationship for each reach based on the estimated relationships at the representative sites, rather than assume a continuous change in the relationship along the length of the river.

The river can be divided into six reaches :-

Reach	*Description*
1	Vaal Dam to Balkfontein weir (near Bothaville)
2	Balkfontein - Bloemhof Dam
3	Bloemhof - Riverton Weir (near Kimberley)
4	Riverton - Harts River confluence
5	Harts - Riet River confluence
6	Riet - Orange River confluence



The Vaal River catchment

The catchment areas in km^2 at the various sites are as follows:

Engelbrechtsdrif	38 600	
Vaal barrage	47 100	
Balkfontein	79 900	
Bloemhof	109 000	
Riverton	121 000	
Orange River confluence	194 000	(effective 140 000)

14.8.3 The design storm

In large catchments, the magnitude of the flood peak and the shape of the flood hydrograph depend on the location of the storm centre(s) within the catchment and the time intervals between discrete storm events which contribute towards the flood. For example the hydrograph of the February, 1975 flood in the Vaal Dam catchment which had a return period of just less than 50 years, (check this assumption), was the result of the cumulative effect of three successive but discrete storm events over a period of eleven days with periods of no rain between them, and with storm centres over different parts of the catchment. There was appreciable rainfall over the previous ten days and several minor floods had occurred earlier in the season.

The 1974 floods in the lower Vaal River and Orange River catchments were the consequence of unusual meteorological conditions which gave rise to exceptionally high rainfall over a large area which stretched from the Cape Midlands through to southern South West Africa, and which persisted for several months prior to the severe flood along the Orange River in March of that year. This flood produced the highest recorded flood level at Upington and had a return period of about 50 years. Similar floods occurred in the 1987/88 season.

It would therefore be unreasonable to assume a single design storm for generating long return period floods along the Vaal River downstream of Vaal Dam. However, there is no adequate meteorological data base which would allow the estimation of the possible time and space distribution of storm rainfall for long return period floods over the whole Vaal River catchment. For this reason it is doubtful whether reliable results could be obtained by using deterministic methods for flood frequency estimates.

14.8.4 Statistical analysis of historical floods

Because of the natural flood peak attenuating effect of the many large dams in the Vaal River catchment and overbank storage for flows with return periods exceeding 20 years, it would be more appropriate to use the maximum daily flows expressed in m^3/s as a unit rather than instantaneous peak flows. At Vaal Dam the instantaneous peak flows are approximately 15% higher than the maximum daily flows for return periods of up to about 50 years. It is anticipated that this difference will decrease with longer return period flows as well as in a downstream direction due to flood peak attenuation effects. For the reasons given in the chapter on flood routing it would not be reasonable to reduce these peaks any further to accommodate deliberate flood peak attenuation operation of Vaal Dam or Bloemhof Dam.

Good records are available at Engelbrechtsdrif/Vaal Dam since 1923. When plotted on log normal probability scales all the peaks lie comfortably within the 95% confidence band, with the highest ranked flood falling close to the fitted curve. The earlier flood peak data at

Vereeniging strengthen the confidence in the analysis based on the shorter record at Vaal Dam. The log normal distribution will provide reliable estimates for the maximum daily flow-frequency relationship at Vaal Dam.

14.8.5 Regional maximum flood

Fig 13.4 in chapter 13 shows the boundary between *K*-regions 4,6 and 4,0 crossing the Vaal River just downstream of Vaal Dam, and the boundary between *K*-regions 4,0 and 3,4 meeting the Vaal River upstream of the Vet River confluence ie upstream of Bloemhof Dam. Thereafter the river forms the boundary of these regions as far as Riverton. Between Riverton and the Riet River confluence the river is within the 3,4 *K*-region, and in the short reach between the Riet and Orange River confluences the *K*-value increases again to 4,0. The HRU Report 1/72 (Fig B1) divides the Vaal River into two separate regions just downstream of the Vet River confluence.

In a table in TR137 a *K*-value of 3,4 is recommended for the whole reach between Bloemhof Dam and the Orange River confluence.

The assumed constant RMF *K*-values for the six regions can be derived as follows.

A *K*-factor of 4,6 is reasonable for the Vaal Barrage catchment (area 47 100 km^2), and it can be assumed that the resulting RMF will be essentially constant within the reach from Vaal Dam to Balkfontein weir. The underlying assumption is that the flood peak attenuation by channel and flood plain storage is approximately equivalent to accretions from tributary inflows along this reach.

The flood peak attenuation will have a somewhat greater effect than tributary inflow between Balkfontein and Bloemhof, and the RMF should be reduced subjectively for this reach.

There are no major tributaries between Bloemhof Dam and the Harts River confluence and it can be assumed that the natural flood absorption in this reach would reduce the RMF peak further from Bloemhof Dam to Riverton, and a still further between Riverton and the Harts River confluence. This would be restored downstream of the Harts River confluence, and increased again downstream of the Riet River confluence.

Base the RMF for Reach 6 on the catchment area of the St. Claire diversion weir at Douglas, (193 800 km^2, K = 4,0).

This interpolation may seem to be somewhat arbitrary but it is well within the range of accuracy of any alternative methods of assessment.

A PMF of 25 000 m^3/s was used for the design of the raising of Vaal Dam. This dam is not only the most important dam in the country from a water supply point of view, but it is also close upstream of Vereeniging where the centre of the town and a large proportion of the residential area would be inundated by flows exceeding 15 000 m^3/s. From a structural safety point of view, the use of a very conservative PMF value is justified for Vaal Dam, but not for other structures along the Vaal River with the possible exception of earth fill dams with large storage capacities. Even in this situation the use of the Francou-Rodier equation with a more conservative *K*-factor would be more realistic than the PMF method.

14.8.6 Flood peak-frequency estimates

Hydrological records downstream of Vaal Dam cannot be used directly for flood frequency estimation because they are influenced by the upstream dams including Vaal Dam (see the Riverton case study above). It would be very difficult to quantify this influence and extrapolate it to long return period floods.

The normal procedure for long return period estimation in this situation is to ignore the effect of upstream dams and fall back on empirical-probabilistic methods where equations have the form :

$$Q_T = C.K_T.A^m \tag{1}$$

This method was proposed by Roberts in 1951 who used the Hazen distribution to determine *K(T)* and *C* was his catchment coefficient *'K'*. He found that the exponent *'m'* varied between 0,49 and 0,52 and he therefore recommended that a value of 0,5 be used for all cases. No attempt was made to regionalize the equation.

In a their report HRU 4/69 and 1/71 Pitman and Midgley adopted a similar approach. The formula that they recommended being :

$$Q_T = P \,.\, C^{0,2} \,.\, K_T \,.\, A^{0,6}$$

where P = Mean annual precipitation
C = An index related to the shape of the catchment
K_T was derived from the LEV1/MM distribution but also contained a regionalization adjustment.

Note that in this case the recommended exponent applied to the catchment area was 0,6 which was held constant while regionalization was achieved by varying K_T with veld zones and also by introducing mean annual precipitation into the equation.

In 1980 Kovâcs re-analysed all available flood maxima in South Africa and applied the empirical Francou-Rodier equation which has the form

$$Q_{max} = 10^6 . \left[\frac{A}{10^8} \right]^{1-0,1K}$$

This reduces to the following equations for the values of k recommended by Kovàcs in TR 137 :-

k-value	Q(max)
5,6	302 A ^{0,44}
5,4	209 A ^{0,46}
5,2	145 A ^{0,48}
5.0	100 A ^{0,50}
4,6	47,9 A ^{0,54}
4,0	15,9 A ^{0,60}
3,4	5,25 A ^{0,66}
2,8	1,74 A ^{0,72}

These equations are in the same form as equation (1) less the factor K_T which is related to the return period.

To summarize, three completely independent approaches have found that in South African rivers flood peaks increase with increase in catchment area raised to the following powers :-

Method	Range	Recommended
Roberts (1951)	0,49 - 0,52	0,5
Pitman and Midgley (1971)	-	0,6
Kovàcs (1988)	0,44 - 0,72	0,44 - 0,72
Adopted for the Vaal River	-	0,54 - 0,66

Of the three methods, that proposed by Kovàcs has a much larger data base and is also compatible with world-wide experience.

Therefore it would not be unreasonable to assume that the flood peaks for specified return periods along the Vaal River downstream of Vaal Dam bear the same relationship to the RMF for the reach as the relationship at Vaal Dam. The other assumption inherent in this calculation is that the coefficient of variation of the logarithms of the flows is constant along the river. This assumption applies to all of the regional statistical analysis methods discussed above.

14.8.7 Tabulate the results

Fill in the results in the table below.

Flood-frequency relationships for the Vaal river downstream of the Vaal Dam (Figures represent maximum daily flows expressed in m^3/s).					
Reach	Return period (years).				Regional Maximum Flood
	20	50	100	200	(RMF)
Vaal Dam					
1. Vaal Dam - Balkfontein weir					
2. Balkfontein - Bloemhof Dam					
3. Bloemhof - Riverton Weir					
4. Riverton - Harts River Confluence					
5. Harts - Riet River Confluence					
6. Riet - Orange River Confluence					

14.8.8 Flood hydrograph shape

Uncertainties regarding suitable storm rainfall estimates together with lack of information regarding the flood peak attenuation effect of the river channels, flood plains and storage dams in the catchment make it very difficult, if not impossible, to derive a reliable set of hydrograph shapes for long return period floods at various sites along the Vaal River using deterministic methods.

Using the February, 1975 flood as an example, this was the result of three discrete storm events, which produced two distinct peaks at Vaal Dam, but there was only a single peak in the Vereeniging area largely due to the way in which the flood was routed through the dam. The peak was further attenuated between Vereeniging and Bloemhof Dam. The actual 6-hour peak inflows into Vaal Dam were 3 818 m^3/s and 3 711 m^3/s spaced 48 hours apart with a minimum flow of 2 800 m^3/s between them.

The peak flow at Vereeniging was 3 840 m^3/s, and the 6-hour peak inflow into Bloemhof Dam was only 4 330 m^3/s despite substantial flows in downstream tributaries.

There is also the question of the use of flood hydrograph data for design purposes. It must always be borne in mind that in those dams where the maximum allowable water level at the dam wall is a constraint (i.e. all dams with fully gate-controlled spillways including Vaal Dam and Bloemhof Dam), the maximum flood peak attenuation is achieved by making full use of the storage in the dam up to this level. Should another major storm event occur in the area immediately upstream of the dam at a time when the water level in the dam is at or near this maximum, then the additional flood peak attenuation capability of the dam will be minimal. Long return period floods for large catchments may well be the result of two or more closely spaced storms and it would be unwise to rely on a significant flood absorption capability of any proposed dam in the Vaal River for the design of downstream dams.

Returning to the 1975 flood at Vaal Dam, the peak 6-hour inflow of 3 818 m³/s was the result of the cumulative effect of two successive but discrete storm events. The maximum discharge of 3 247 m³/s at that time (15% attenuation) would have been adequate to route the flood through the dam. When the third storm occurred the dam was already surcharged; the catchment was very wet; the rivers in the catchment were still full; there was an unfavourable weather forecast; and the Vereeniging municipality was pressing for an early decision on the proposed release from the dam so that they could organize the evacuation of threatened houses. Under the circumstances the discharge was increased to 3 664 m³/s which was only 2% less than the second peak of 3 711 m³/s. This is not an unusual set of circumstances, and it is quite possible that operational constraints and uncertainties regarding probable inflows may necessitate discharges which exceed peak inflows, particularly those which result from multiple storm events.

Post facto analyses of historical floods will provide hypothetical flood peak attenuations which are seldom achievable in practice.

It is therefore recommended that for design purposes :-

(a) Design flood hydrographs should be based on observed hydrograph shapes during the period of record, and that the discharge ordinates should be increased linearly with increase in the design discharge.

(b) The flood peak attenuation effects of upstream dams may be ignored. (Attenuation within the river channel is already accounted for in the derivation of the flood peak estimates).

(c) No allowance should be made for flood peak attenuation within the basin of the structure being designed.

14.8.9 Flood routing procedures

Although none of the existing dams in the Vaal River was specifically designed for flood absorption, they all have a limited flood absorption capability and this is used wherever possible.

Grootdraai Dam has some flood absorption potential which could be used to minimize the flow in the Standerton municipal area.

The Vereeniging municipal area downstream of Vaal Dam is particularly vulnerable to flooding, and the flood absorption storage that has been created by the raising of Vaal Dam will be used to minimize the peak flow along this stretch of the river. This means that the expected flows in the Klip and Suikerbos rivers which flow into the Vaal River upstream of Vereeniging must be included in the optimisation calculations.

In the case of the Orange River downstream of the Vaal-Orange confluence, it would be most unusual if the flows in the two rivers upstream of the confluence were such that optimum attenuation in the two rivers separately would produce optimum attenuation along the reach downstream of the confluence. In practice, if the minimization of the peak flow downstream of the confluence is paramount, then this will necessitate a greater than optimum discharge in one or both of the upstream rivers. The reason for this is that optimisation is achieved by separating the time of arrival of the two peaks at the confluence, and if the individual flood peaks are either advanced or delayed, then greater than the minimum possible discharges will result.

For example, during the 1976 floods Bloemhof Dam was operated in conjunction with the Hendrik Verwoerd Dam to minimize the flow in the Orange River downstream of the Vaal-Orange confluence. This was very successful. However, the PK le Roux Dam has since been completed and the combined flood absorption capability of the two Orange River dams is such that the additional assistance that can be provided by the Vaal River dams is minimal, and as downstream flood attenuation can be achieved only by greater than optimal discharges along the Vaal River it should not be attempted. This implies that the operating rules for the Vaal-Orange system as a whole should be as follows :-

(a) *Vaal River to Orange confluence*
Optimize flows, ignoring effect on lower Orange.

(b) *Orange downstream of Vaal confluence*
Optimize flows, taking the expected flow in the Vaal River into account.

(c) *Orange between PK le Roux Dam and Vaal confluence*
Greater than optimum flows will result from the above two priorities, but the flows will still be appreciably less than they would have been without the two Orange River dams.

14.8.10 Design considerations for additional structures

If another storage structure is built on the Vaal River its effect on the system will depend on its flood absorption capability. From a purely operational point of view, the following types of spillway are listed in order of preference :-

(i) Uncontrolled.

(ii) Uncontrolled section plus gated section, each capable of discharging 50% of the design flood.

(iii) Fully gate controlled, but with ample surcharge storage above full supply level (this requirement reduces the advantage of gated spillways).

(iv) A design which envisages full gate control but which has no allowance for surcharge above full supply level with any gate in the closed position, should not be considered.

The situation in (iv) above cannot be avoided in the case of barrages. Although surcharge storage could be incorporated in the design, this also means an increase in the volume of stored water that has to be discharged before all gates can be fully opened.

Unlike dams, all of which have some flow retardation capability, barrages will have the opposite effect i.e. stored water must be discharged through the gates prior to the arrival of the flood peak in order to change the water surface profile from a level pool to the slope appropriate for the flood discharge through the reservoir. The flood peak will then pass through the barrage neither retarded nor attenuated. The tail of the hydrograph will have to be chopped off to reinstate the storage.

The travel time of a flood wave through the system is proportional to the root of the depth of flow which is greater within the barrage than in the natural channel. Because of the operational requirements plus the more rapid flood wave movement, a barrage will have the effect of appreciably shortening the time of travel of the flood through the reach of river that it occupies. A barrage will therefore nullify the natural flood absorption capability of the reach of river occupied by the basin of the barrage. A series of barrages may accelerate the passage of the flood and increase the flood peak above what it would have been without the structures. The greater the volume of water stored within the barrages, the greater the risk of this occurring.

Another consequence of having to release stored water prior to the arrival of the flood peak is the shortening of the time available for warning property owners in downstream areas vulnerable to flooding. This then becomes a serious operational constraint.

Delays in discharging stored water increase the risk of inundating areas upstream of the barrage.

If barrages were to be built across the Vaal River it is most important that operational constraints be included in the design considerations. For example, economics may require fewer, more widely spaced barrages whereas operational considerations would favour smaller, more closely spaced structures. The effect of flood operational procedures on the location and levels of water intake works would also have to be considered. The installation of automatic, fail-safe flood gates would be strongly recommended.

Some of the difficulties can be overcome by extending the servitude areas upstream of the structure and taking out servitudes along the river downstream of the structure if it is anticipated that flows greater than (or in advance of) flows that would naturally occur are possible. In both cases a liberal allowance should be made for less than optimum gate operation procedures.

If not taken into account in the design of the structures, the factors mentioned above could result in the operators later being in a situation where they cannot avoid causing damage in upstream or downstream areas greater than would have occurred had the structures not been built.

14.8.11 Additional flood routing requirements

It was recommended above that the flood peak attenuation effect of upstream dams should be ignored in the design of downstream structures, the reason being that operational constraints and other factors may prevent the utilization of the full flood absorption potential of these dams, particularly during long return period floods. These considerations will vary from flood to flood.

It has been shown that barrages could advance the time of arrival as well as the magnitude of the flood peak.

The major requirement for flood routing is to ensure the safety of the dams and barrages. Once safety is assured, the next priority is to ensure that downstream damage is not greater than it would have been had the structures not been built.

Discharge optimisation must be within the above requirements and other constraints in effect at that time. Note that while optimisation is usually synonymous with peak discharge minimization this is not always the case, particularly along the lower reaches of the Vaal and Orange Rivers where a prolonged flood may cause more damage than a higher flood of shorter duration.

Consequently the possible loss of crops through prolonged inundation should be considered as well as the possible sloughing of river banks due to rapid decrease in flow.

14.8.12 The 1974 and 1975 floods

The 1974 and 1975 floods in the Orange and Vaal Rivers have been quoted several times in this study. This is because they had return periods within the range 20 to 50 years, i.e. they are the "once in a lifetime" events. They are good examples to use when testing operating procedures for new structures, but the following factors should be borne in mind.

During the 1974 and 1975 floods in the Orange and Vaal Rivers the floods came late in the season after several preceding minor floods. The hydrologists had time to sharpen their wits, develop computer programs and get observers out into the field. While this is the most likely situation for very large floods, it is floods in the 10 to 20-year return period range which are the ones that are more likely to occur. They can develop at any time within the flood season with little or no prior warning. Lack of information on upstream flows and general unpreparedness could easily result in non-damaging inflows being turned into damaging outflows. To minimize the possibility of this happening the designers in consultation with the hydrologists and system operators should make liberal allowances for communication, electrical, mechanical, *and human* failures in their designs.

It must always be borne in mind that the risk of damage due to equipment failures or other unavoidable causes during minor floods can be far greater than the risk of damage due to much larger floods with all systems operating. Murphy's Law is just as relevant to flood routing as it is to other activities in life.

Case study

14.9 LESOTHO : REGIONAL ANALYSIS

14.9.1 Problem definition

A flood peak frequency analysis is required for proposed dams in Lesotho. Twelve representative flow gauging stations have been selected. The lengths of record vary from 12 years to 21 years with an average of 16 years. These are short records and a wide variation in the statistical properties can be expected. Furthermore, due to the ruggedness of the terrain and consequent access difficulties natural control sections were used in most cases instead of fixed gauging structures as used in South Africa. The control sections were routinely calibrated.

The control section characteristics were subject to change after high floods, and in one case a large boulder obstructed the section making subsequent gaugings suspect. The calibrations for all stations were thoroughly examined and reviewed. One of the review calibrations resulted in the *Sigma Beta rating curves*. These rating curves were used to extract the annual maxima at each station.

The twelve data sets have been included in the program REGDATA

What is your best estimate of the Q-T relationship at each site based on a regional analysis, assuming that all the catchments are located within a hydrologically homogeneous region?

14.9.2 Procedure

Refer to the regional analysis procedure recommended in the next chapter and proceed as follows :-

1. Use the program REGDATA to create a data file with the name **LESOTHO.DTA** containing the twelve data sets.

2. Run the program REGFLOOD.

3. For the first trial run analyse one station only, do not send the results to the printer, but explore all of the options including the graphical displays.

4. When you are ready, proceed with the full analysis, preferably with a printer output as well as the screen dumps. This may take a number of hours.

14.9.3 Interpretation

This is not a trivial problem even for an experienced hydrologist. Anyone who intends undertaking regional analyses should spend several days on this data set as it highlights many of the problems that are likely to be encountered in practice.

The critical decisions that have to be made at each site are which other sites should be included in the analyses, what weight should be given to the regional parameters, and which distributions give the most satisfactory results.

The regional weighting is fixed in this program but can be changed by alterations to the program algorithms.

14.9.4 Botswana regional analyses

Carry out a similar study for the Botswana region using the data in the file REGDATA to create the data file BOTSWANA.DTA, and compare the results from these two vastly different hydrological regions.

Case study

14.10 UMGENI RIVER DAMS : FLOOD RISK ANALYSIS

14.10.1 Problem definition

The purpose of this case study is to demonstrate the applicability of all the recommended flood risk estimation methods in a situation requiring a high degree of judgment on the part of the analyst. It includes a situation which requires the recalibration of the unit hydrograph and rational methods.

The upper uMgeni catchment has been chosen as it has many of the characteristics and problems that will be encountered when carrying out flood risk analyses for dam safety evaluation and the estimation of the spillway design flood.

The spelling of the word uMgeni follows that shown on the maps published by the Trigonometrical Survey.

A : DIRECT STATISTICAL ANALYSIS METHODS

14.10.2 Available records

The flow gauging station U2M01 on the uMgeni River at Howick was established in May 1901 making it the second oldest flow gauging station in South Africa. (The oldest is station C2M10 on the Vaal River at Vereeniging which was established a year earlier but closed in 1922). The station was damaged by floods in 1909 and then closed. The weir was rebuilt in December, 1948. Between 1948 and 1956 the station had a limited gauging capacity and most of the annual maxima exceeded the calibration limit during this period. It is only since 1956 that reasonably reliable records are available. These are in the form of daily observations up to October, 1961 when a continuous water level recorder was installed. It is likely that many of the maxima derived from daily observations underestimate the flood peaks which probably occurred between observations.

The gauging station U2M02 was established on the uMgeni River at Inanda Location in January 1936 and was operated by the Durban Municipality. It consisted of a gauging weir with a limited capacity, gauge plates in the river channel and an automatic recorder since 1968. The station was closed in 1975 when a new station U2M15 was built at the site. This station had a limited gauging capacity and no maximum flow data are available for the period 1975 to 1987. The recorder was washed away during the 1987 floods, but the flood peak was estimated to be between 5 100 and 5 500 m^3/s from subsequent slope-area calculations.

Station U2M03 on the uMgeni River at Richmond has a very limited gauging capacity and is unusable for flood analysis.

Station U2M04 on the uMgeni River at the Nagle Dam site was established by the Durban Municipality in 1931 but closed in 1939 when the dam was built.

Station U2M05 was established by the Durban Municipality upstream of Nagle Dam in November, 1950. Good peak flow estimates are available and have been brought up to date to include the September, 1987 peak.

Station U2M06 on the Karkloof River was established in 1954, but has too many gaps in the record to be useful for this analysis.

Station U2M07 was established in 1954 on the Mpofana River at Weltevreden which is upstream of Midmar Dam. While there are some uncertainties in the record it is sufficiently complete to be considered as a candidate for this analysis.

Stations U2M08, -09, -10 and -11 are small catchments in the Henley Dam area and were not considered suitable for this analysis. The same applied to station U2M12 on the Sterk River.

Station U2M13 in the uMgeni River at Petrus Stroom upstream of Midmar Dam has a limited gauging capacity which makes it unsuitable for this analysis.

Station U2M14 was established in 1964 in the uMgeni River at Albert Falls immediately downstream of the future Albert Falls dam site. Although this station has a limited gauging capacity, the Department of Water Affairs has recently re-worked the data and flood peak maxima are available up to and including the September, 1987 flood.

If the two oldest stations on the uMgeni River - those at Howick (since 1901) and Inanda Location (since 1935) had been in operation and provided reliable annual flood peak maxima since their first establishment, the estimation of flood risks at the present uMgeni River dams would have been straight forward. As it is, the only reliable records within the uMgeni River catchment that are suitable for flood risk analyses are:

U2M01	uMgeni River at Howick	(1956 - 1987)
U2M05	uMgeni River at Nagle Dam	(1950 - 1987)
U2M07	Mpofana River at Weltevreden	(1955 - 1987)
U2M14	uMgeni River at Albert Falls	(1964 - 1987)

The Howick station is downstream of Midmar Dam, the Albert Falls and Nagle stations are both downstream of Midmar Dam and the Albert Falls dam. The effects of these upstream dams, and the method used to allow for them must be addressed.

The short length of the reliable records at these stations makes flood risk analysis, particularly estimates of severe floods suspect unless confirmation of their acceptability can be obtained from a wider study.

14.10.3 Wider investigation.

Natal rivers are not well endowed with flow gauging stations capable of registering high flows. The main reason for this is that the principal interest in the past was the low flows that could be used for direct abstraction rather than in the future construction of large dams. A further complication is that a number of gauging stations were destroyed by the September, 1987 floods which were the most severe on record in a number of rivers. The 1959 floods in southern Natal, the 1987 floods in central Natal, and the 1984 floods further to the north caused by the tropical cyclone Domoina necessitated a closer look at the frequency of floods of this magnitude in Natal in general, including the uMgeni River. The Division of Hydrology of the Department of Water Affairs has already published reports on the Domoina floods in northern Natal in 1984, and 1987 floods in central Natal. Preliminary estimates of annual flood peaks in the affected gauging stations were obtained from the Department and have been included in a data file on the disk.

The first step in the regional analysis is the identification of candidate gauging stations in the T, U, V, W and X drainage regions which extend from south of the Natal-Transkei border to north of the Natal-Transvaal border. The selection of the representative stations in these regions to be included in the data set was based on the availability of data in the publications of the Department of Water Affairs, the length of these records and the absence of appreciable gaps in the records.

In many cases the peak flows exceeded the capacity of the gauging stations. In those situations where the maximum recorded discharge for an extreme flood could be calculated using slope-area methods, the stage-discharge relationships for smaller peaks were derived by linear interpolation beyond the calibration range on a log-log plot. Where no high flood values could be calculated, linear extrapolation on a log-log plot was used to estimate the flood peaks. These approximations are adequate for the purpose of this study.

Thirty four candidate stations were identified: seven in the T-region; nine in the U-region which includes the uMgeni River catchment; eight in the V-region which includes the Tugela River basin; nine in the W-region further to the north; and one in the X-region in the Komati River system.

Because of the inherent difficulties in obtaining reliable estimates of high flows, the accuracy of the results varied from station to station. The second step in the regional analysis was therefore the elimination of stations where the gauged results were suspect. There are several indicators that can be used for this purpose, the most useful being a visual examination of the plotted results. Another indicator of anomalies is whether or not a Wakeby distribution can be fitted to the data.

Data files for these regions are included on the disk.

Six of the stations can be eliminated at this stage, two in the U-region and four in the W-region, leaving seven, seven, eight, five and one station in the T, U, V, W and X regions respectively. Do you agree?

14.10.4 Frequency distributions

The most likely candidate distributions are:

Code	Distribution	Number of parameters	Moment estimators
LN/MM	Log normal	2	conventional
LP3/MM	Log Pearson 3	3	conventional
GEV/PWM	General extreme value	3	probability weighted
WAK/PWM	Wakeby	5	probability weighted

The LN/MM distribution is the simplest and oldest of the distributions and is still widely used in practice throughout the world. The LP3/MM distribution using **regionally** derived skewness coefficients is the recommended method in the USA and Australia, while the GEV distribution using maximum likelihood estimators is recommended in the comprehensive study undertaken in the United Kingdom. The GEV distribution with probability weighted moments (the GEV/PWM distribution) is a candidate alternative method for the GEV/ML.

The WAK/PWM distribution is a newcomer and is not yet in general practice. As it has five parameters it is more flexible than the other distributions, but the accuracy of the estimates of the values of these parameters depends on the number of stations used in the analysis. Is there significant difference between the results using the GEV/PWM and WAK/PWM distributions in this region?

14.10.5 Results of the regional analyses.

The main purpose of regional analyses is to identify hydrologically homogeneous regions, and then derive regional weights to be applied to the moments of the distributions at the stations of interest.

One of the difficulties in carrying out direct statistical analyses is the effect of upstream utilization on the flood peaks. In years of low rainfall, upstream storage dams, often including a multitude of farm dams, and in the case of the uMgeni River system, major storage dams, will have an appreciable effect of reducing the recorded flood peak at a downstream gauging station. This effect will decrease in years of high rainfall, and will be minimal in the case of extreme floods.

These low outliers have a significant effect on the skewness of a data set. Although mathematical methods for identifying outliers are available, hydrological data sets are far too short for the adoption of such procedures with confidence.

In this study graphical plots of all 34 stations used in the analysis should be examined; the low outliers which will have an undesirable effect on the upper end of the fitted curves should be identified visually; and these can then be excluded from the subsequent statistical analysis.

Run the program REGFLOOD for each region and compare the statistics and regionally derived moments for the selected stations in regions T, U, V and W respectively.

The main interest at this stage of the analysis is to compare the statistical properties of the uMgeni River data sets with those in the rest of Natal. For the purpose of comparison identify values exceeding the following thresholds in the tables:-

Raw data	$C_V > 2{,}0$
	$g > 4{,}0$
Log data	$C_V > 0{,}25$
	$g > 1{,}3$

The coefficient of variation (C_V) is a measure of the variability of the data about the mean value. This is the slope of the plot on the graphs.

The skewness coefficient g is a measure of the skewness of the distribution of the data. This is the curvature of the plotted values on the graph. Zero skewness of the logarithms (the plotted points lie on a straight line) indicates that a log normal distribution would be appropriate. The log normal fit is a useful reference line when interpreting the results.

The stations in the uMgeni River catchment (stations with the prefix U2) show anomalously high C_V and g values when compared with the other rivers in Natal, particularly when compared with the statistics of the stations in the immediately adjacent T-region which is the Tugela River catchment. It is significant that the only station in the T-region with an abnormally high skewness is station V2M02 on the Mooi River which is in the same headwater region as the upper uMgeni River.

It is also highly significant for the purposes of this analysis that the gauging station at Nagle Dam on the uMgeni River downstream of the Midmar and Albert Falls dams exhibits the highest skewness coefficient of all of the stations analysed in this study.

The average skewness of the log transformed data in the different regions is of interest. The tables show that the average for the U-region is 0,94 compared with the lower values of 0,69 (T-region), 0,42 (Tugela basin), and 0,82 in the W-region which includes stations which registered floods caused by the tropical cyclone Domoina. (You may get different values depending on which stations you included, and the number of low outliers excluded in each station.)

14.10.6 Interpretation

If stations outside the uMgeni River system had exhibited statistical properties similar to those of the uMgeni stations then they could have been combined with the uMgeni stations to derive more reliable estimates of the regional parameters.

In this case the uMgeni values are higher than those in the other rivers in Natal. On the basis of available evidence it is not possible to determine with a reasonable degree of confidence whether the uMgeni anomalies are the result of one or more of the following:-

1. The effect of upstream dams including farm dams. (Very likely, but probably not the sole reason).
2. The effect of catchment utilization, particularly afforestation.
3. Differences in flood producing catchment characteristics including catchment topography. (Very likely as the results of the deterministic methods indicate).
4. Differences in the frequency and magnitude of severe storms. (The uMgeni is located in the geographic centre of the combined effect of the 1959, 1984 and 1987 storms).
5. Chance differences in the frequency of severe storms. (Cannot be evaluated).

14.10.7 Narrower regional analysis

A decision has to be made on whether the uMgeni results are anomalous and therefore more weight should be given to stations in a wider region, or whether to use the uMgeni stations only in a regional analysis. The higher the values of CV and g, the larger the estimated flood for a given risk. As the use of a larger number of stations will reduce the regional weightings and consequently flood peaks, it is recommended that a more conservative approach be adopted and the full regional analysis to limited to the following stations:-

U2M01	uMgeni at Howick
U2M05	uMgeni at Nagle Dam
U2M07	Mpofana at Weltevreden
U2M14	uMgeni at Albert Falls
U4M02	Mvoti at Mistley.
V2M02	Mooi at Mooi River

These stations are included in the data file MIDMAR.DTA. Run the program REGFLOOD with this data set and determine the best flood peak-frequency relationship for Midmar Dam (uMgeni River at Howick) and Albert Falls Dam.

14.10.8 Discussion of the results.

Enter the results in the table below to illustrate the differences in the results obtained when using the regionally weighted moments, compared with those based on single station moments.

Comparison of estimates of the 100 year return period flood peak (1% annual exceedance probability). All values are in m³/s.			
	LP3/MM	GEV/PWM	WAK/PWM
Howick	(a) (b)		
Albert Falls	(a) (b)		

(a) Based on station moments.
(b) Based on regionally weighted station moments.

A discussion of the reliability of the direct statistical analysis based on regionally weighted station moments is deferred until the other methods of analysis have been presented.

B : DETERMINISTIC METHODS

14.10.9 Introduction

The 1959 floods in southern Natal prompted the establishment of the University of the Witwatersrand's Hydrological Research Unit (HRU). Since then there have been two extreme flood-producing storms. A severe test of the 20-year old HRU methods is to compare their results with those of more recent methods when applied to the uMgeni catchments.

14.10.10 Calculations

Tabulate the results of calculations for the original and alternative rational and unit hydrographs for all of the Natal stations included in the program DETFLOOD Enter the estimates for the 100-year return period flood at the two dams using the deterministic methods and their alternative algorithms in the table below :-

Dam	Rational		Unit hydrograph	
	original	alternative	original	alternative
Midmar				
Albert Falls				

The results using the deterministic methods are appreciably higher than those derived from the direct statistical analyses methods. Further comparison and evaluation is deferred until later in this study.

C ESTIMATES OF THE MAXIMUM FLOOD

14.10.11 Probable maximum flood

The only information on the PMP estimation in South Africa is that provided in HRU Report 1/72. This is an upper envelope value of observed rainfall maxima in South Africa. This is used in the unit hydrograph method to estimate the probable maximum flood.

14.10.12 Regional maximum flood

The *K*-factors recommended in TR137 for the uMgeni River are 5,2 for Midmar and Albert Falls dams and 5,4 for Nagle and Inanda dams. (See Fig 13.4 in Chapter 13).

14.10.13 Results

The results of these analyses are discussed later in this study.

D FLOOD HYDROGRAPHS

14.10.14 Observed hydrographs of the 1987 floods

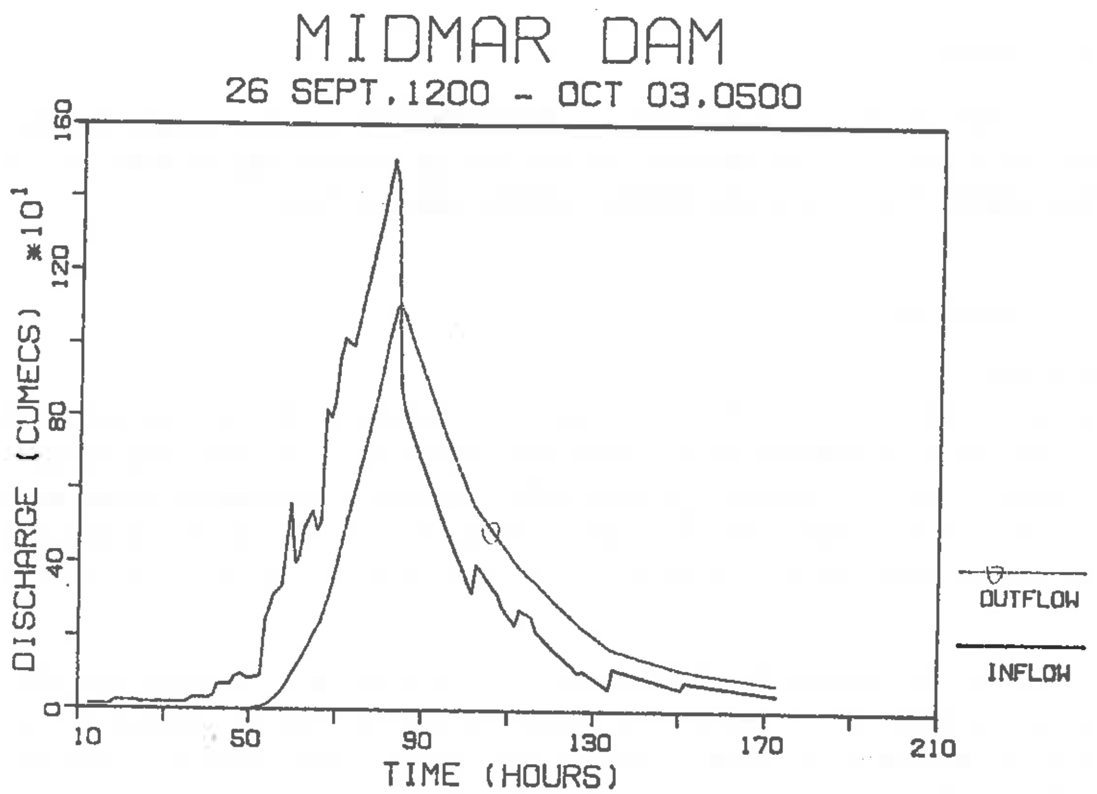
The inflow and outflow hydrographs of the 1987 flood peaks at Midmar and Albert Falls dams were supplied by the Division of Hydrology of the Department of Water Affairs and are reproduced below.

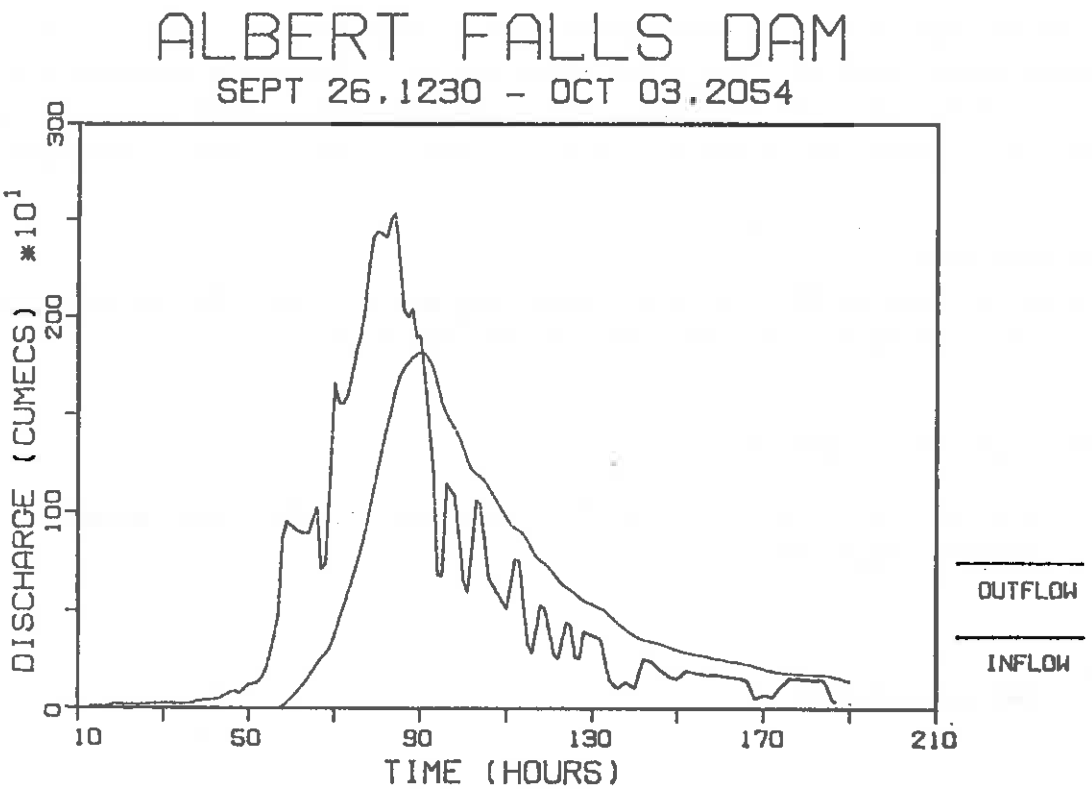
The durations of the rising and falling limbs of the 1987 flood hydrograph at Albert Falls Dam were 40 hours and 70 hours, and the total duration was 110 hours. The flood peak of 2 500 m^3/s is equivalent to a 50-year return period flood, derived from the deterministic methods.

Compare these with the hydrographs generated by the unit hydrograph method for various return periods. Note that these hydrographs have much shorter times to peak than the observed hydrograph.

It is also interesting to compare the critical storm durations. Compare the time of concentration for the rational method with the critical storm durations for the two implementations of the unit hydrograph method. The actual time to peak was 40 hours!

Once again this demonstrates that it is the storms with durations considerably longer than the catchment response time which cause the extreme floods and not the shorter more intense rainfall producing storms with durations equal to the catchment response time.


MIDMAR DAM
26 SEPT.1200 - OCT 03.0500
DISCHARGE (CUMECS) *10^1
0
40
80
120
160
10
50
90
130
170
210
TIME (HOURS)
OUTFLOW
INFLOW


ALBERT FALLS DAM
SEPT 26.1230 - OCT 03.2054
DISCHARGE (CUMECS) *10^1
0
100
200
300
10
50
90
130
170
210
TIME (HOURS)
OUTFLOW
INFLOW

E : COMPARISON OF THE RESULTS

14.10.15 Graphs

Use the graph option #8 in the program REGFLOOD to produce graphs for the two uMgeni River dams. Plot the results of the deterministic methods and the RMF calculations on these graphs. Study the graphs together with the comment below.

14.10.16 Interpretation

Midmar Dam
The gauging station U2M01 at Howick is located downstream of Midmar Dam and the flood peaks since the construction of the dam have been influenced by the dam. The record is not long enough to allow two separate statistical analyses relating to the position before and after the construction of the dam. The seven lowest values, six of which are obvious low outliers, can be omitted from the analysis to provide a rough correction for the effect of Midmar Dam.

When running the program REGFLOOD you had the opportunity to assess the effect of using regional parameters to improve the estimate at Howick. Did this resulted in a poorer visual fit of the data at this station? What is the reason for this? Does this mean that the regionally weighted parameters are less reliable than the parameters of the station alone? Do any of the fitted curves accommodate the exceptional 1987 flood peak? Does this imply that the 1987 flood has a return period of between 200 and 500 years?

The original algorithms of the deterministic methods all give higher values than the direct statistical analysis methods. (This is not always the case). Unlike the different statistical analysis methods the results tend to converge rather than diverge with increase in return period. What is the return period of the 1987 peak based on the deterministic methods?

Albert Falls Dam
The results are very similar to those at Midmar Dam/Howick, except that the 1987 peak has a lower return period than the peak at Midmar Dam for all methods.

14.10.17 Inter-site comparison

The results at both dams are consistent from site to site. This strengthens the recommendations which follow.

F : RECOMMENDATIONS

14.10.18 Return period flood peaks

Enter your recommended values for the annual flood peak maxima at the two dams in the following tables:-

MIDMAR DAM : RECOMMENDED FLOOD PEAK MAXIMA		
Return period years	Flood peak m^3/s	Derivation method
2		Unit hydrograph-alternative implementation
5		- do -
10		- do -
20		- do -
50		- do -
100		- do -
200		??????

ALBERT FALLS DAM : RECOMMENDED FLOOD PEAK MAXIMA		
Return period years	Flood peak m^3/s	Derivation method
2		Unit hydrograph-alternative implementation
5		- do -
10		- do -
20		- do -
50		- do -
100		- do -
200		??????

14.10.19 Design maximum flood

Tabulate the RMF and PMF for the two dams and compare these with South African and world maxima.

Refer to the SANCOLD guidelines on safety in relation to floods referred to in Chapter 12 and determine the recommended minimum design flood and the safety evaluation flood.

The possibility of the breaching of Midmar Dam on the safety evaluation flood at Albert Falls Dam during the same flood has to be considered. What would be the effect of the additional volume of water discharged into the river and the time and rate of breaching during a flood of this magnitude be on the discharge into downstream dams? Would it be reasonable to assume that the peak of the flow through a breach at Midmar Dam would coincide with the peak of the flood as it moved down the river? Would it be reasonable to assume that both of these dams might breach concurrently with an extreme flood? What would the cascade effect of multiple breaching be on the propagation of a flood through all the major dams in the uMgeni River?

Dam break hydraulic routing programs are available. These will provide information on river discharges resulting from dam breaks, but these have to be superimposed on realistic estimates of the flood hydrographs.

What is your estimate of the magnitude of the safety evaluation flood that includes provision for the effect of breaching of one or more of the upstream dams during the flood?

14.10.20 Design flood hydrograph

Enter the values that you recommended for use in flood hydrograph estimation at Midmar Dam in the table below. It should have a much longer time base than that determined from the unit hydrograph method, and should accommodate the situation that can be expected in an extreme flood where heavy antecedant rainfall and consequently high river flows can be expected prior to the rainfall which directly results in the flood peak.

Provide the flows as percentages of the peak flow as a linear relationship with flood magnitude may be assumed. This is a reasonable assumption for design and dam safety evaluation purposes, as the actual hydrograph shapes can be expected to vary widely from flood to flood.

MIDMAR DAM DESIGN HYDROGRAPH			
Time (hours)	**Discharge** (% of peak)	**Time** (hours)	**Discharge** (% of peak)
0	0.0	105	
10		110	
15		115	
20		120	
25		125	
30		130	
35		135	
40		140	
45		145	
50		150	
55		155	
60		160	
65		165	
70		170	
75		175	
80		180	
85		185	
90		190	
95		195	
100		200	

14.10.21 Calibration

The original algorithms of the rational and unit hydrograph methods were never calibrated against *the statistical properties* of gauged records, apparently because of the lack of confidence in direct statistical analyses. Users of the suite of computer programs are now in a position to investigate the relative reliability of the methods recommended in this handbook when applied to the uMgeni River and the adjacent catchments. The programs REGDATA and DETFLOOD include data for common gauging stations.

Start with REGFLOOD and carry out a regional analysis on five of the stations in the data file MIDMAR.DTA, leaving out the station on the uMvoti River at Mistley which will be used to test the conclusions. Dump the composite log normal graphs for each of the five stations after having determined the best regional relationships.

Run the program DETFLOOD and do the full calculations for the same stations (menu numbers 2 to 6), including the original and alternative implementations of the rational and unit hydrograph methods. Plot the results on the screen dumps. For each of the five stations determine the correction factor that has to be applied to the alternative implementation of each deterministic method for the 2 year and 100 year return periods. Calculate the *average* correction factor for each method and return period (four values in all). Now recalculate the flood peak-frequency relationship for the six stations using the calibration factors in the alternative implementations of the rational and unit hydrograph methods. Are you satisfied that the calibrated deterministic methods can be applied with confidence at ungauged sites?

To test your conclusion continue with DETFLOOD and use the original plus calibrated alternative methods to calculate the flood peak-frequency relationship for the uMvoti River at Mistley. Return to REGFLOOD and calculate this relationship for the station on its own ie without the benefit of regional weighting. Compare the results graphically. Do you agree that calibrated deterministic methods are significantly better than the original implementations when applied to ungauged sites within a region containing gauged records that can be used for calibration?

14.10.22 Acknowledgments

All the data used in this study were supplied by the Division of Hydrology of the Department of Water Affairs as were the 1987 flood inflow and outflow hydrographs at Midmar and Albert Falls dams, and the latest estimates of the K-factors for the regional maximum flood method. Without all of this information it would not have been possible to produce a meaningful study of the flood risks at these two dams.

The solution to this difficult problem requires an understanding of most of the procedures given in this handbook and is an appropriate ending to the case studies.