Water Research Commission

University of the Witwatersrand
Johannesburg

Design Flood Determination in SWA - Namibia

W V Pitman and J A Stern

Report No. 14/81
Hydrological Research Unit
1981

WATER RESEARCH COMMISSION

UNIVERSITY OF THE WITWATERSRAND

JOHANNESBURG

DESIGN FLOOD DETERMINATION IN SWA-NAMIBIA

W V Pitman and J A Stern

REPORT NO 14/81

Hydrological Research Unit
University of the Witwatersrand
JOHANNESBURG 2001.

December 1981.

ISBN 0 85494 710 8

DESIGN FLOOD DETERMINATION IN SWA-NAMIBIA

PREFACE

Even when Report 1/72 was updated in 1979, we still had not enough information to extend the Floods Manual into the arid regions. In this, the last report in the HRU numbered series, the short-coming has, we hope, been rectified, thanks to the co-operation of officials at the Directorate of Water Affairs and Weather Office, Windhoek.

As the data were sparse, broad generalizations had to be made. These may have been too sweeping and the findings will probably have to be modified as fresh data become available. Although the relevant rainfall and design storm data from HRU reports 3/79 and 2/80 have, for convenience, been incorporated in this manual, the reader should make himself familiar with those reports.

I trust that the two worked examples will prove helpful. The opportunity has been taken to incorporate additional guidance in estimating the value of C in the Rational Formula.

The work was undertaken in part fulfilment of a contract between the University and the Water Research Commission and my grateful thanks go to the Commission for generous financial aid to the Unit, apart from the contract funds, as also to the CSIR and the University Council. Finally, I thank the members of my staff and our co-workers from various consulting firms for their loyal service to the Unit.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

CHAPTER 1 INTRODUCTION

In 1969 the Hydrological Research Unit published its Design Flood Manual (Report no. 4/69). It was felt at the time, however, that the manual did not offer adequate guidance for design flood determination in the arid areas - in particular SWA-Namibia. The metricated version of the Design Flood Manual - Report no. 1/72 - updated in 1979 (grey cover), with revised depth-duration-frequency diagram - still did not offer adequate design information for the dry regions.

When, therefore, the opportunity arose for the Unit to acquire all the daily rainfall data for SWA-Namibia it was grasped in the hope that analysis of this material would yield an improved basis for arid zone design flood and storm determinations.

The first study, comprising an analysis of point rainfall, resulted in HRU Report no. 3/79 - Analysis of SWA-Namibia rainfall data. That report contains an isohyetal map and a coaxial diagram from which, given the mean annual precipitation at a problem point, the maximum depth of precipitation of given duration appropriate to a given frequency of occurrence can be estimated. The depth-duration-frequency relationship is relevant to design flood determinations for small catchments.

For large catchments depth-area-duration-frequency relationships are needed for different regions of the country and, to develop these, storms have to be studied on an areal basis. Report no. 2/80 - Analysis of large-area storms in SWA/Namibia - provides the necessary design storm data for estimating floods for large catchments.

In 1980 authority was received for members of the Unit to collect streamflow records from the Directorate of Water Affairs, Windhoek. Hydrographs of major floods were abstracted from the records at about twenty of the most reliable gauging weirs in the country, together with flood peak data from all rated gauging sites.

Although the streamflow data are rather sparse and records generally too short for reliable statistical analysis, it has nevertheless been possible to provide information for design flood determination over a large part of the country. (Much of SWA-Namibia is covered by loose superficial deposits in which the runoff from all but the severest of storms is completely absorbed).

1.1 Format of the Report

Chapter 2 describes how the study was conducted, e.g. selection and preparation of the basic data, statistical analysis, hydrograph analysis, investigation of rainfall-runoff relationships, determination of average storm losses, identification of homogeneous regions, generalization of characteristic diagrams and so forth.

Chapter 3 is aimed at those using the report as a design flood manual. The reader is taken step by step through a couple of worked examples chosen with an eye to illustrating most of the problems that may be encountered in practice.

Appendix A contains all the information, in the form of maps, diagrams and tables, required for design flood determination. Repeated here for the sake of convenience are the maps and diagrams initially presented in Reports 3/79 and 2/80 for the purpose of design storm determination. However, it is recommended that the reader study those two reports before proceeding to apply the material transposed.

Appendix B is devoted to a description of the computer program written to transform a stage hydrograph (in digital form) into a discharge hydrograph and to calculate the flood volume. The program also plots the stage- and discharge hydrographs on a CALCOMP plotter.

1.2 A Warning

Generalisations are often said to be dangerous and as the results of this study are presented largely in the form of regional generalisations it follows that there lurk some dangers in uninformed application of the results to specific designs. It must be emphasized that, in order to arrive at some of the design graphs in Appendix A, fairly liberal extrapolation of relationships based on rather meagre samples was obligatory. It goes without saying that as fresh data come to hand need may be found to adjust the results.

Finally, one should avoid presenting a final design of the flooding aspects of an important project without having referred to local flow data (if any) and without having inspected the catchment and the relevant gauging stations.

CHAPTER 2 METHODOLOGY

2.1 Introduction

There are two main approaches to design flood determination; choice depends upon the circumstances, e.g. the accuracy desired and the limitations inherent in the basic data. The first approach is statistical - an ordering and transposition of past experience. The second is deterministic - an attempt to determine result from cause. Deterministic methods are therefore founded on the statistics of the causative events (storm rainfall) rather than on a ranking of the experiences that result - i.e. the floods.

The main purpose of this chapter is to explain how the various design graphs in Appendix A were derived using both statistical and deterministic methods.

2.2 Precipitation

Records from 572 daily-read rain gauges, stored on magnetic tape, comprised the basic data-set for both the point rainfall study (Report no. 3/79) and the analysis of large area storms (Report no. 2/80). In addition to these daily data, information from nine autographic recording stations was employed in the point rainfall analysis to enable daily rainfall extremes to be disaggregated to shorter-duration values.

Since the analyses of the storm data are described in Reports 3/79 and 2/80 only the main results are reproduced here in Appendix A.

2.2.1 Point rainfall : small-area storms

Figure A1 is an isohyetal map of SWA-Namibia that permits mean annual precipitation (MAP) to be estimated at any location in the country. Once the MAP of a problem site has been established the depth of point rainfall likely to be equalled or exceeded in return periods up to 100 years, for durations of 15 minutes to 5 days, can be abstracted from the co-axial diagram, Figure A2.

2.2.2 Large-area storms

Scrutiny of the results of depth-area-duration analyses of 36 major large-area storms suggested that SWA-Namibia could be sub-divided into three meteorologically homogeneous regions, viz. North, South and Coastal, as shown in Figure A3. For the North and the South regions co-axial diagrams have been produced to define the inter-relationships of depth, area, duration and return period. These diagrams appear as Figure A4 (North region) and Figure A5 (South region). For the coastal region, which is very sparsely endowed with raingauges, resort must be had to Figure A6 from which the areal reduction factor (ARF) can be estimated. The ARF is then applied to point rainfalls derived from Figure A2.

2.2.3 Medium-area storms

A storm that covers an area greater than a few square kilometres but extends over an area significantly smaller than that of the large-area storm region in which it falls may be considered to be a medium-area storm. Most design problems will fall into this medium-area category.

Application of an ARF to point rainfalls may be expected to yield the most accurate results for small-area storm design. Conversely, application of the relevant depth-area-duration-frequency diagram would probably yield the best estimation of storm rainfall over a relatively large area. By means of Figure A7 the designer may weight the results derived from the two different methods. (However, in the Coastal region one would have no option but to refer to Figures A2 and A6).

The final diagram in this series of aids to storm design - Figure A8 - provides a basis for the disaggregation of one-day rainfalls to short-duration values for storms designed according to the large-area approach, i.e. via Figures A4 and A5.

2.3 Streamflow

Streamgauging stations throughout SWA-Namibia are maintained by, and the records processed by, the Directorate of Water Affairs in Windhoek. Observations are made at about 60 stations but not all records were suitable for analysis in this floods survey. Many of the stations are merely rated (or, in some cases, unrated) sections in the natural river channels. Since it had not yet been possible, up to the time of writing, to have the stage-discharge ratings checked by in situ current meter measurements, selection was confined to those stations equipped with gauging weir and automatic recorder.

Although practically all records are short, (the longest not more than 20 years), it was nevertheless possible to select 22 stations that were well-distributed throughout the territory and had records of the order of ten years or longer. The stations are listed in Table 2.1 and plotted on the map, Figure 2.1.

Selection of major flood events, for subsequent analysis of hydrograph shape and volume (see later), was facilitated by inspection of the monthly summary sheets for each gauging station. Months yielding very high volumes of runoff were first identified. It was then a relatively simple matter to scan each of those months of the record to identify the actual flood hydrographs. The summary sheets were also the source of data on maximum peak discharge in each year (required for flood frequency analysis), and on maximum recorded discharges. Flow data were abstracted in the form of stage hydrographs which were converted to discharge hydrographs by means of a computer-program (see Appendix B) with the aid of the relevant rating table. The program plots both stage and discharge hydrographs and also calculates the volume of each individual flood (this for the purpose of establishing rainfall-runoff relationships).

2.4 Experience envelopes

Estimation of floods associated with very long return periods, merely by extrapolation of probability distributions

Table 2.1 Details of streamflow gauges

Station reference number	Station name	River	Co-ordinates Lat. (S)	Long. (E)	Catchment area (km^2)	Length of record (yrs)	Stat. analysis	Hydro-graph analysis
0482M01	Tsamab	Ham	28°09'	19°15'	2300	10	√	√
0483M01	Norechab	Hom	28°07'	18°38'	5750	10	√	√
0491M01	Gras	Fish	24°11'	17°21'	9360	9		√
0493M01	Rietkuil	Hutup	25°07'	17°31'	5800	4		√
0496M01	Seeheim	Fish	26°49'	17°48'	46750	17	√	√
0497M02	Aikanes	Löwen	26°53'	18°06'	7760	8		√
2531M01	Ousema	Omuramba	21°12'	17°08'	4660	17	√	√
2931M01	Khowarib	Hoanib	19°16'	13°53'	7800	11	√	
2961M01	Petersburg	Ugab	20°12'	16°08'	2115	16	√	√
2962M03	Vingerklip	Ugab	20°24'	15°26'	10280	12	√	√
2971M01	Omaruru	Omaruru	21°26'	15°57'	3100	13	√	
2971M02	Etemba	Omaruru	21°25'	15°38'	3700	12	√	√
2972M01	Henties Monument	Omaruru	21°56'	14°23'	13300	13		√
2981M01	Swakophöhe	Swakop	21°57'	17°01'	2660	10	√	√
2984M01	Westfalenhof	Swakop	22°17'	16°25'	9520	16	√	√
2986M01	Ameib	Khan	21°50'	15°38'	5200	11	√	√
2991M01	Schlesien	Kuiseb	23°17'	15°48'	6580	17	√	√
3111M02	Mentz	Black Nossob	23°06'	18°42'	8400	5		√
3112M01	Otjivero	White Nossob	22°17'	17°58'	1790	6		√
3112M02	Amasib	White Nossob	23°05'	18°39'	7000	2		√
3121M01	Rehoboth	Haris	23°20'	17°03'	2400	7		√
3122M01	Nauaspoort	Usib	23°04'	17°12'	640	10	√	√

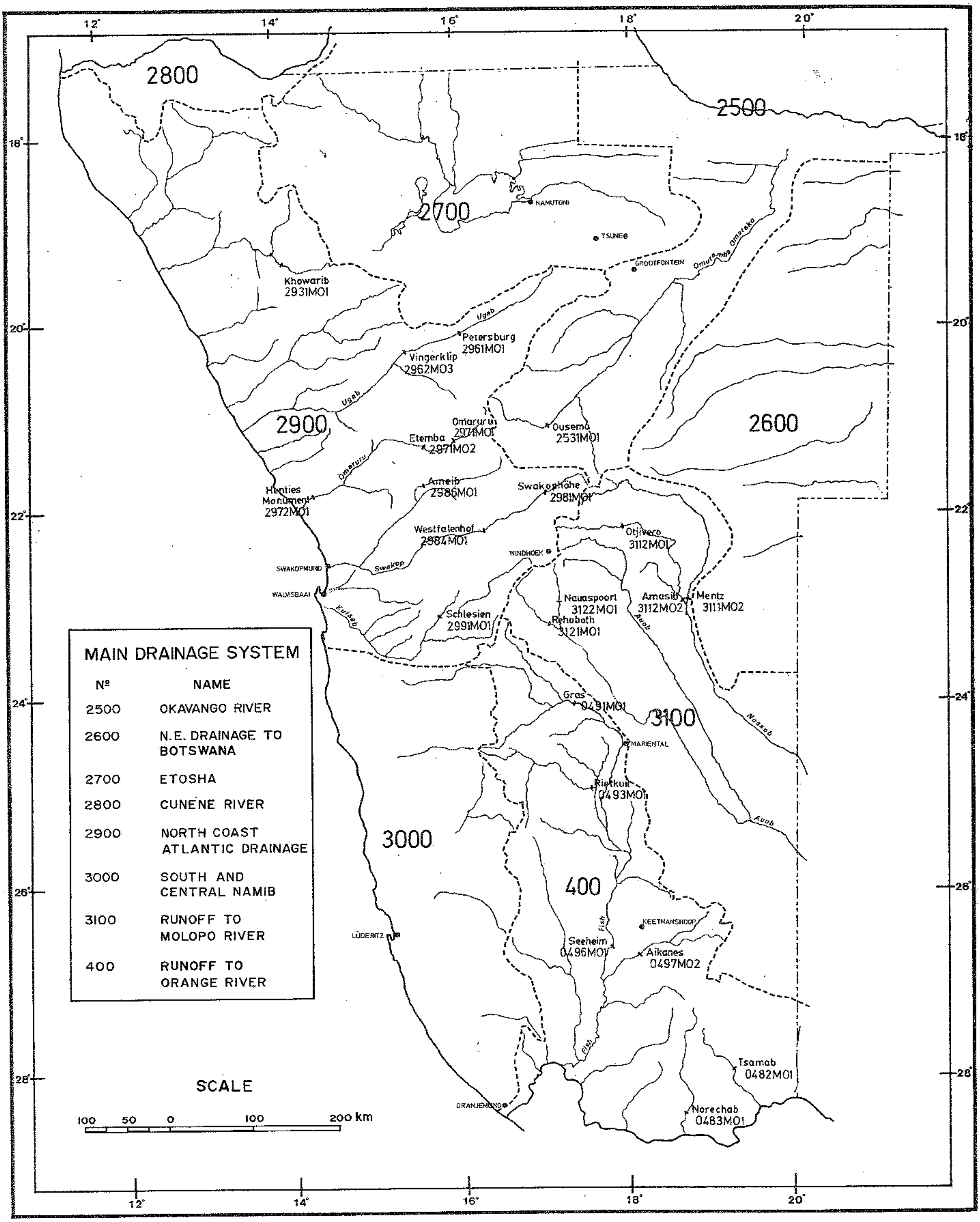

Figure 2.1 Map of SWA-Namibia showing location of stream gauges

that have been fitted to relatively short records is generally considered to be most unreliable. Various probability distributions, fitted to relatively long records (say $\pm$ 50 years) can range widely - for example by a factor of five if extrapolated to say the 10 000-year event. Where records are short, as is the case for SWA-Namibia, extrapolation even to the 100-year event can be highly inaccurate. For estimating the likely magnitudes of rare events resort must be had to experience envelopes.

2.4.1 Creager envelopes

Figure A9 is a peak flood experience diagram on which the highest peak discharges to be found in the records for SWA-Namibia have been plotted against catchment area. Approximate positions of the gauging points can be identified by the code numbers that refer to the drainage subdivisions shown in Fig. 2.1.

The envelope curves drawn on the diagram are those proposed by Creager (1964). Creager ratings of flood peaks have become familiar among engineers. No particular significance should be attached, however, to the formula (see eqn. 2.1); it merely represented an acceptable shape to the envelope of Creager's data collected from many parts of the world.

Creager's formula was $$Q = 46CA^{0,894A^{-0,048}} \qquad (2.1)$$

in which Q = flood peak in ft^3/s

A = catchment area in square miles

and C is the Creager rating.

The data have been converted to metric system for plotting on Figure A9 which serves as a useful rough guide to general flood experience in SWA-Namibia. Since, of the numerous factors that are known to affect flood runoff, only catchment area and general geographical location have been taken into account, the diagram can offer little more than a preliminary estimate of the possible range of extremes at a problem point, with practically no clue as to frequency of occurrence. Nevertheless, the practical engineer rightly draws comfort from a knowledge of what has already been experienced in a region of interest and will not lightly adopt, for a major structure, a design flood lower than that given by the upper envelope of locally experienced peak discharges.

The maximum Creager rating observed in SWA-Namibia as a whole is seen to be slightly less than 30, which is less than half that of the envelopes of South Africa's floods derived from a considerably larger and longer data base (see HRU 1/1972). It is a matter for conjecture whether the lower SWA-Namibia rating means that that country has a lower flood potential than the rest of Southern Africa or merely that the data base is so much smaller.

2.4.2 Francou-Rodier envelopes

Francou and Rodier (1967) compiled a catalogue of 1200 maximum recorded flood peaks representative of all regions of the world. When plotted against corresponding catchment areas on a log-log scale they found that for hydrologically homogeneous regions the envelopes representing regional upper bounds were straight and converged towards a single point lying at approximately $A = 10^8 km^2$ and $Q = 10^6 m^3/s$.

The equation given by Francou and Rodier for these converging envelopes can be written as:

$$Q = 10^6 \; (A.10^{-8})^{1-0,1K} \qquad (2.2)$$

in which Q = flood peak (m^3/s)
A = catchment area (km^2)
and K is a regional coefficient.

They found that practically all 1200 points lay within an upper envelope curve of $K = 6$ and a lower envelope curve of $K = 0$.

Recently Kovacs (1980) established envelope curves for five regions in South Africa, based on the Francou-Rodier equation. Values of K for the five regions varied from a maximum of 5,25 for the south-eastern coastal region to a minimum of 2,5 for the flat, desert areas of the north-western interior.

The envelopes have been superposed on the flood peak experience diagram for SWA-Namibia, Figure A10. The resulting Francou-Rodier envelope has a K value of 4,3, which is fairly close to the value of 4,6 suggested by Kovacs for the central plateau region of South Africa.

It should be stressed that experience envelopes are merely indicative of past events that have been recorded and, consequently, do not represent physical upper bounds to the flood regime of a region. As time goes by and more and more data are assembled these curves are bound to shift upwards. The catastrophic flood at Laingsburg in January 1981, for instance, seems to have exceeded the appropriate envelope based on the Francou-Rodier formula. In correspondence associated with compilation of the report of the ICOLD Committee on Hydraulics of Dams, of which South Africa is an active member, the French Committee objected strongly to any notion that the Francou-Rodier envelopes might be indicative of probable maximum flood (PMF).

2.5 Statistical analysis of flood peaks

The purpose of statistical analysis is to derive a relationship between flood magnitude and return period. The return period is the average interval between years during which floods exceeding a given magnitude were experienced and is therefore the reciprocal of the probability that a flood exceeding that magnitude will occur in any one year.

The relationship between flood peak Q and return period T may be derived from two alternative series of flood peaks. The partial duration series, or series of peaks above a given threshold, comprises all flood peaks that exceed that threshold value. It is not generally used because of the difficulty of deciding whether peaks that occurred close together were mutually independent. The annual maximum series, usually preferred by the engineer, is the series comprising the highest flood peaks in each (hydrological) year. Although the return period T deduced from the annual maxima series differs from that deduced from the partial duration series, because some of the higher floods may not have been the highest of the year, the difference in T for long return periods is only about 0,5 year.

The aim of statistical analysis is to reveal the relationship between Q and p, expressed as a distribution function. The form of the distribution depends on the type of series and cannot be deduced from theoretical reasoning unsupported by empirical values. An annual maxima series may be ranked from largest to smallest and then paired against plotting positions that are related to both frequency and return period. Values of Q are then plotted against plotting positions on a probability paper appropriate to the chosen distribution function. A curve is fitted to the points so that flood peaks for any return period can be read. A disadvantage is that the resulting curve would probably be drawn differently by each different analyst. An alternative method, numerical estimation, relies on selection of a distribution function on the basis of experience; its parameters are derived from the observed data by applying analytical rules.

To overcome the necessarily large errors in estimation of Q for a return period T, i.e. Q(T), from short records, the method of regionalisation of the flood peak data has been adopted . In this method each series is rendered dimensionless by dividing the Q values by their mean $\bar{Q}$ (or by Q(T'), where T' is a preselected return period close to that of the mean). The relationship between $Q(T)/\bar{Q}$ and T is then estimated from the mean pattern of the individual probability plots.

Choice of an appropriate statistical distribution for the SWA-Namibia data was limited to Gumbel, log-Gumbel or log-normal.For each of the 14 stations considered suitable for frequency analysis (see Table 2.1) all three distributions were fitted to the ranked flood peaks by the graphical method described above. The choice of distribution that best fitted the data for each station was based on visual inspection of the three sets of plots. Although no one particular distribution yielded the best fit in all cases the log-normal fared best overall and was accordingly adopted for the purpose of establishing a regional frequency curve for SWA-Namibia.

Figure 2.2 illustrates an example of the log-normal distribution as fitted to the data for the Omuramba river at Ousema.

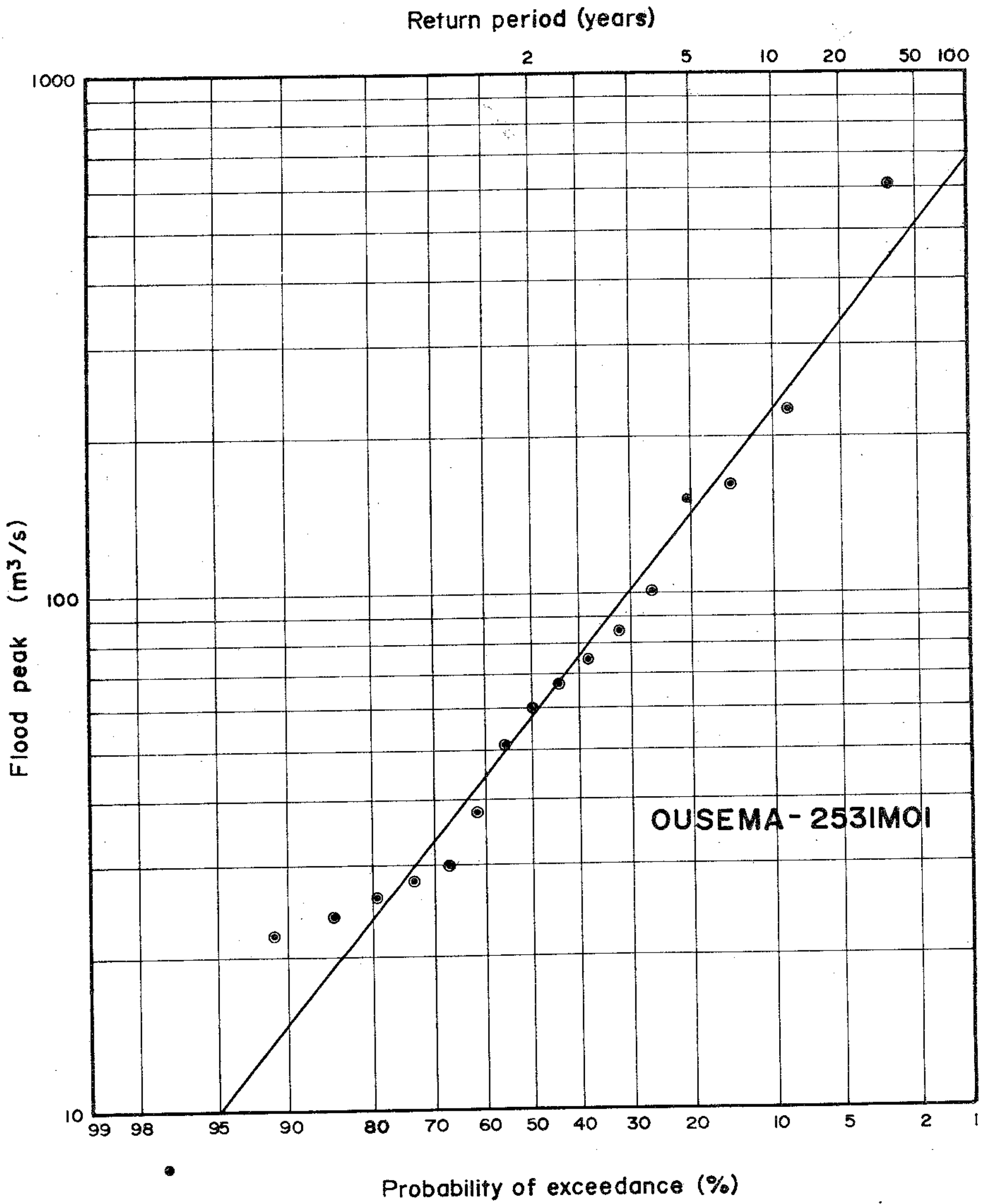

Figure 2.2 Frequency analysis of flood peaks recorded at Ousema

Plotting positions were calculated according to the Hazen formula, viz:

$$p = \frac{1}{T} = \frac{2m-1}{2n} \qquad (2.3)$$

where m is the rank in descending order
and n is the number of years of record.

In the normal distribution the mean is equal to the median, i.e. the value of the variate that is equalled or exceeded 50% of the time, or the two-year event of an annual series. Therefore, the flood frequency curves for all the individual records were rendered dimensionless for the purpose of comparison by dividing the values by Q(2).

The dimensionless frequency curves are drawn on Figure 2.3. The large sampling errors associated with such short records resulted in considerable scatter and consequently it was not possible to identify regional dissimilarities among groups of catchments in different localities. Accordingly, the average frequency curve, shown on Figure 2.3, computed for all stations was assumed to apply to SWA-Namibia as a whole.

Comparison of this curve with the seven regional flood frequency curves derived for South Africa (Pitman and Midgley, 1967) indicates that the SWA-Namibia curve is similar to that for region 6 - the central and northern areas of Transvaal. It would appear, therefore, that the regional flood frequency curve proposed for SWA-Namibia, although based on such limited data, is entirely plausible.

Since an estimate of the 2-year flood, Q(2), enables one to dimensionalise the flood frequency curve for a particular catchment, Q(2) was plotted against catchment area. This simple approach did not prove successful, however, (a) because of the large scatter that resulted and (b) because nearly all the gauging stations commanded catchments that were within the relatively narrow range 2000 - 10 000 km^2. Accordingly, it was decided rather to employ the dimensionless frequency curve for the purpose of estimating the relationship between return period and storm losses; this approach is discussed in section 2.8.

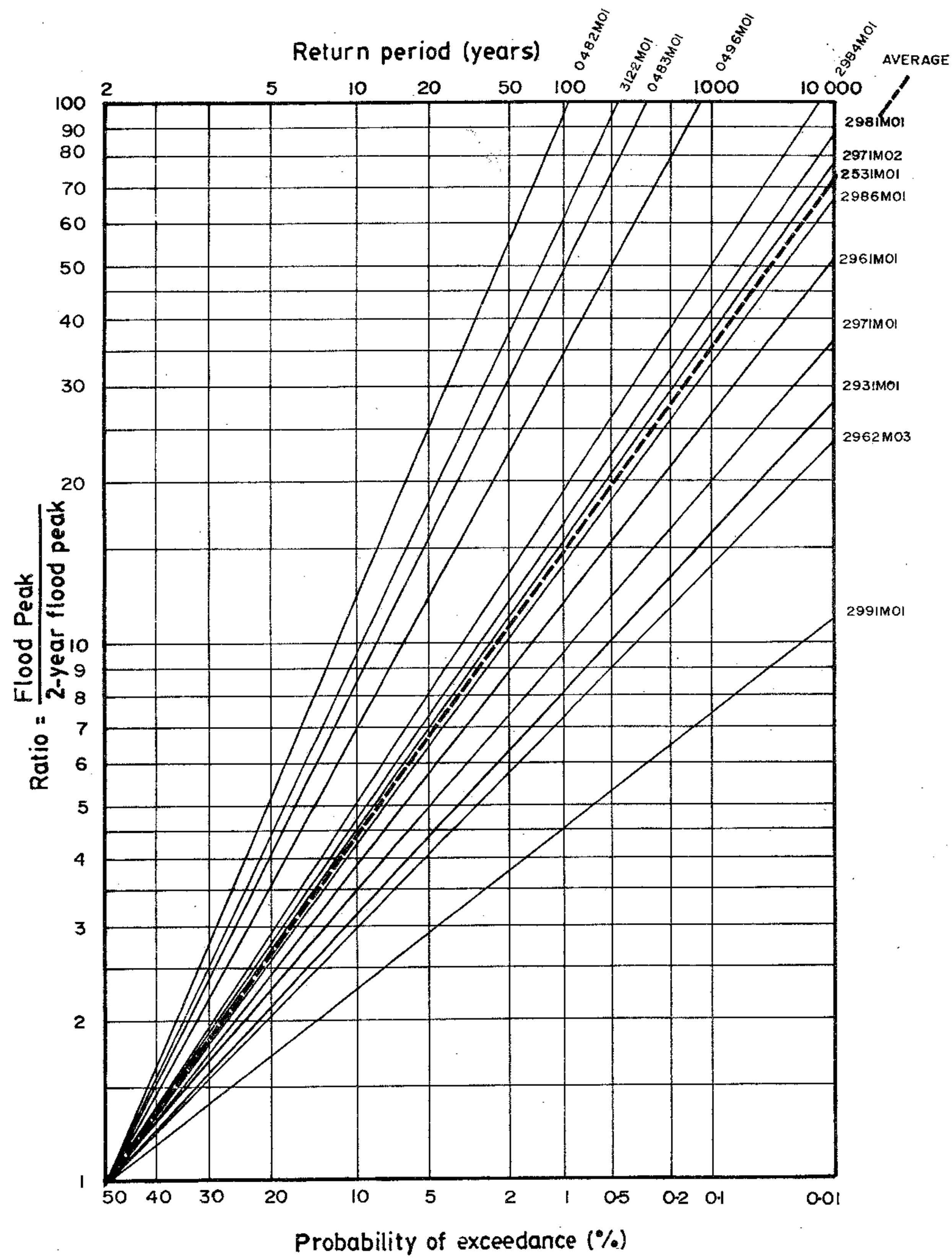

Figure 2.3 Derivation of regional flood-frequency curve

2.6 Hydrograph analysis

In addition to methods for estimating peak flood discharges, procedures are required whereby the shape and volume of a flood hydrograph can be derived. The complete flood hydrograph would be needed for routing through a reservoir, or where an estimate of the flood resulting from a specific storm is required.

The unit hydrograph technique is perhaps the most widely used for synthesizing flood hydrographs. One of the main assumptions is that causative rainfall is uniformly distributed over the catchment and occurs at a constant uniform rate. Examination of the SWA-Namibia flood hydrographs, however, revealed in general marked dissimilarities among individual events owing to non-uniformity of rainfall distribution, with different parts of the catchment responding for different storms. Consequently, the unit hydrograph approach had to be abandoned.

Attention was then turned to the possibility of synthesizing hydrographs by routing excess rain (i.e. that portion of the total storm rainfall that becomes direct runoff) through a single reservoir-type storage. This method was applied successfully to catchments in South Africa by Bauer and Midgley (1974).

Routing of excess rain through storage is achieved by application of the Muskingum procedure as modified by Nash (1959). The Muskingum equation is:

$$O_2 = C_o I_2 + C_1 I_1 + C_2 O_1 \qquad (2.4)$$

in which I is the rate of inflow to a channel reach,
O is the rate of outflow, and the subscripts 1 and 2 refer to the beginning and end of a time step respectively.

The coefficients C_o, C_1 and C_2 as derived by Nash are:

$$C_o = -\frac{K}{\Delta t}(1 - C_2) + 1 \qquad (2.5)$$

$$C_1 = \frac{K}{\Delta t}(1 - C_2) - C_2 \tag{2.6}$$

$$C_2 = e^{-\frac{\Delta t}{K(1-x)}} \tag{2.7}$$

in which K is the routing constant,
Δt is the time step and
x is a weighting factor ($0 \leq x \leq 0,5$)

If one assumes reservoir-type storage then the weighting factor x becomes zero and equation 2.7 simplifies to :-

$$C_2 = e^{-\frac{\Delta t}{K}} \tag{2.8}$$

Once the time interval, Δt, for step-wise solution of the Muskingum equation (2.4) has been chosen the timing and shape of the outflow hydrograph will be determined by the value of K, the routing constant. Inspection of equation 2.4 reveals that when $I_1 = I_2 =$ zero, the value of O_2 is given by the simple expression:

$$O_2 = C_2 O_1 \tag{2.9}$$

Taken in the context of an excess rainfall routing, this situation applies after the storm has ceased, i.e. during the period of hydrograph recession. Hence it is possible to estimate the value of K by examination of the hydrograph recession, as shown hereunder.

Substituting for C_2 (equation 2.8) in equation 2.9 yields:

$$O_2 = O_1 \, e^{-\frac{\Delta t}{K}}$$

$$\text{or } O_1/O_2 = e^{\frac{\Delta t}{K}}$$

$$\ln O_1 - \ln O_2 = \Delta t/K$$

$$\text{i.e.} \quad K = \frac{\Delta t}{\ln O_1 - \ln O_2} \tag{2.10}$$

In other words, K is the reciprocal of the slope of the hydrograph recession limb when discharge is plotted to a logarithmic scale. Accordingly, flood hydrographs were plotted on semi-log paper and straight lines fitted to that portion of the recession limb considered to be attributable to direct runoff. Figure 2.4 depicts a number of hydrographs, for the streamflow gauge at Schlesien, plotted on semi-log paper for the purpose of estimating the routing constant, K, while Table 2.2 lists K values computed for the recorded flood hydrographs at each gauge.

The data in Table 2.2 indicate a fairly wide range of K for hydrographs of a particular catchment. This phenomenon is considered to be due to flood events caused by the responses of varying proportions of the catchment for different storms. In other words, a hydrograph associated with a low value of K (steep recession) is assumed to relate to an event for which a small proportion of the catchment yielded direct runoff, and vice versa. The validity of this assumption is borne out by the fact that K is basically a function of the natural storage in a catchment. The K value should therefore depend primarily on the area of catchment, or, more specifically, the area of catchment that yields direct runoff. The maximum observed K for a catchment should therefore most likely reflect the condition of complete catchment response.

Accordingly, the maximum observed values of K were plotted against catchment area (on log-log paper). A straight line fitted to the data points yielded the following equation:

$$K = 0{,}2A^{0{,}42} \qquad (2.11)$$

The best-fit line is shown together with the data points on Figure All. It is interesting to note that, within the size range of the gauged catchments, the proposed K-area relationship is in fairly close agreement with the relationships derived by Bauer and Midgley for both their "False Karoo" zone of the north-eastern Cape Province and their "Bushveld" zone of the central and northern Transvaal.

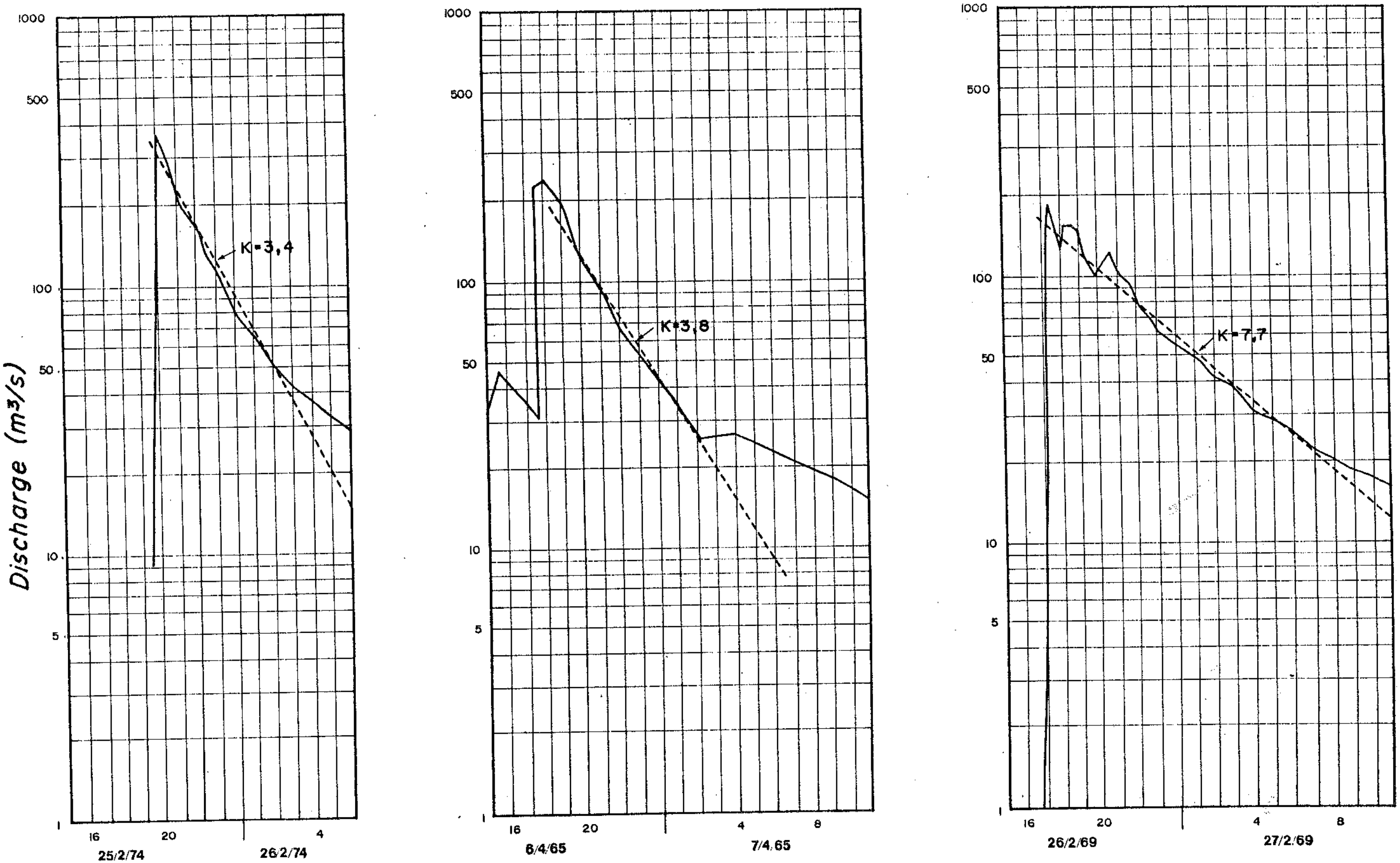

Figure 2.4a Hydrographs recorded at Schlesien- plotted on semi-logarithmic paper

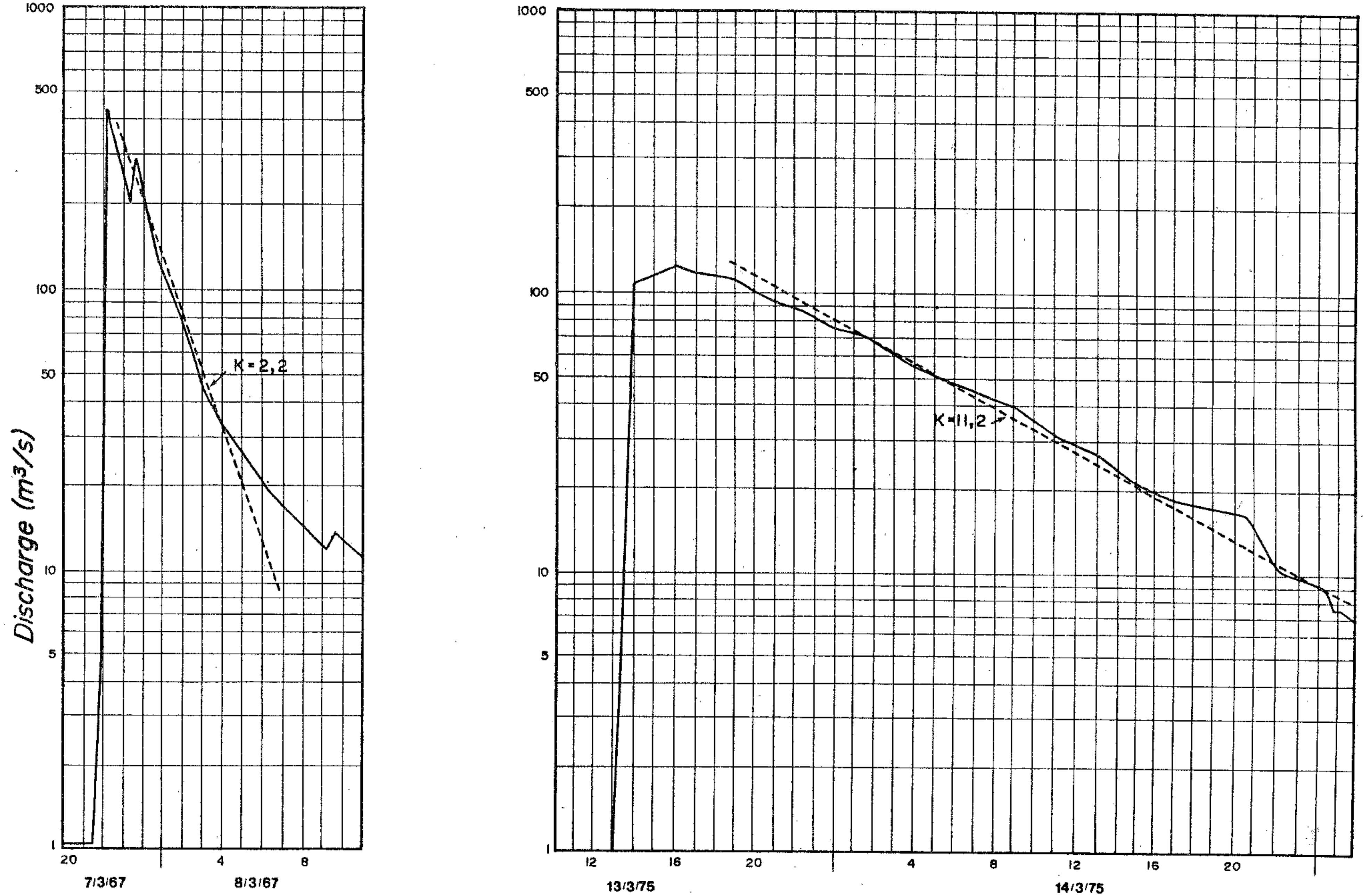

Figure 2.4b Hydrographs recorded at Schlesien - plotted on semi-logarithmic paper

Table 2.2 Estimation of routing constant, K

Station Ref. No.	Station Name	Date of Storm	O_1	O_2	Δt (hrs)	$K = \frac{\Delta t}{\ln O_1 - \ln O_2}$
0482M01	Tsamab	29/03/77 - 31/03/77	150	1.2	16	/3.3/
		18/04/73 - 18/04/73	580	6	8	1.8
		19/01/74 - 19/01/74	400	8.5	7	1.9
		21/02/74 - 22/02/74	1400	22	13	3.2
		12/03/80 - 13/03/80	230	2	15	3.2
0483M01	Norechab	02/04/71 - 03/04/71	85	2.5	4	1.2
		20/01/74 - 21/01/74	54	5.5	6	2.7
		07/03/76 - 08/03/76	32	2.4	10	3.9
		03/12/76 - 07/12/76	110	31	8	6.3
		03/12/76 - 07/12/76	72	34	10	/13.3/
0491M01	Gras	07/02/71 - 11/02/71	800	300	7	7.1
		07/02/71 - 11/02/71	600	130	14	9.2
		07/02/71 - 11/02/71	140	40	12	/9.6/
		27/01/74 - 30/01/74	250	50	8	5.0
		26/02/74 - 02/03/74	360	50	11	5.6
		31/03/76 - 04/04/76	650	70	17	7.6
		30/03/78 - 01/04/78	520	22	17	5.4
		23/02/79 - 26/02/79	700	29	21	6.6
0493M01	Rietkuil	20/02/74 - 21/02/74	340	14	14	4.4
		01/03/76 - 04/03/76	400	90	28	/18.8/
0496M01	Seeheim	05/04/61 - 09/04/61	4600	2400	10	/15.4/
		17/03/72 - 21/03/72	4200	1600	6	6.2
		21/02/74 - 24/02/74	3400	1200	13	12.5
0497M02	Aikanes	19/04/73 - 22/04/73	520	32	20	7.2
		15/01/74 - 16/01/74	400	34	7	2.9
		11/03/75 - 12/03/75	220	38	13	/7.4/
		21/01/76 - 24/01/76	340	44	9	4.4
2531M01	Ousema	28/01/74 - 31/01/74	300	3	20	4.4
		01/03/74 - 04/03/74	380	3.8	19	4.1
		04/02/76 - 07/02/76	360	6.5	10	2.5
		22/01/78 - 26/01/78	340	5.6	20	/4.9/

Table 2.2 - cont.

Station Ref. No.	Station Name	Date of storm	O_1	O_2	Δt (hrs)	$K = \frac{\Delta t}{\ln O_1 - \ln O_2}$
2961M01	Petersburg	11/02/62 - 13/02/62	120	5	12	3.8
		04/02/70 - 06/02/70	95	5.4	11	/3.9/
		08/01/74 - 09/01/74	58	2.2	10	3.1
2962M03	Vingerklip	24/03/69 - 27/08/69	160	5.4	11	3.3
		10/02/71 - 12/02/71	210	16	15	3.0
		14/03/74 - 15/03/74	270	3.8	21	4.9
		03/03/75 - 04/03/75	140	7.5	15	/5.1/
		04/04/76 - 05/04/76	190	5.8	4	1.2
2971M02	Etemba	15/11/67 - 17/11/67	150	9	7	2.5
		14/02/71 - 16/02/71	560	18	8	2.4
		14/03/72 - 14/03/72	460	20	10	3.2
		27/01/74 - 29/01/74	900	25	12	/3.4/
		27/02/74 - 28/02/74	750	27	10	3.0
		04/02/76 - 05/02/76	500	12	5	1.4
2972M01	Henties Monument	28/01/74 - 29/01/74	520	85	18	9.9
		25/02/74 - 28/02/74	400	15	39	/11.9/
2981M01	Swakophöhe	06/02/71 - 07/02/71	1000	25	18	/4.9/
		06/04/72 - 06/04/72	540	9	11	2.7
		18/03/73 - 20/03/73	560	2.9	15	2.9
		30/01/74 - 31/01/74	700	12	11	2.7
		08/03/75 - 08/03/75	54	4.4	4	1.6
		16/02/79 - 17/02/79	700	5.8	6	1.3
2984M01	Westfalen-hof	25/03/65 - 28/03/65	95	11	16	7.4
		03/02/66 - 07/02/66	90	22	14	/9.9/
		03/02/66 - 07/02/66	70	7	9	3.9
		14/02/67 - 16/02/67	54	7	13	6.4
		13/02/71 - 14/02/71	230	44	7	4.3
		28/03/72 - 29/03/72	300	30	7	3.1
		21/01/74 - 21/01/74	320	34	10	4.5
		25/02/74 - 27/02/74	400	14	6	1.8
		03/02/76 - 04/02/76	750	22	12	3.4

Table 2.2 - cont.

Station Ref. No.	Station Name	Date of Storm	O_1	O_2	Δt (hrs)	$K = \frac{\Delta t}{\ln O_1 - \ln O_2}$
2986M01	Ameib	15/11/67 - 16/11/67	140	4.2	8	2.3
		26/03/71 - 27/03/71	280	14	6	2.0
		27/01/74 - 29/01/74	500	11	12	3.2
		27/02/74 - 01/03/74	210	27	11	/5.4/
		04/03/80 - 05/03/80	270	15	5	1.8
2991M01	Schlesien	05/04/65 - 09/04/65	125	15	8	3.8
		28/04/66 - 30/04/66	35	8.5	6	4.3
		07/03/67 - 08/03/67	330	20	6	2.2
		26/02/69 - 27/02/69	110	30	10	7.7
		30/03/73 - 01/04/73	75	25	10	9.1
		25/02/74 - 26/02/74	300	50	6	3.4
		13/03/75 - 15/03/75	120	49	10	/11.2/
		27/01/76 - 28/01/76	42	4.6	10	4.5
		13/03/77 - 17/03/77	90	15	18	10.1
3111M02	Mentz	27/02/74 - 01/03/74	10.8	1.11	17	7.5
		07/03/75 - 09/03/75	13	1.4	18	/8.1/
3112M01	Otjivero	25/03/71 - 28/03/71	54	5.8	9	4.1
		07/04/72 - 11/04/72	90	5.8	11	4.0
		25/02/74 - 27/02/74	172	21	17	/8.1/
3112M02	Amasib	29/01/76 - 30/01/76	80	3.5	5	1.6
		09/02/76 - 11/02/76	60	4.6	5	/2.0/
3121M01	Rehoboth	15/03/72 - 17/03/72	160	23	15	7.7
		28.03/73 - 28/03/73	270	22	13	5.2
		05/03/76 - 06/03/76	190	2	10	2.2
		15/03/77 - 19/03/77	90	7	27	10.6
		26/01/79 - 27/01/79	100	4	9	2.8
		26/02/79 - 28/02/79	52	2.6	32	/10.7/
3122M01	Nauas-poort	22/03/71 - 23/03/71	38	1.4	5	1.6
		16/03/72 - 16/03/72	160	5.2	6	1.8
		20/01/74 - 21/01/74	270	2.4	6	1.3
		27/02/74 - 28/02/74	290	10	7	/2.1/

It was not possible to distinguish any regional differences in the SWA-Namibia data, therefore the curve in Figure All (i.e. equation 2.11) may be assumed to apply to the territory as a whole.

2.7 Rainfall-runoff relationships

Unless reasonable allowance can be made for the difference between total storm input and net effective (excess) rain, employment of deterministic methods of flood hydrograph synthesization cannot be attempted. It was therefore imperative to provide the designer with a basis for estimating storm losses. In other words, having compiled a design storm hyetograph, how must the designer modify this to establish the volume of precipitation that becomes direct runoff before he takes the final step of converting this excess rain with the aid of the Nash-Muskingum equation to a flood hydrograph?

As mentioned in section 2.3, a computer program was written to plot the gauged hydrographs and to calculate the total volume of water discharged during each flood. By inspection of the plotted hydrographs it was possible to ascertain the likely day (or days) on which the causative rain occurred. The next step was to abstract the daily falls recorded at all rainfall stations situated within, or adjacent to, the catchment area of interest. Finally, planimetry of the isohyetal map drawn to fit the measurements at each raingauge yielded the storm input to the catchment which could then be compared with the amount of excess rain occurring as flood runoff.

To facilitate execution of this procedure resort was had to the two computer programs employed in the analysis of large-area storms in SWA-Namibia by Pitman (1980). The first program (SOFTA) scans the magnetic tape on which the daily rainfall data are stored and abstracts daily falls recorded on pre-selected dates at all stations lying within a demarcated area. Output from the program, comprising latitude, longitude and rainfall for each station is stored on disc in a format suited to the requirements of a contouring program (SURFACE II). When running the program SURFACE II, the latitude and longitude scales were adjusted so that the isohyetal map produced by the program was drawn to a scale of 1 : 1 000 000. The isohyetal maps could then be directly superposed on the maps of catchment boundary that were drawn to the same scale.

Figure 2.5 illustrates an example of an isohyetal storm map superimposed on the catchment of the Swakop river at Westfalenhof. Results of all the rainfall-runoff computations are summarized in Table 2.3; the final column in this table lists the flood runoffs as percentages of causative rainfall. A plot of percentage runoff against storm rainfall, as shown on Figure 2.6, reveals no clear relationship between rainfall and runoff. In fact, the diagram indicates percentage runoff to be virtually independent of the causative rainfall. Although Figure 2.6 gives some indication of the percentage runoff to be expected for extreme events (i.e. $\pm$ 20%) it does not enable the designer to arrive at estimates of average losses.

Since it is the aim of the designer to synthesize a flood hydrograph with a peak (and volume) associated with a given return period, it follows that average losses may be defined as those losses which, when subtracted from a design storm of T-year return period, will yield a T-year flood hydrograph. It is therefore necessary to establish a relationship between storm losses, or percentage runoff, and return period.

An average (non-dimensional) frequency curve applicable to flood peaks, which can also be assumed to apply to flood volumes, has been derived for SWA-Namibia (see section 2.5). It was also possible to compile an average (non-dimensional) frequency curve of storm rainfall from the information contained in the diagrams applicable to design storm determination, i.e. Figures A2, A4 and A5. These two curves are drawn on Figure 2.7, where it can be seen that the frequency curve of storm rainfall exhibits a much flatter slope than the curve for floods. If one were to dimensionalise the curves using data for a gauged catchment the rainfall curve must fall above the flood curve, if both are expressed in the same units, e.g. millimetres. Since the vertical scale on Figure 2.7 is logarithmic, a constant percentage runoff would be indicated by

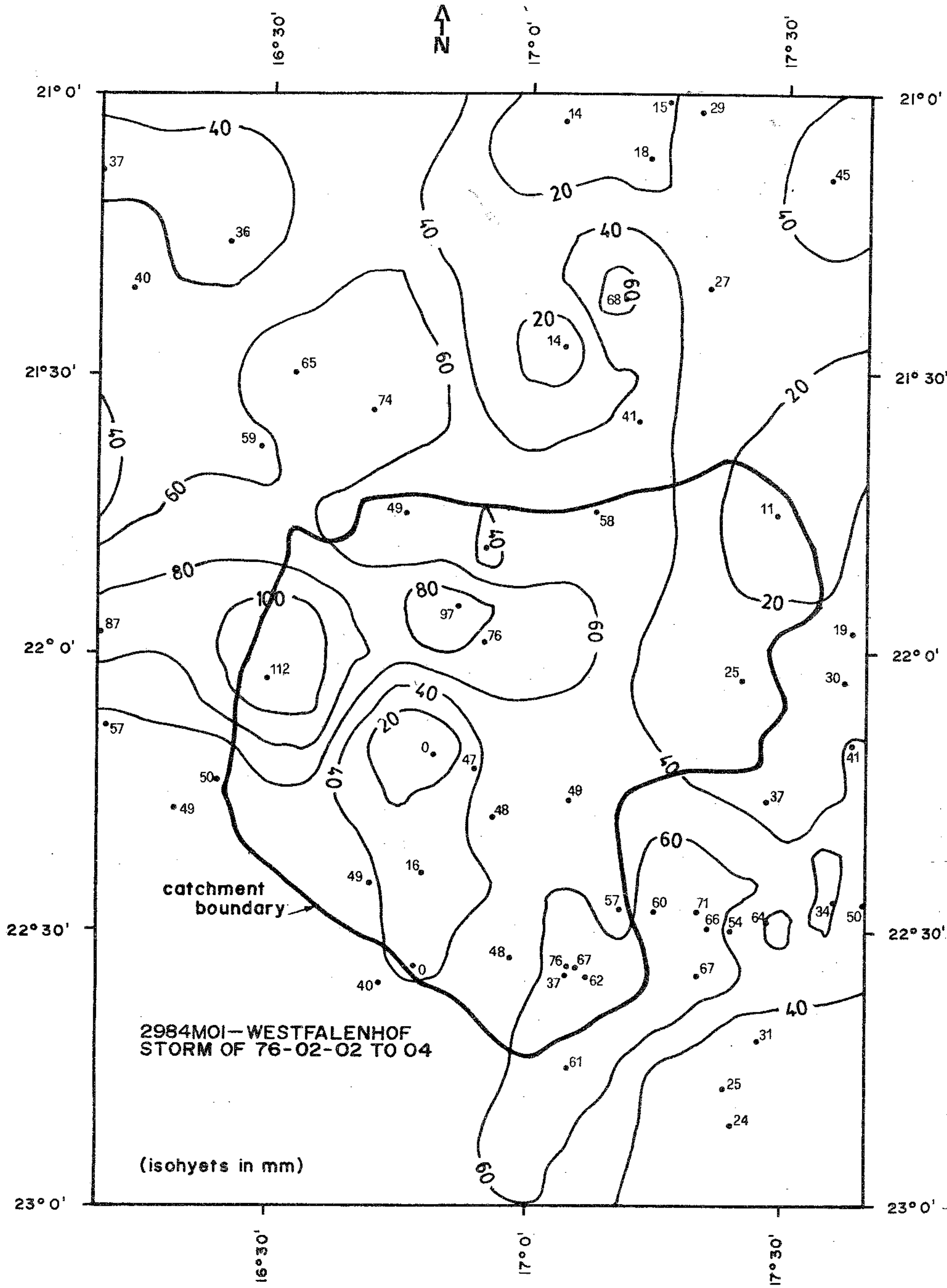

Figure 2.5 Example of isohyetal storm map - catchment of Swakop river at Westfalenhof

Table 2.3 Calculation of runoff as percentage of storm rainfall

Station Ref. No.	Station Name	Date of Storm	Flood Volume ($10^6 m^3$)	Flood Runoff (mm)	Storm Rainfall (mm)	Runoff as percentage of storm rainfall
0482M01	Tsamab	17-18/04/73	5.79	2.52	19.85	12.70
		18-19/01/74	8.07	3.51	30.86	11.37
0491M01	Gras	14-17/03/72	82.92	8.86	63.83	13.88
		26-29/01/74	77.10	8.24	48.36	17.04
		25-28/02/74	73.71	7.88	50.83	15.50
0493M01	Rietkuil	19-20/02/74	8.35	1.44	21.01	6.85
0496M01	Seeheim	15-18/03/72	567.96	12.15	68.93	17.63
		20-22/02/74	396.14	8.47	47.84	17.70
0497M02	Aikanes	5-07/01/72	42.25	5.44	51.39	10.59
		17-19/04/73	14.91	1.92	30.00	6.40
		14-16/01/74	7.69	0.99	37.12	2.67
		20-22/01/76	15.82	2.04	21.10	9.67
2531M01	Ousema	27-29/01/74	18.19	3.90	36.88	10.57
		27/2 + 1-2/03/74	25.82	5.54	39.21	14.13
		3-5/02/76	19.14	4.11	25.09	16.38
2961M01	Petersburg	10-11/02/62	2.37	1.12	22.87	4.90
		03-04/02/70	1.39	0.66	26.92	2.45
		07-08/01/74	0.57	0.27	21.74	1.24
2962M03	Vingerklip	20-22/11/67	5.20	0.51	15.45	3.30
		10-11/02/71	6.71	0.66	29.05	2.27
		29-31/01/74	4.87	0.48	39.35	1.22
		13-14/03/74	6.30	0.62	22.63	2.74
		02-04/03/75	2.60	0.25	55.27	0.45
2971M02	Etemba	13-14/03/72	6.25	1.69	38.18	4.43
		26-28/01/74	23.08	6.24	60.04	10.39
		25-28/02/74	11.44	3.09	60.09	5.14
2972M01	Henties Monument	27-28/01/74	11.45	0.86	25.94	3.32
		23-25/02/74	75.97	5.71	34.52	16.54
2981M01	Swakophöhe	05-07/02/71	11.45	4.30	68.94	6.24
		05-06/04/72	14.83	5.58	35.32	15.80
		29-31/01/74	10.78	4.05	41.38	9.79

Table 2.3 - cont.

Station Ref.No.	Station Name	Date of Storm	Flood Volume ($10^6 m^3$)	Flood Runoff (mm)	Storm Rainfall (mm)	Runoff as percentage of storm rainfall
2984M01	Westfalenhof	12-13/02/71 27-28/03/72 20-21/01/74 24-26/02/74 02-04/02/76	4.49 11.85 6.25 17.50 7.12	0.47 1.24 0.66 1.84 0.75	14.75 34.82 13.51 54.29 49.91	3.19 3.56 4.89 3.39 1.50
2986M01	Ameib	14-15/11/67 25-26/03/71 26-28/01/74 26-27/02/74	2.66 4.01 19.99 6.29	0.51 0.77 3.84 1.21	30.66 21.01 65.78 17.86	1.66 3.66 5.84 6.77
2991M01	Schlesien	06-08/03/67 25-26/02/69 29-31/03/73 24-26/02/74	5.18 4.74 6.03 4.83	0.79 0.72 0.92 0.73	10.24 21.55 12.77 32.49	7.71 3.34 7.20 2.25
3111M02	Mentz	26-28/02/74 06-08/03/75	0.22 0.48	0.03 0.06	32.85 21.57	0.09 0.28
3112M01	Otjivero	05-07/04/72 24-26/02/74	7.30 8.64	4.08 4.83	47.51 62.12	8.59 7.78
3112M02	Amasib	08-09/02/76	0.38	0.05	42.61	0.12
3121M01	Rehoboth	14-16/03/72 27-28/03/73 04-06/03/76	6.70 6.97 1.38	2.79 2.90 0.58	52.06 27.82 56.87	15.36 10.42 1.02
3122M01	Nauaspoort	15-16/03/72 18-21/01/74 25-28/02/74	1.18 1.49 3.21	1.84 2.33 5.02	67.30 11.05 81.04	2.73 21.09 6.19

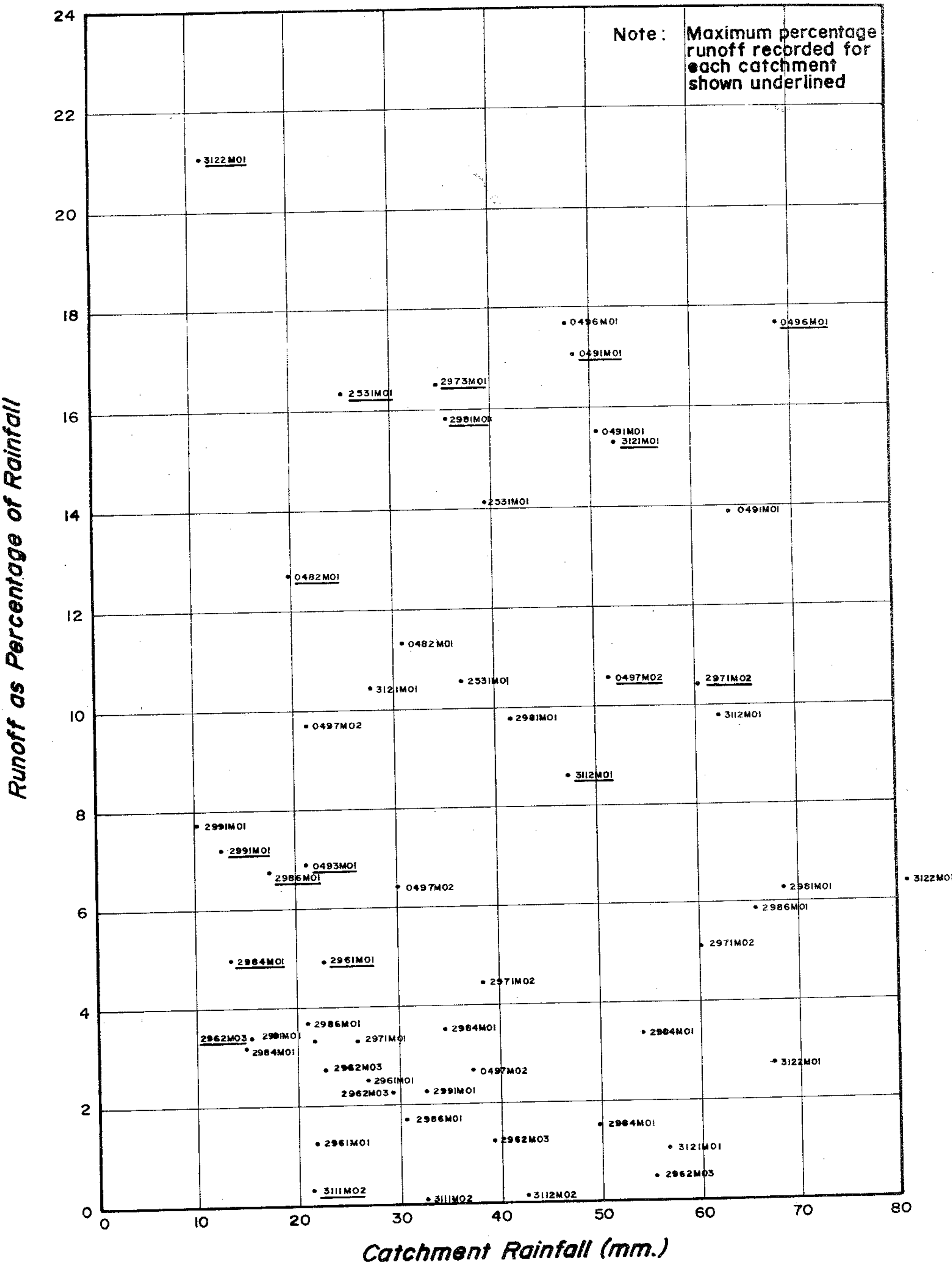

Figure 2.6 Plot of percentage runoff against storm rainfall

Table 2.3 - cont.

Station Ref.No.	Station Name	Date of Storm	Flood Volume ($10^6 m^3$)	Flood Runoff (mm)	Storm Rainfall (mm)	Runoff as percentage of storm rainfall
2984M01	Westfalenhof	12-13/02/71	4.49	0.47	14.75	3.19
		27-28/03/72	11.85	1.24	34.82	3.56
		20-21/01/74	6.25	0.66	13.51	4.89
		24-26/02/74	17.50	1.84	54.29	3.39
		02-04/02/76	7.12	0.75	49.91	1.50
2986M01	Ameib	14-15/11/67	2.66	0.51	30.66	1.66
		25-26/03/71	4.01	0.77	21.01	3.66
		26-28/01/74	19.99	3.84	65.78	5.84
		26-27/02/74	6.29	1.21	17.86	6.77
2991M01	Schlesien	06-08/03/67	5.18	0.79	10.24	7.71
		25-26/02/69	4.74	0.72	21.55	3.34
		29-31/03/73	6.03	0.92	12.77	7.20
		24-26/02/74	4.83	0.73	32.49	2.25
3111M02	Mentz	26-28/02/74	0.22	0.03	32.85	0.09
		06-08/03/75	0.48	0.06	21.57	0.28
3112M01	Otjivero	05-07/04/72	7.30	4.08	47.51	8.59
		24-26/02/74	8.64	4.83	62.12	7.78
3112M02	Amasib	08-09/02/76	0.38	0.05	42.61	0.12
3121M01	Rehoboth	14-16/03/72	6.70	2.79	52.06	15.36
		27-28/03/73	6.97	2.90	27.82	10.42
		04-06/03/76	1.38	0.58	56.87	1.02
3122M01	Nauaspoort	15-16/03/72	1.18	1.84	67.30	2.73
		18-21/01/74	1.49	2.33	11.05	21.09
		25-28/02/74	3.21	5.02	81.04	6.19

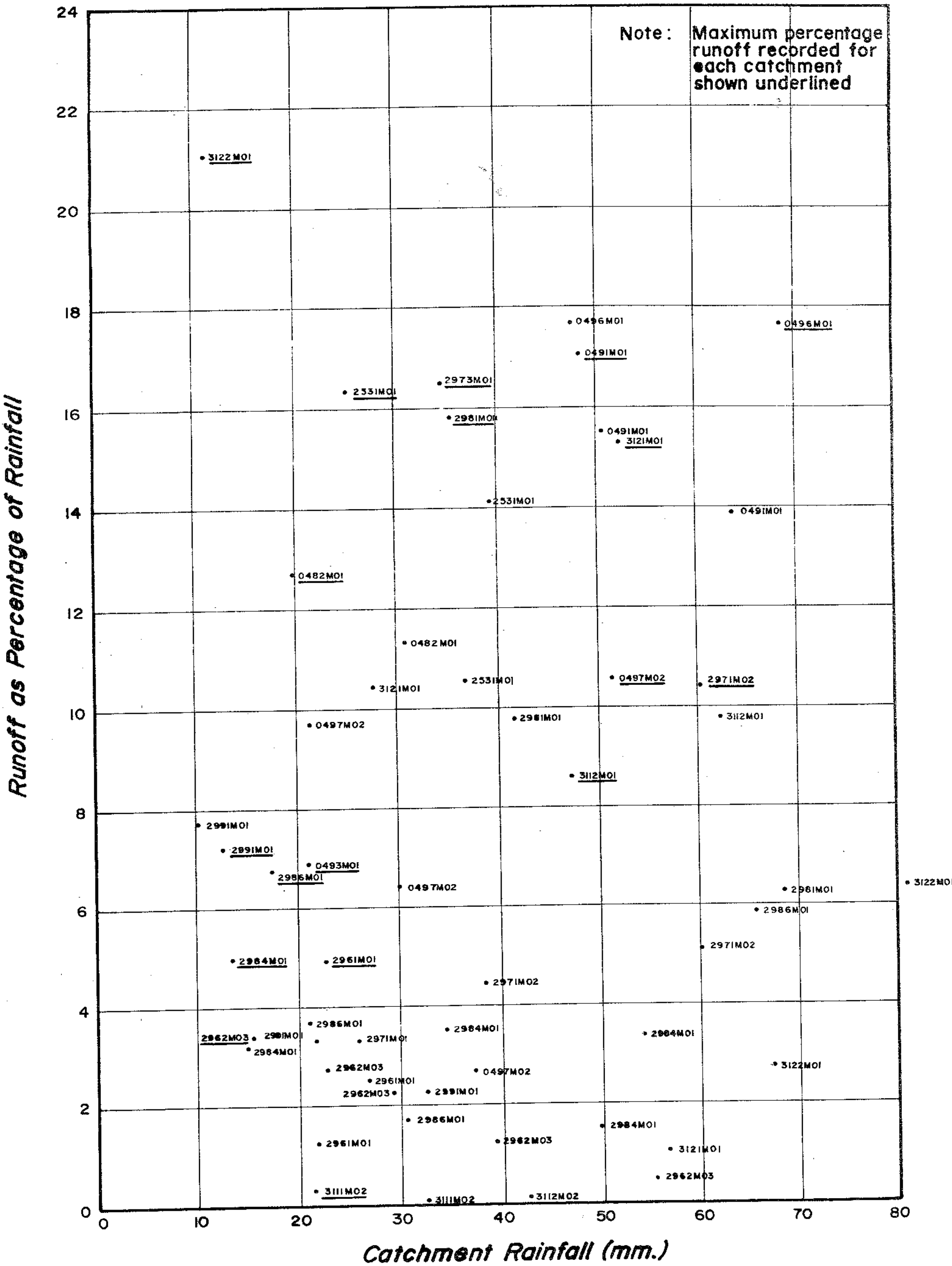

Figure 2.6 Plot of percentage runoff against storm rainfall

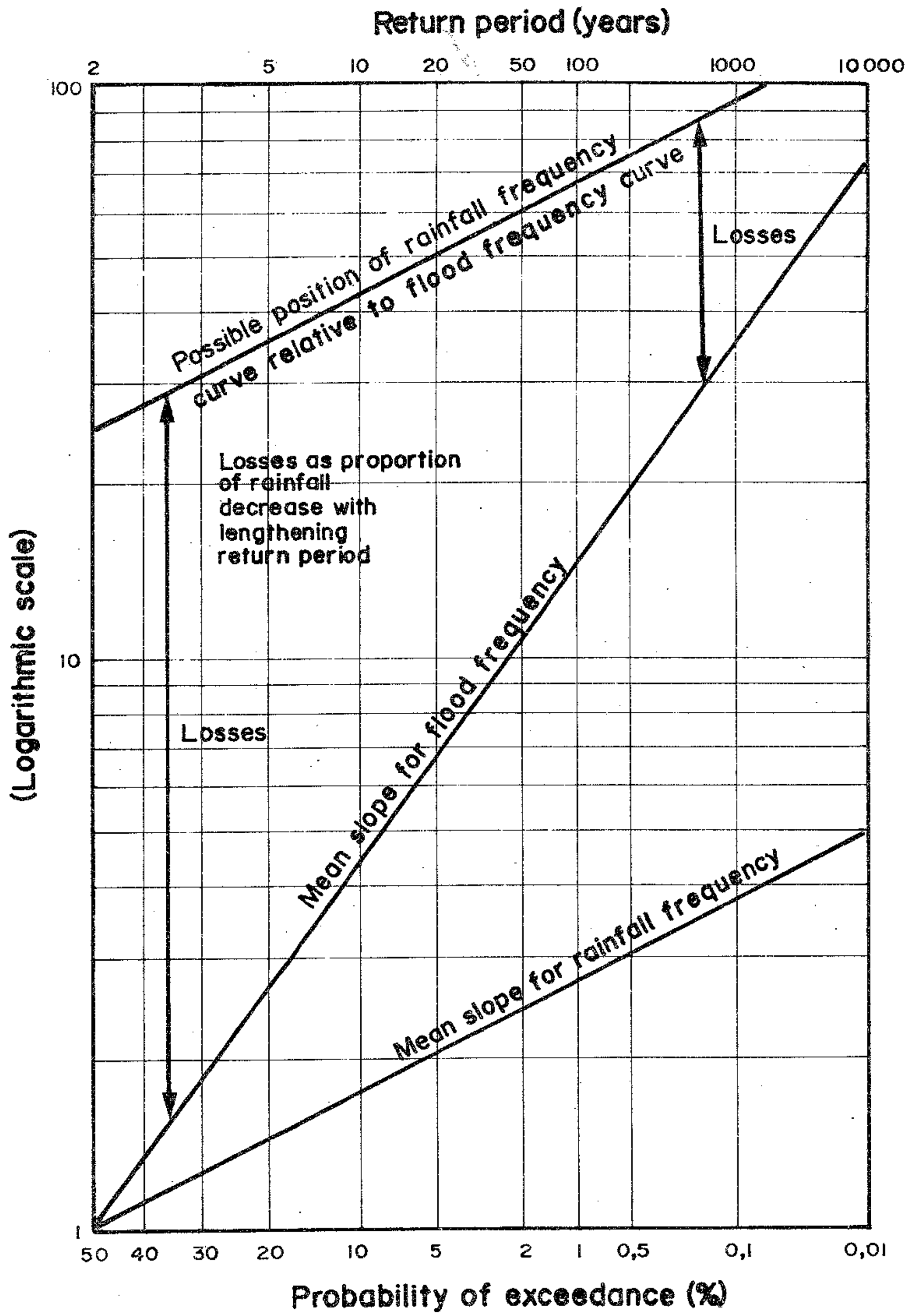

Figure 2.7 Comparison of frequency curves for flood peaks and storm rainfall

parallel lines. However, the steeper flood frequency curve suggests convergence of the two curves as return period increases. In other words, the proportion of storm rainfall contributing to direct runoff must increase with lengthening return period.

Before proceeding to estimate such a relationship between percentage runoff and return period it was necessary to establish any regional differences that might be apparent. Inspection of Figure 2.6 (or Table 2.3) revealed some marked differences between catchments in terms of catchment response, and these differences were to some extent corroborated by comparison of recorded flood peaks (e.g. Figure A9). Superposition of the map showing stream gauge location (Figure 2.1) on a geological map of SWA-Namibia indicated that, in general, the catchments of low response fell wholly or partially in the dolomitic zones or in areas covered by loose, unconsolidated deposits (e.g. sand). Of the remainder of the catchments it was not possible to distinguish any significant difference in flood behaviour owing to the small sample size.

For the sake of interest the essential features (in an hydrological context) of the geological map are reproduced in Figure 2.8. This map provided a basis for sub-division of the country into two zones, i.e. one of "high" flood potential and one of "low" flood potential. In drawing the regional boundary of this map, presented as Figure A.12, the area of very low rainfall along the north-west coast was also included in the "low" zone.

Since nearly all of the gauged catchments lie wholly or partially in the "high" zone, there was sufficient information available to establish a mean relationship between percentage runoff and return period. However, it was not possible to derive such a relationship for the "low" zone, owing to lack of data. Some guidance for these areas may nevertheless be sought from the data in Table 2.3 or Figure 2.6.

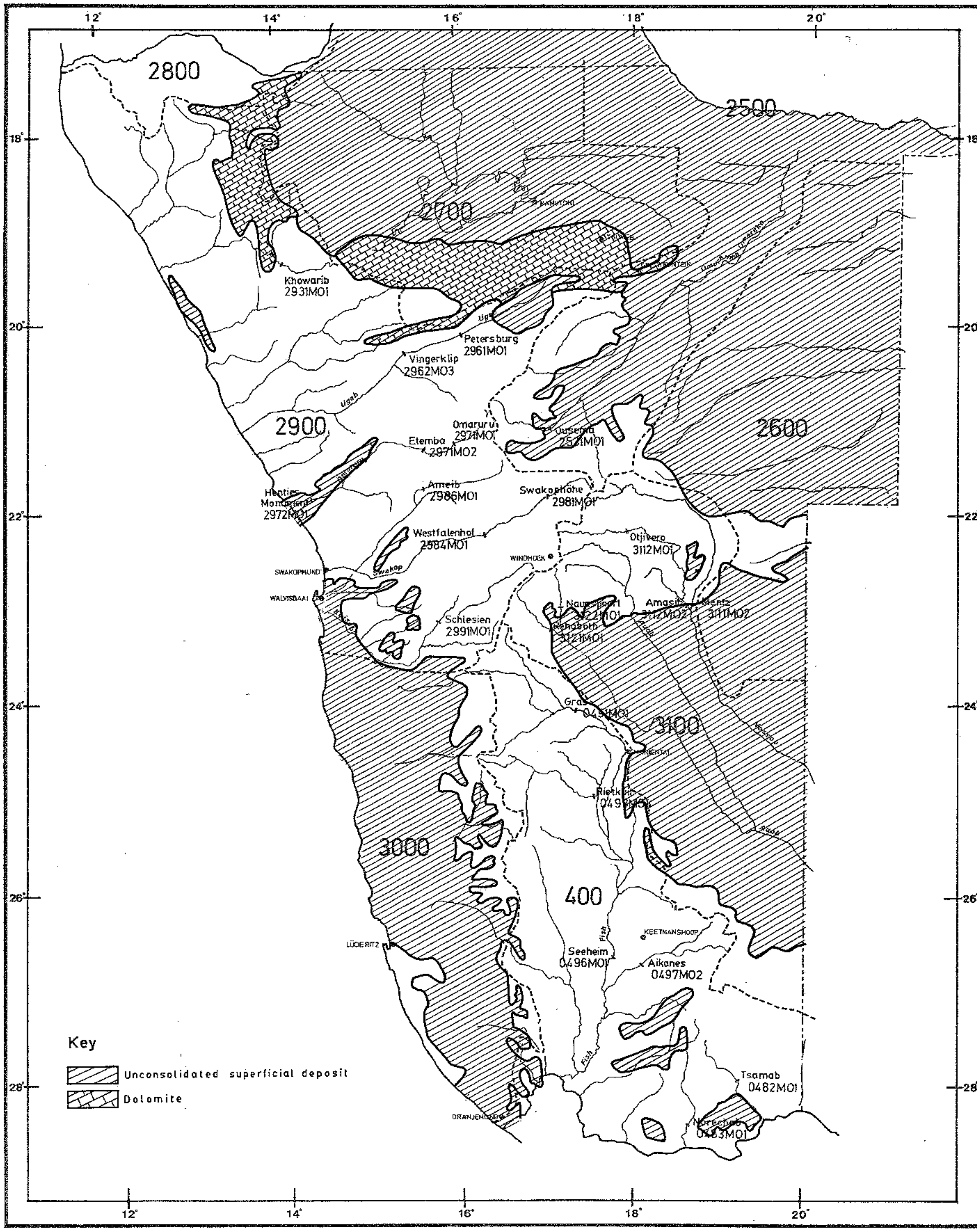

Figure 2.8 Simplified geological map of SWA-Namibia

Note: Unhatched areas are the relatively impermeable formations

The optimum relationship between return period and percentage runoff for the "high" zone was derived by trial and error in the following manner, viz:

i) A percent runoff-frequency curve, satisfying the the constraints imposed by Figure 2.7, was proposed. For example, one could argue that this curve might pass through a runoff of 20% (approx. highest recorded) at a return period of 200 years (approx. total number of station-years of data analysed) as an initial trial.

ii) For each gauged catchment lying within the "high" zone design storms for a range of durations about the estimated critical duration (i.e. the duration yielding highest peak discharge) were computed for return periods of 2 , 10 and 50 years.

iii) Excess rainfall, estimated from the assumed relationship in step (i), was routed through storage by means of the Nash-Muskingum equation with routing constant, K, determined from equation 2.11.

iv) For each gauge, the peak of each hydrograph generated by the storm of critical duration was plotted against return period on the log-normal plots derived by statistical analysis of observed peaks (see Figure 2.2).

v) Comparison between observed and synthesized flood frequency curves, for all catchments, indicated the direction and magnitude of adjustment to the proposed percent runoff-frequency curve of step (i).

vi) Steps iii), iv) and v) were re-run with the new curve to check that the adjustments to it were in fact optimal.

The adopted frequency relationship between percentage storm runoff and return period, as derived by the technique described above, is shown on Figure A13.

It is possible to make a rough check on the validity of this rainfall-runoff relationship if one assumes that all the recorded flood events are independent. Table 2.4 lists the number of runoff events expected to exceed various percentages of storm rainfall, based on a total period of 200 years which is the approximate number of station-years analysed. The final column lists, for comparison, the actual number of events exceeding the selected thresholds ascertained from Figure 2.6.

Table 2.4 Validity check on adopted rainfall-runoff-frequency relationship

Runoff	Derived from Figure A.13		Observed
(% rainfall)	Return period (yrs)	No. of events	No. of events
6	7	29	28
8	14	14	21
10	27	7	17
12	50	4	12
14	83	2	10
16	140	1	6
18	220	1	1

The relatively high number of observed events compared to the calculated number can be ascribed to the non-independence of the events. Inspection of Table 2.3 shows that many of the storm dates appear in the data for several catchments, thus substantiating the argument for non-independence. It is also of note that the period January - February 1974, during which exceptional rains were experienced over most of SWA-Namibia, accounts for a disproportionately high number of floods. In the light of the foregoing, it would seem that the proposed percentage runoff-frequency relationship is entirely plausible.

2.8 Information for rapid estimation of design floods

The main theme of this chapter has been centred on the development of various graphs intended for the determination of design floods, i.e. floods associated with specific probabilities of occurrence. These design diagrams are presented in Appendix A together with the experience envelopes (Figures A9 and A10) which, although they are not related to probabilities, permit rapid estimation of flood potential as reflected by historic data.

Synthesis of a flood of specific return period, however, comprises a number of steps, viz.:

i) Design storm determination for a range of durations (Figures A1 - A8)
ii) Estimation of excess rain for each storm (Figure A13)
iii) Estimation of the routing constant, K (Figure A11)
iv) Nash-Muskingum routing of excess rain for each storm (Equations 2.4 - 2.8)
v) Selection of event yielding largest peak.[1]

The amount of computational effort involved in this process can be considerably reduced if a sufficiently accurate assessment of the critical duration can be made beforehand. In the trial calculations needed to establish critical storm durations, D, for the purpose of estimating storm losses (see section 2.7), it was found that D varied between one-half K and K, where K is the routing constant, i.e.:

$$0{,}5K \leq D \leq K$$

Furthermore, a storm of duration 0,75K was found to yield flood peaks of magnitude not less than 97 per cent of the maximum peak in all the cases that were tested.

To the designer concerned with flood peaks only and who wishes merely to make a rapid estimation of maximum discharges associated with various return periods, synthesization of complete hydrographs merely to establish the peaks would appear to be an unnecessarily tedious process. Consequently, it was considered worthwhile to publish Figure A14. This diagram was

---

1

In situations where the hydrograph is to be routed through a reservoir, this criterion is not necessarily valid.

constructed by plotting, for various return periods, peak discharges of flood hydrographs (synthesized by the complete method described above) against catchment area for a selection of catchments. The parametric lines relating discharge to return period were drawn to a slope of 0,5, indicating that for hydrologically similar regions flood peaks tend to be proportional to the square root of effective catchment area. This relationship was found by Pitman and Midgley (1967) to hold for catchments in South Africa and the flood peaks synthesized for selected catchments in SWA-Namibia followed this relationship fairly closely - equivalent to adoption of a Francou-Rodier K of 5,0.

Also shown in Figure A14 are the two experience envelopes to SWA-Namibia data drawn according to the equations proposed by Creager and Francou-Rodier respectively (see Figures A9 and A10). Not unexpectedly, these two curves lie well above the proposed 100-year line; the Francou-Rodier K = 4,3 envelope indicates discharges ranging from $1{,}5Q_{100}$ to $2{,}5\ Q_{100}$ and the Creager 30 curve $1{,}5Q_{100}$ to $3{,}0Q_{100}$, where Q_{100} is the indicated 100-year peak discharge.

Extrapolation of the flood frequency relationship, according to the regional curve in Figure 2.3, would place a 1000-year line fairly close to the Creager 30 envelope, indicating a Q_{1000}/Q_{100} ratio of the order of 2,5. Since the experience curves are based on several hundred station-years of data the equivalent return period of 1000 years does not seem unreasonable.

In South Africa, where an appreciable body of data has been assembled for estimating probable maximum flood, PMFs of the order of five times the 100-year event are typical. According to Figure 2.3 a flood of such magnitude would have a return period of about 10 000 years (as extrapolated on the adopted frequency distribution paper). In the absence of adequate data the 10 000-year curve drawn on Figure A14 might well be adopted as a rough guide for estimating probable extreme discharges. Indeed, many engineers seem to consider the PMF to be associated with return periods of the order of 10 000 years.

This contradicts the first paragraph of chapter 2.4!

2.9 Additional information related to design floods from small areas

The sample of gauged catchments for which reasonably adequate data on flood flows were available is not only small but is also confined to fairly large catchments; inspection of Table 2.1 shows that all but one of the gauges command catchments exceeding 1000 km^2 in area. However, many problems - particularly those concerned with urbanization - relate to the estimation of flood runoff from small areas of the order of a few square kilometres.

The results of the various analyses of extreme rainfall - described in detail in HRU Reports 3/79 and 2/80 and summarised in diagrammatical form in Figures A1 to A8 - enable the designer to synthesize storms covering any area up to 100 000 km^2. Provided one can (a) assess the infiltration losses during a storm so as to convert total rainfall into excess rainfall and (b) perform the routing process whereby excess rainfall is attenuated as it travels as runoff to the catchment outlet, the design storm data can be employed to synthesize the design flood hydrograph. The problem here is that both Figure A13 (from which the percentage of total rain that becomes excess rain can be estimated) and Figure A11, which provides estimates for routing constant K (attenuation effect), are based on data from large catchments. While some degree of extrapolation would be in order it would be unwise to apply the relationships in Figures A13 and A11 to catchments as small as say 10 km^2.

For urban areas particularly, there are sophisticated deterministic models such as SWMM and ILLUDAS that can be applied with local storm data. In fact, Watson (1981a) has successfully modified the ILLUDAS model for use on urban catchments in South Africa and he claims that the principles are applicable also to rural catchments. Whereas urban flood hydrology is fairly well defined once the causative rainfall has been established, response to rainfall on rural catchments depends heavily on local conditions (vegetal cover, soil texture, antecedent moisture, etc.). One must therefore exercise considerable caution when applying a model to rural areas. Watson (1981b) has developed other desk-top techniques for handling small-catchment flood runoff - both urban and rural. Stephenson (1981) explains applications of kinematic flow theory to the determination of storm runoff from either urban or rural catchments. Constantinides and Stephenson (1981) have developed two-dimensional kinematic routing techniques for generating the flood hydrograph for catchments of variable topography and shape, accounting for variable time and space distribution of rainfall and infiltration. Their technique can account also for canalization, obstruction and diversion of the flow.

A widely used method, designed specifically for estimating floods from small rural catchments, is the United States Soil Conservation Service hydrograph generating technique. The SCS method takes into account many factors affecting storm runoff, viz. land use, soil characteristics, antecedent moisture, surface retardance, slope, hydraulic length and catchment shape, in addition to the parameters of the design storm. A prerequisite to application of this method is a detailed classification of the soils into units or groups that are relatively homogeneous with respect to hydrological response.

A much simpler method, but one which relies more heavily on judgment, is the so-called Rational Formula:

$$Q = CIA \tag{2.12}$$

in which Q is peak discharge in m^3/s

C is a dimensionless coefficient

I is point rainfall intensity in m/s

A is catchment area in m^2.

(If I and A are expressed in the usual units of mm/h and km^2 respectively the equation becomes Q = 0,278 CIA).

Factors affecting peak runoff, such as infiltration, evapotranspiration, antecedent conditions and attenuation effects of natural storage within the catchment, are all lumped in the empirical coefficient, C.

The first step in the Rational method is the calculation of the catchment response time, usually referred to as the time of concentration, and often erroneously believed to be the time taken for runoff from rain falling on the most remote part of the drainage system to reach the catchment exit. It is in fact the time taken for the flood response wave to traverse the catchment, i.e. a somewhat shorter time than the runoff travel time. For design purposes the critical storm duration is taken to be the time of concentration.

Various empirical formulae for estimating time of concentration have been offered. Most of these give strong emphasis to the general slope of the catchment. Perhaps the most widely used is that suggested by the former U S Bureau of Reclamation; when converted to metric system it reads:

$$T_c = \left[\frac{0{,}87}{H} L^3 \right]^{0{,}385} \tag{2.13}$$

in which T_c is the concentration time in hours
L is the length of the longest watercourse in km
and H is the difference in elevation between the outlet and the source of the longest watercourse in metres.

When estimating time of concentration for an urban area the Bransby-Williams formula is usually preferred, viz:

$$T_c = 0{,}96 \left[\frac{L^{1,2}}{H^{0,2} . A^{0,1}} \right] \tag{2.14}$$

in which T_c, L and H have the same meaning as in equation 2.13
and A is the catchment area in km^2.

Recommended values of the runoff coefficient C - based on experiments conducted in the USA - are shown in Figure A15. *
However, it must be stressed that these values should, wherever possible, be tempered with knowledge of local conditions.

* [illegible] rural [illegible] from [illegible] calculation [illegible]

CHAPTER 3 WORKED EXAMPLES

This chapter contains two worked examples to demonstrate the handling of the design aids in the report with the aid of a pocket calculator. It must be remembered that the diagrams in Appendix A are based on generalizations and are therefore approximate. Each of the curves in the diagrams is contained within a confidence band the width of which must be assessed on the basis of supplementary investigations, where possible. In any event it is always advisable to inspect the catchment and the river gauges, if any, before adopting the results of calculations based on the information in this report.

The damage costs of failure of a hydraulic structure to meet requirements can be multiplied by the annual probability of failure. If plotted to a size-of-structure base the curve of annual damage costs will decline as size of structure increases. One can plot a curve to the same base showing the rising annual construction cost of providing a larger and larger structure, and then add the ordinates of the two curves to yield the total annual cost. The sum of a rising and a falling curve will reflect a minimum which is the optimum size of structure. The hydrological problem thus boils down to establishing the relationship between size of flood at the problem site and its return period.

3.1 Large-area design flood procedures

In this example it is desired to synthesize, for a range of exceedance probabilities, design flood hydrographs at Hardap Dam on the Fish river, near Mariental. The catchment (shown in Figure 3.1) is 13 400 km^2 in extent.

Procedure

i) Routing constant, K, is read off Figure A11 or determined from the formula relating K to catchment area, A.

$$\text{i.e.}\quad K = 0{,}2.13\ 400^{0{,}42} = \underline{10{,}8 \text{ hours}}$$

ii) Design storms

The first step is to select a storm duration (or range of durations) based on the value of K. Critical duration lies between the limits 0,5K and K, with the best overall estimate equal to 0,75K. For the purpose of this exercise we shall select a duration of approximately 0,75K, but it should be appreciated that this is not necessarily the critical duration in all cases. Furthermore, if the design flood of a given return period is to be routed through a reservoir for the purpose of checking the spillway capacity additional hydrographs must be synthesized from storms longer than critical in order to ascertain the flood that will yield the maximum outflow rate from the reservoir.

Estimated critical duration = 0,75.10,8 = 8,1
say 8 hours

a) Point-rainfall approach

From rainfall map, Figure A1, draw mean annual isohyets on Figure 3.1 and planimeter areas between isohyets to compute mean annual precipitation (MAP) of 210 mm.

Enter the coaxial diagram for point rainfall, Figure A2, at MAP of 210 mm and read off 8-hour rainfall depths for a range of return periods. Note that it is not necessary to select all return periods indicated on the diagram (i.e. 2 , 5 , 10 , 20 , 50 and 100 years). A minimum of two return periods is required to draw the frequency curve but it is best to select three to provide an additional check. Therefore, adopt return periods of 5 , 20 and 100 years.

The areal reduction factor (ARF) is obtained from Figure A6 and multiplied by the point rainfall

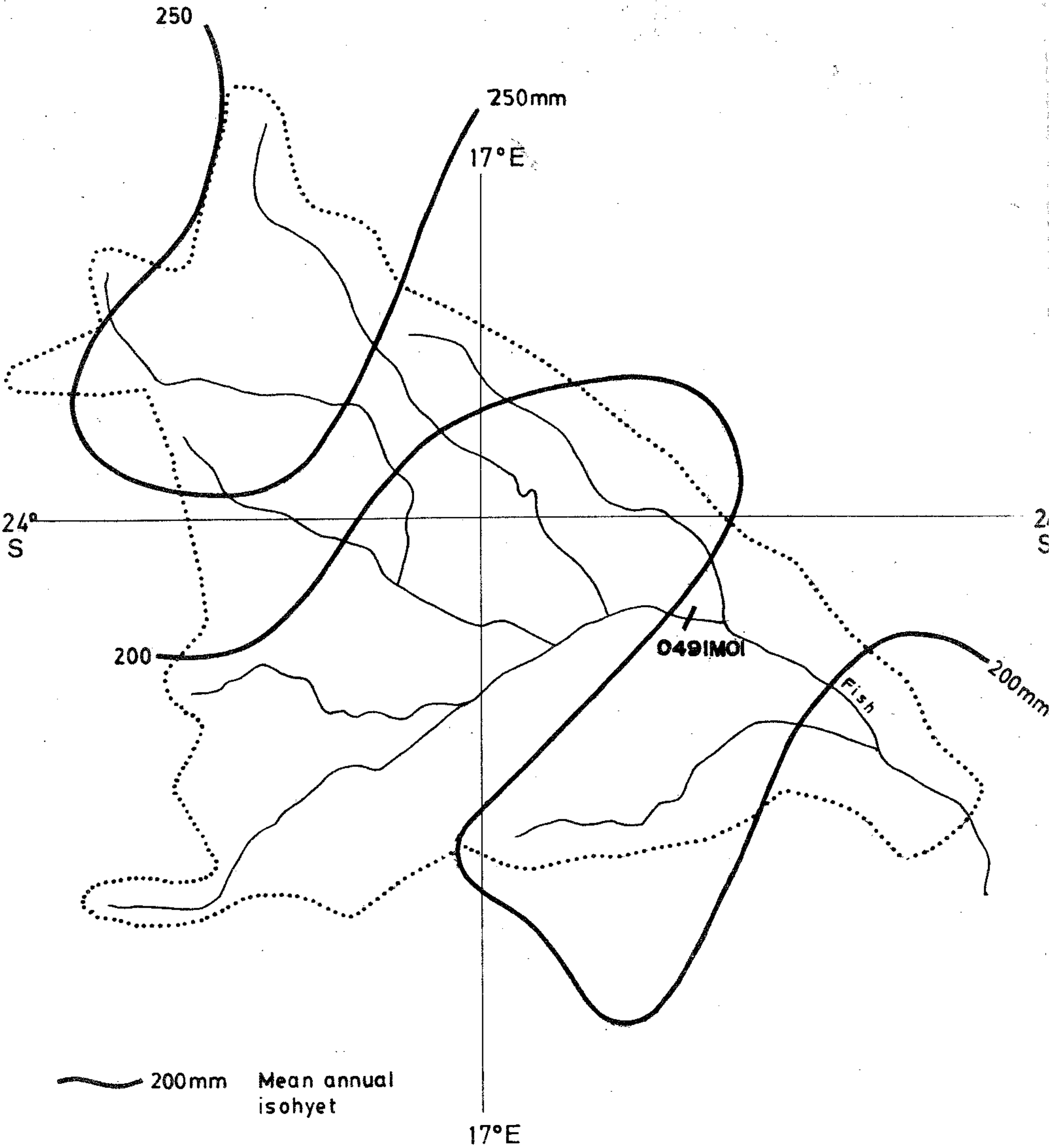

Figure 3.1 Catchment of the Fish river at Hardap dam

to yield average depths as shown below:

	Return period		
	5-year	20-year	100-yr
8-hour point rainfall (mm)	45	75	140
ARF	0,61	0,61	0,61
Average depth (mm)	27	46	85

b) Large-area storm approach

From the map of large-area storm regions, Figure A3, note that the catchment lies within the South region. Enter the relevant coaxial diagram, Figure A5, with area of 13 400 km^2 and read off the one-day rainfalls for return periods of 5 , 20 and 100 years. (Note that if the catchment lies within the Coastal zone one has no option but to rely solely on the point rainfall/ARF approach).

The ratio 8-hour/24-hour rainfall for large areas is obtained from Figure A8 and the 8-hour rainfalls over the catchment are computed below:

	Return period		
	5-year	20-year	100-year
24-hour rainfall (mm)	60	80	100
8-hour/24-hour ratio	0,85	0,85	0,85
8-hour rainfall (mm)	51	68	85

c) Weighted average storm depths

Figure A7 provides a basis for weighting one's estimates of storm rainfall as obtained from methods a) and b) outlined above. For an area of 13 400 km^2

the weighting factor for method a), i.e. "pt. rainfall/ARF", is 0,2 and the factor for method b), i.e. "regional", is 0,8. Accordingly, calculation of storm rainfall is handled as shown hereunder. Answers are rounded to the nearest whole millimetre.

	Return period		
	5-year	20-year	100-year
Method (a) estimate x 0,2 (mm)	5,4	9,2	17,0
Method (b) estimate x 0,8 (mm)	40,8	54,4	68,0
Weighted average depth (mm)	46	64	85

iii) Excess rain

Estimates of excess rain as percentages of total storm rainfall can be obtained from Figure A13. Before accepting information from this diagram, however, one should ascertain the position of the catchment on the regional map, Figure A12. The problem catchment lies entirely within the "high" region therefore no adjustment to the curve in Figure A13 is necessary.

For the purpose of Nash-Muskingum routing, excess rain has to be converted from millimetres depth to cubic metres per second input rate as follows:

If d is the depth (mm) of excess rain in 8 hours over the 13 400 km^2 catchment and q is the equivalent input rate in m^3/s, then

$$q(m^3/s) = \frac{(d.10^{-3}).(13400.10^6)}{8.3600} \quad \frac{m.m^2}{s}$$

$$= 465.d$$

The computations for excess rain are set out below:

	Return period		
	5-year	20-year	100-year
Rainfall depth (mm)	46	64	85
Percentage runoff	5,1	9,1	14,8
Excess rain (mm)	2,35	5,82	12,58
Excess rain (m^3/s)	1092	2710	5853

iv) <u>Routing of excess rain</u>

The first step is to select the length of time step for the routing procedure and to calculate the values of the coefficients (see equations 2.5 - 2.8). In this instance, a one-hour time step has been adopted, i.e. $\Delta t = 1$.

$$C_2 = e^{-\Delta t/K} = e^{-1/10,8} \qquad = \underline{0,912}$$

$$C_1 = \frac{K}{\Delta t}(1-C_2) - C_2$$

$$= 10,8\ (1 - 0,912) - 0,912 \qquad = \underline{0,038}$$

$$C_o = -\frac{K}{\Delta t}\ (1 - C_2) + 1$$

$$= -10,8\ (1 - 0,912) + 1 \qquad = \underline{0,050}$$

(Note that $C_o + C_1 + C_2 = 1$)

The next step is to perform the routing by step-wise solution of the Nash-Muskingum equation, viz:

$$O_2 = C_o I_2 + C_1 I_1 + C_2 O_1$$

The computations for the 100-year flood hydrograph are set out below; note that the inputs, I, are at the rate of 5853 m^3/s for the duration of the storm (8 hours):

$$O_2 = 0{,}050\ I_2 + 0{,}038\ I_1 + 0{,}912\ O_1$$

Time (h)	I (m^3/s)	O (m^3/s)
0	0	0
1	5853	293
2	5853	782
3	5853	1228
4	5853	1635
5	5853	2006
6	5853	2345
7	5853	2654
8	5853	/2935/
9	0	2899
10	0	2644
11	0	2411
12	0	2199
13	0	2006
14	0	1829
15	0	1668
16	0	1521
17	0	1388
18	0	1265
19	0	1154
20	0	1053
21	0	960
22	0	875
23	0	798
24	0	728
.	.	.
.	.	.

The column of outflows, O, constitutes the 100-year flood hydrograph which has a peak of 2935 m^3/s. Hydrographs associated with other return periods can be determined in similar manner. Thus, peak discharge rates for the 5-year and 20-year floods are calculated to be 548 and 1359 m^3/s respectively.

The 100-year hydrograph is shown in Figure 3.2.

v) Flood frequency curve and comparison with quick method

The three computed flood peaks are plotted against return period on log-normal probability paper and a straight line fitted to the data as shown in Figure 3.3. This enables one to read off flood peaks for any desired return period.

As a rough check on the answers one can obtain quick estimates of flood peaks for various return periods from Figure A14. Peaks derived by this quick method are compared below with those derived by the full method described in steps i) to iv) above and interpolated on Figure 3.3. In this case the agreement is fairly good, but it must be remembered that Figure A14 gives peaks only; use of the full method is necessary to synthesize flood hydrographs.

Also shown are rough estimates of the probable maximum flood (PMF) derived by multiplying the 100-year peak by a factor of 5 (or extrapolating the frequency curve to a return period of 10 000 years).

Return period (years)	Flood peaks (m^3/s)	
	Figure 3.3	Figure A14
2	220	190
5	550	500
10	910	820
20	1350	1250
50	2150	2050
100	2950	2800
PMF	14750	14000

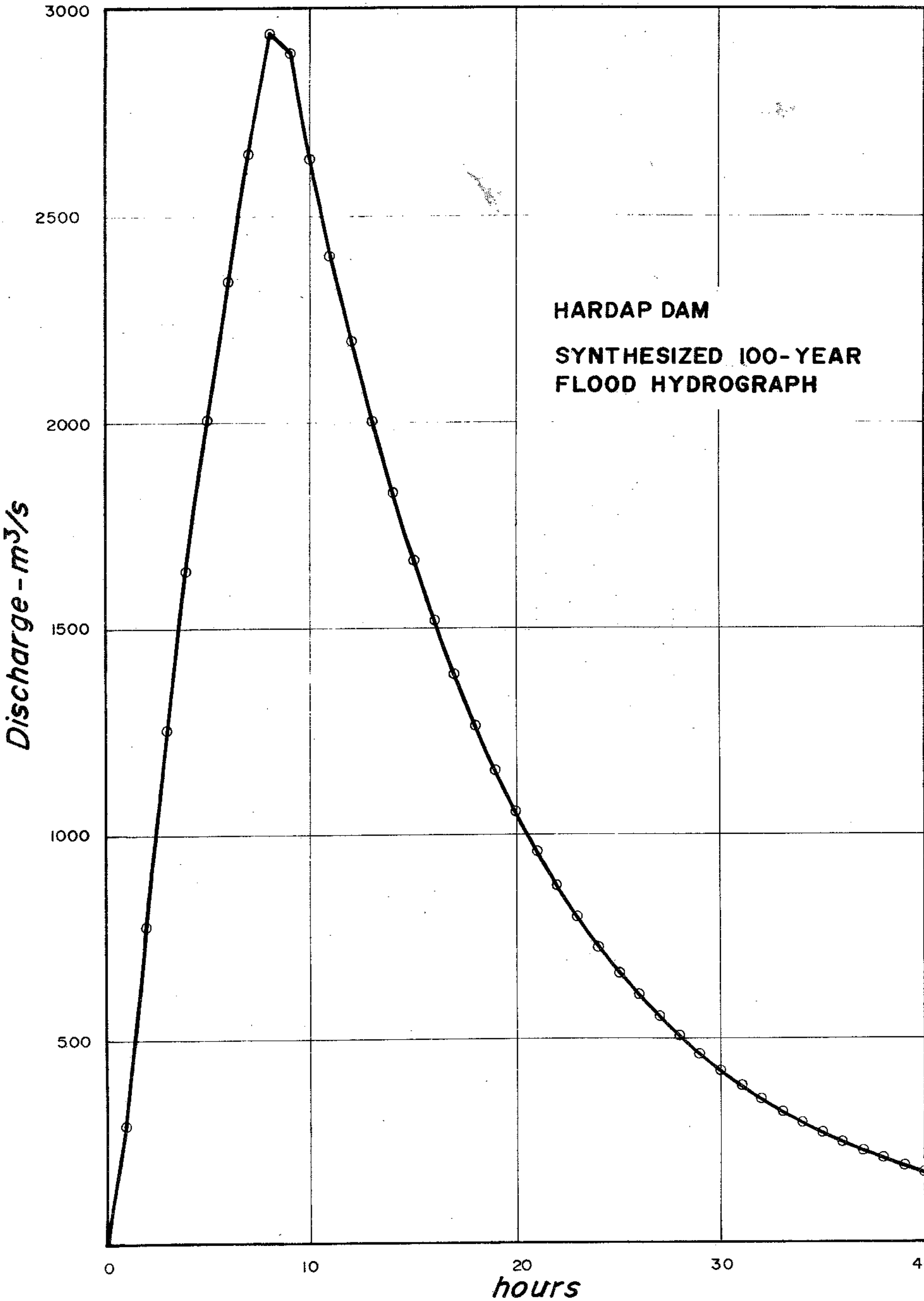

Figure 3.2 Synthesized 100-year flood hydrograph - Hardap dam

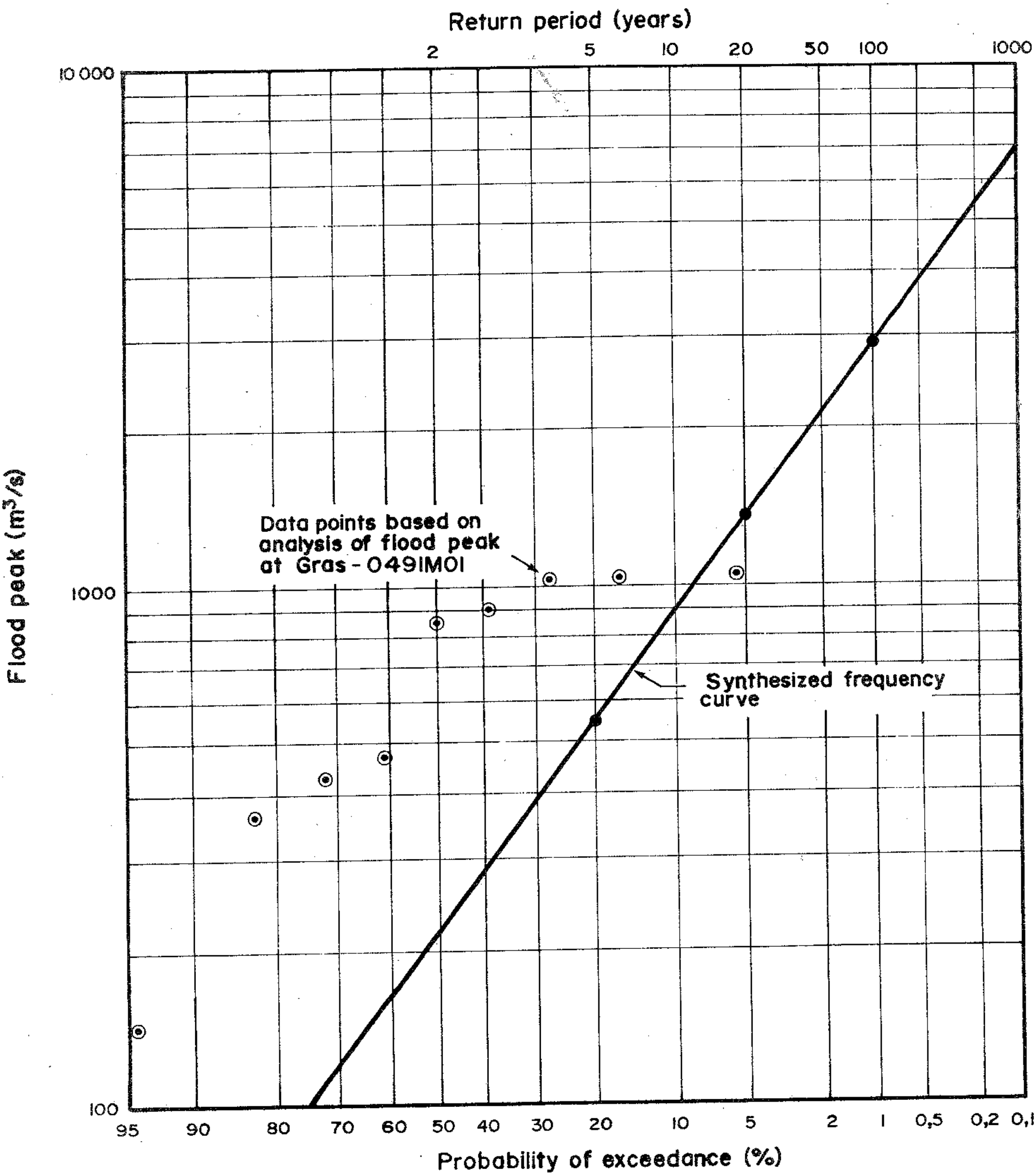

Figure 3.3 Synthesized flood frequency curve - Hardap dam

vi) Comparison with gaugings

Some distance upstream of Hardap dam lies gauging station no. 0491M01 at Gras, commanding a catchment of 9360 km^2. The position of the gauge is plotted on Figure 3.1. The nine-year record, although too short for reliable statistical analysis, may at least provide some indication as to the plausibility of the synthesized frequency curve in Figure 3.3. Calculation of plotting positions is set out in the table below; observed flood peaks are multiplied by 1,20, i.e. the square root of the ratio of catchment areas $\sqrt{13400/9360}$, to render them applicable to the problem site.

Year	Peak (m^3/s)	Rank=m	Plotting position $p(\%)=(\frac{2m-1}{18}).100$	Peak x 1,2 (m^3/s)
1970/71	865	2	16,7	1038
1971/72	872	1	5,6	1046
1972/73	119	9	94,4	143
1973/74	859	3	27,8	1031
1974/75	392	6	61,1	470
1975/76	754	4	38,9	905
1976/77	722	5	50,0	866
1977/78	306	8	83,3	367
1978/79	363	7	72,2	436

When plotted on Figure 3.3, the peaks at Hardap based on adjusted observations at the Gras gauge plot mostly above the synthesized frequency curve. This is to be expected as the decade of the 1970s was known to have been relatively wet (see section 2.7). The diagram illustrates that a distribution based on a short record can be highly misleading and that the band of confidence surrounding a short-record frequency distribution curve can be extremely broad.

3.2 Small-area design flood procedure

In this example it is assumed that a secondary road is to traverse a valley by way of an embankment. It is proposed to pass the floodwaters from the catchment, which is situated in the suburbs of Windhoek, through a pipe culvert. The problem is to decide for what discharge the culvert should be designed. Although most municipalities and government bodies stipulate the return periods to be adopted in hydrological design, this example demonstrates the principles of hydro-economic design.

Assume that the catchment is 4 km^2 in extent and that 75% of the catchment is covered by residential housing and 25% is devoted to suburban business development.

Procedure

i) Time of concentration

a) Delineate catchment boundary and measure area: 4 km^2 (given)

b) Measure length of longest watercourse: 2,5 km (given)

c) Determine difference in elevation between source and outlet of main watercourse: 65 m (given)

d) Calculate time of concentration using Bransby-Williams formula applicable to urban areas:

$$T_c = 0{,}96 \frac{L^{1,2}}{H^{0,2}A^{0,1}} \qquad \text{hours}$$

$$= \frac{0{,}96 . 2{,}5^{1,2}}{65^{0,2} . 4^{0,1}} = 1{,}09 \text{ hours, say } \underline{1 \text{ hour}}$$

ii) Estimation of C

From Figure A15 note that C for residential (single family) areas ranges from 0,30 to 0,50 and that C for suburban business development lies between 0,50 and 0,70. Calculate weighted mean C as follows:

	Rational C		
	minimum	average	maximum
Residential (75%)	0,30	0,40	0,50
Suburban business (25%)	0,50	0,60	0,70
Weighted mean C	0,35	0,45	0,55

iii) Point rainfall intensities

From mean annual rainfall map, Figure A1, or from examination of local records, note that MAP for Windhoek is approximately 350 mm. Enter Figure A2 at MAP of 350 and read off point rainfalls for a one-hour duration and a range of return periods.

	Return period (years)					
	2	5	10	20	50	100
1-hour rainfall (mm)	25	35	44	55	76	96

iv) Peak discharges

Peak discharges, Q, are derived from the rational formula $Q = CIA$ as shown below. Estimates of Q for both minimum and maximum estimates of runoff coefficient C are made in order to establish a confidence band to be used in the subsequent hydro-economic analysis. Note that the true confidence band would be somewhat wider owing to the inaccuracies inherent in the estimation of rainfall intensity I from local data or the coaxial diagram, Figure A2. (Note $A = 4 \times 10^6 m^2$)

	Return period (years)					
	2	5	10	20	50	100
Rainfall intensity (mm/h)	25	35	44	55	76	96
I $(m/s.10^{-6})$	6,94	9,72	12,22	15,28	21,11	26,67
$Q_{min}=C_{min}IA(m^3/s)$	9,7	13,6	17,1	21,4	29,6	37,3
$Q_{ave}=C_{ave}IA(m^3/s)$	12,5	17,5	22,0	27,5	38,0	48,0
$Q_{max}=C_{max}IA(m^3/s)$	15,3	21,4	26,9	33,6	46,4	58,7

v) Hydro-economic analysis

a) Construction costs

Hydro-economic analysis entails first of all the costing of the embankment plus culverts for a range of discharges likely to bracket the optimum design. For the purpose of this example, we shall assume that this exercise has already been carried out. These construction costs must be converted to equivalent annual costs by multiplying by the capital recovery factor, CRF, which may be computed from the expression:

$$CRF = \frac{i(1+i)^n}{(1+i)^n-1}$$

where i represents the annual interest rate (expressed as a decimal fraction) and n represents the design life of the structure. Construction costs are shown on Figure 3.4; note that a CRF of 0,10 has been assumed for this example.

b) Damage costs

The assumption here is that the embankment will fail if the design capacity of the culvert is exceeded and it is estimated that the cost of damages resulting from failure would amount to R5000 plus the cost of rebuilding the structure at the original price.

Annual damage cost is obtained by multiplying the total repair cost by the probability that the structure will fail in any one year.

c) Hydro-economic optimization

Total (annual) cost of the structure is obtained by adding annual construction cost to annual damage cost. Optimum design is that associated with the scheme which yields minimum total cost. Calculations are set out in the table below and the results are shown graphically on Figure 3.4.

		Costs in rands for return periods of (yrs)					
		2	5	10	20	50	100
Annual construction costs	(min.	750	900	1030	1180	1440	1670
	(ave.	850	1050	1200	1380	1680	1940
	(max.	970	1180	1350	1560	1900	2200
Total damage costs	(min.	12500	14000	15300	16800	19400	21700
	(ave.	13500	15500	17000	18800	20800	24400
	(max.	14700	16800	18500	20600	24000	27000
Annual prob. of failure		0,50	0,20	0,10	0,05	0,02	0,01
Annual damage costs	(min.	6250	2800	1530	840	388	217
	(ave.	6750	3100	1700	940	416	244
	(max.	7350	3340	1850	1030	480	270
Total annual costs	(min.	7000	3700	2560	2020	1828	1887
	(ave.	7600	4150	2900	2320	2096	2184
	(max.	8320	4520	3200	2590	2380	2470

The calculated values of total annual cost suggest that the optimum design should cater for the 50-year flood, while Figure 3.4 indicates that the design discharge ranges from 30 to 47 m^3/s with a mean value of 38 m^3/s. A conservative designer would tend to adopt the higher figure of 47 m^3/s since the curves of total cost on Figure 3.4 rise relatively slowly to the right of the minima.

3.3 Discussion

The two examples discussed in this chapter fall neatly into two categories, viz.:

Ex. 3.1 : Large catchment - zone of high flood potential
Ex. 3.2 : small catchment - flood peaks only.

One can readily visualise flood-related problems that do not fall into either of these two categories, such as :

i) intermediate catchment area
ii) intermediate or large catchment in zone of low flood potential
and iii) hydrograph synthesization for small catchments.

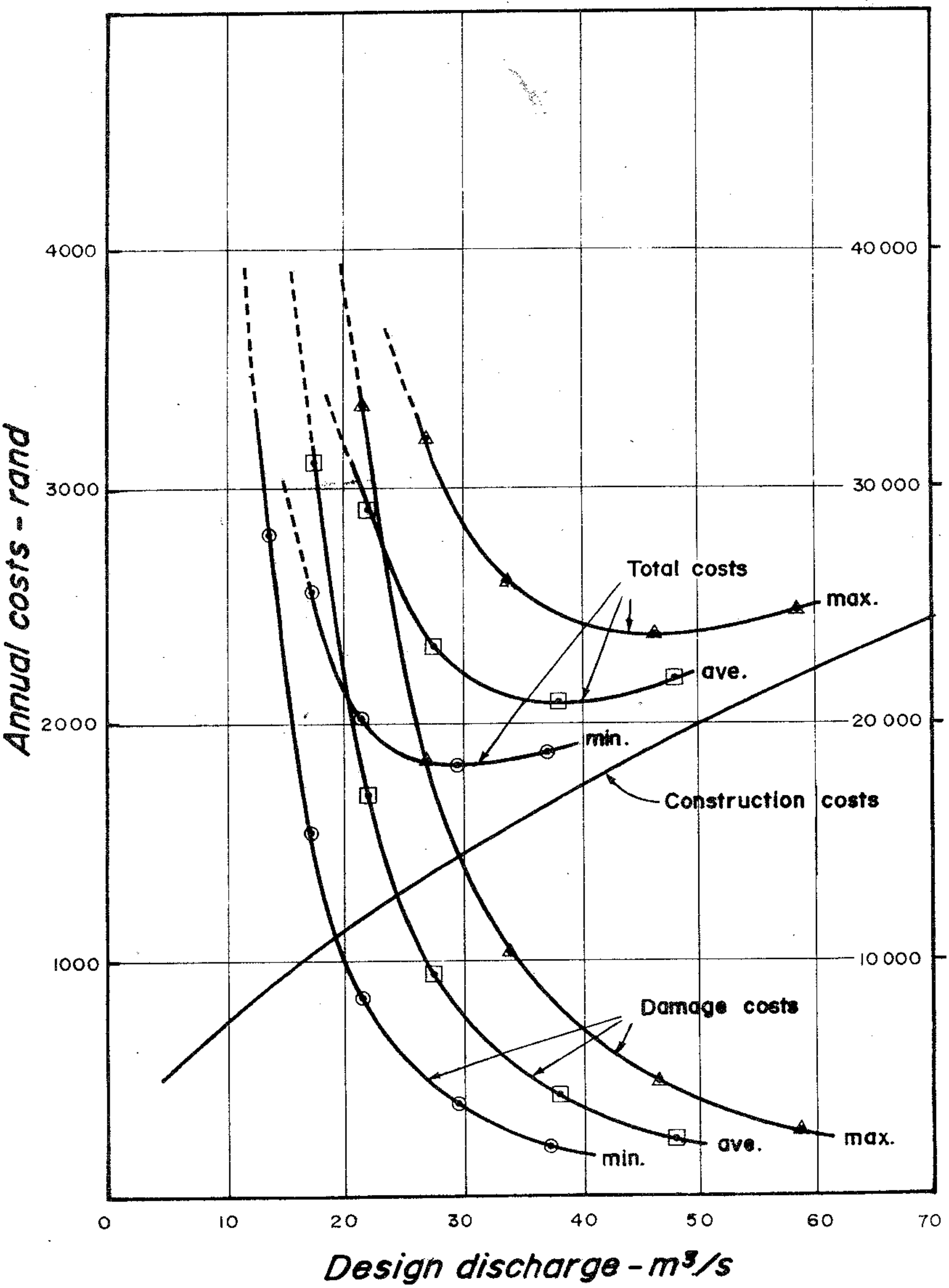

Figure 3.4 Hydro-economic optimization of culvert size

There is no hard and fast rule to define what is meant by small, intermediate and large areas respectively. In the context of this report a small area is defined as one for which the Rational formula and its runoff coefficients, as tabulated in Figure A15, are applicable. In HRU Report no. 1/72 it is suggested that the rational method should be restricted to areas up to about 15 km^2.

In this report a large area may be defined as one for which the excess rainfall routing method may be applied with some degree of confidence. Although the method itself is not limited to any specific range of catchment size there is a dearth of information on routing constant, K, for catchments less than about 500 km^2 in extent. Uninformed extrapolation of the derived K-Area relationship in Figure A11 to areas smaller than say 100 km^2 would therefore be unwise. When dealing with intermediate areas in the range 15 - 100 km^2 the designer is urged to employ both the excess rainfall routing method and the rational method and to weight the answers in the light of local information, if any.

Perhaps the most difficult problem is one concerned with flood design in the zone of low flood potential shown on Figure A12. None of the streamflow records suitable for analysis was relevant to catchments lying entirely within this low-flood zone, consequently there was inadequate information from which to derive a design graph such as Figure A13 (indicating percentage runoff) for this region.

In the absence of local data, some guidance may be sought by referring to the tabulated values of the runoff coefficients presented in Figure A15. For rural areas the value of C is seen to depend on surface slope (C_s), soil permeability (C_p) and vegetal cover (C_v). A typical (small) catchment in the high flood zone could be associated with a surface slope of 3 to 10% (C_s = 0,06), a semi-permeable soil (C_p= 0,12) and a vegetal cover of sparse bush (C_v = 0,07), resulting in a C of 0,25 i.e. (0,06+0,12+0,07). On the other hand, a catchment of similar vegetal cover in the low flood zone might have a flat surface slope of < 3% (C_s = 0,01) and highly permeable soil cover (C_p= 0,03) yielding a C of 0,11 (i.e. 0,01+0,03+0,07). The ratio of the two C's, i.e. 0,11/0,25 = 0,44 could be used as a rough indication of the reduced surface runoff in the low zone compared with the

However, river bed losses, which are known to be highly significant in SWA-Namibia, are likely to be at their greatest in the flat, sandy areas of the low-flood regions. Therefore, it is not unreasonable to expect floods in the low region to be as much as an order of magnitude smaller relatively than floods in the high zone.

Estimation of the shape of a design hydrograph from a small catchment is often desirable where, for instance, a policy of deliberate ponding is adopted to attenuate flood peaks. Although the rational method gives one only the peak, a plausible hydrograph can be constructed by drawing a triangle of time-to-peak equal to time of concentration, T_c, and total base length equal to $2{,}6.T_c$. The area subtended by the hydrograph, viz. $1{,}3.T_c.Q_{peak}$, represents the volume of runoff which should be compared with the volume of causative rainfall to check that the proportion of rainfall contributing to direct runoff has been realistic.

In conclusion, it should be stressed that no amount of data, no matter how comprehensive, can entirely eliminate the need for judgement and careful evaluation of answers derived from generalizations. Design flood determination in SWA-Namibia, based as it is is on relatively meagre data, calls for an especially critical evaluation of results.

REFERENCES

Bauer, S.W. and Midgley, D.C. (1974): *A simple procedure for synthesizing direct runoff hydrographs.* Report No. 1/74, Hydrological Research Unit, University of the Witwatersrand, Johannesburg.

Constantinides, C.A. and Stephenson, D. (1981) : *Two-dimensional kinematic overland flow modelling.* Proc. Second Internat. Conf. Urban Storm Drainage, Urbana, Illinois, U.S.A.

Creager, W.P., Justin, J.D. and Hinds, J. (1964): *Engineering for dams.* Wiley.

Francou, J. and Rodier, J.A. (1967) : "Essai de classification des crues maximales". *Proceedings of the Leningrad Symposium on floods and their computation.* UNESCO.

Hydrological Research Unit (1972) : *Design flood determination in South Africa.* Report No. 1/72, Hydrological Research Unit, University of the Witwatersrand, Johannesburg.

Kovacs, Z.P. (1980): *Maximum flood peak discharges in South Africa - an empirical approach.* Technical Report No. 105, Department of Environmental Affairs, Pretoria.

Midgley, D.C., Pullen, R.A. and Pitman, W.V. (1969) : *Design flood determination in South Africa.* Report No. 4/69, Hydrological Research Unit, University of the Witwatersrand, Johannesburg.

Nash, J.E. (1959): "A note on the Muskingum flood routing method". *Journal of Geophysical Research.* Vol. 64, No. 8.

Pitman, W.V. and Midgley, D.C. (1967) : "Flood studies in South Africa - frequency analysis of peak discharges". *Trans. S. Afr. Instn. Civ. Engrs., August 1967.*

Pitman, W.V. (1980): *Analysis of large-area storms in SWA-Namibia.* Report No. 2/80, Hydrological Research Unit, University of the Witwatersrand, Johannesburg.

Richardson, B.F.C. and Midgley, D.C. (1979): *Analysis of SWA-Namibia rainfall data.* Report No. 3/79, Hydrological Research Unit, University of the Witwatersrand, Johannesburg.

Stephenson, D. (1981) : *Kinematic Flow - introduction and applications.* Report No. 2/81, Hydrological Research Unit, University of the Witwatersrand.

Watson, M.D. (1981a) : *Application of ILLUDAS to stormwater drainage design in South Africa.* Report No 1/81,

(1981b) : *Time-Area method of flood estimation for small catchments.* Report No. 7/81, Hydrological Research Unit, University of the Witwatersrand, Johannesburg.

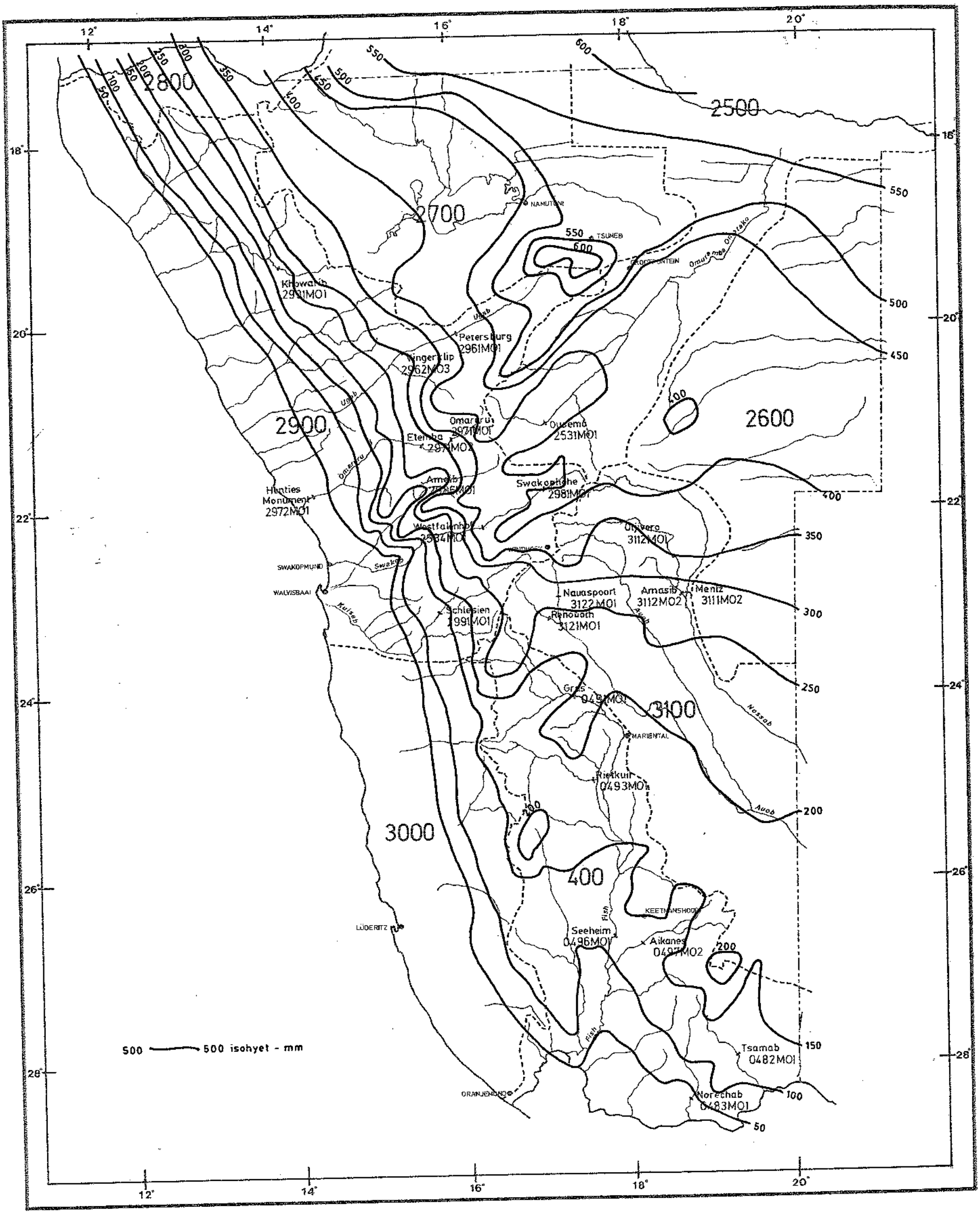

Figure A.1 Mean annual isohyetal map of SWA-Namibia

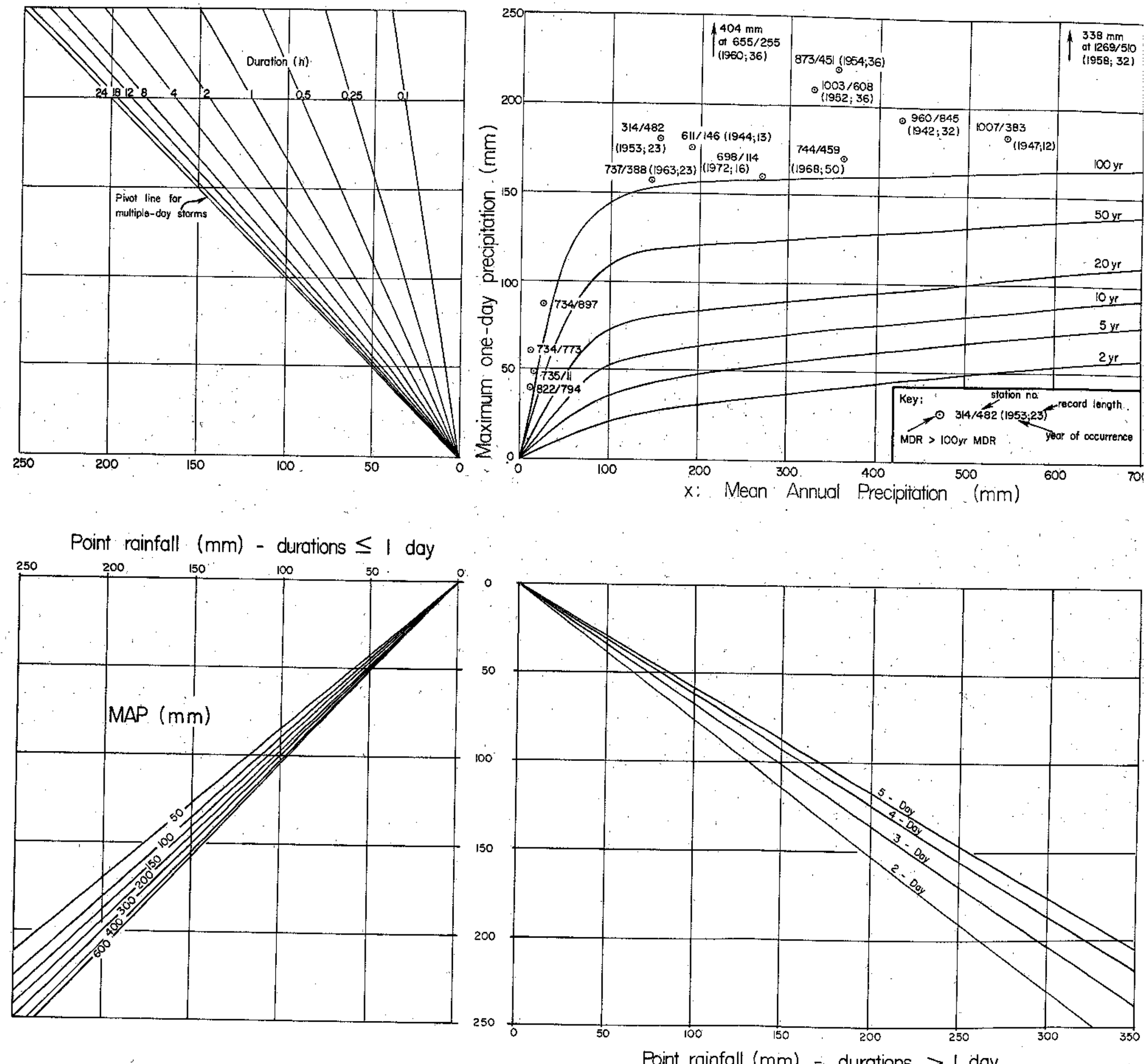

Figure A.2 Depth-duration-frequency relationship for point rainfall

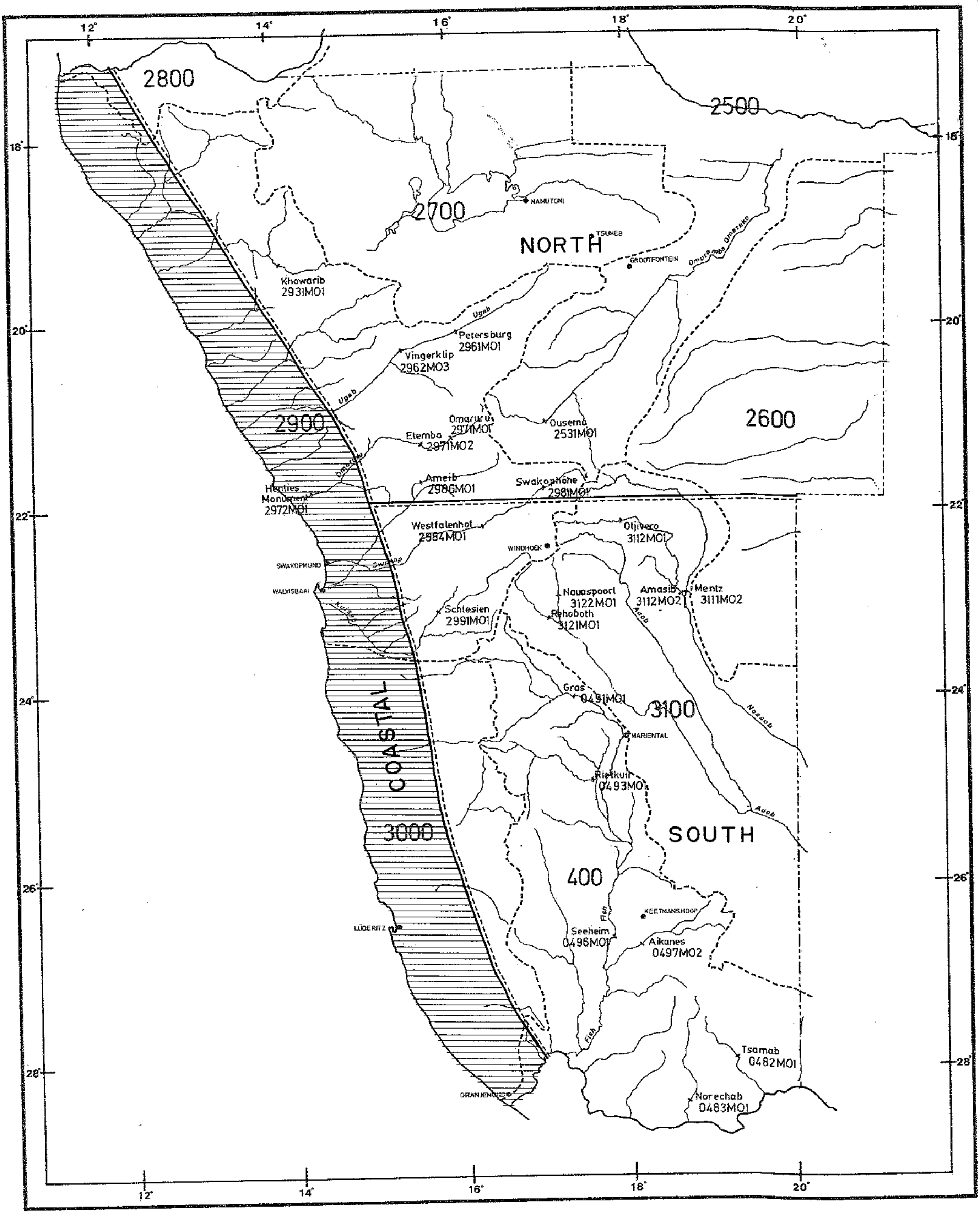

Figure A.3 Regional sub-division for large-area storm design

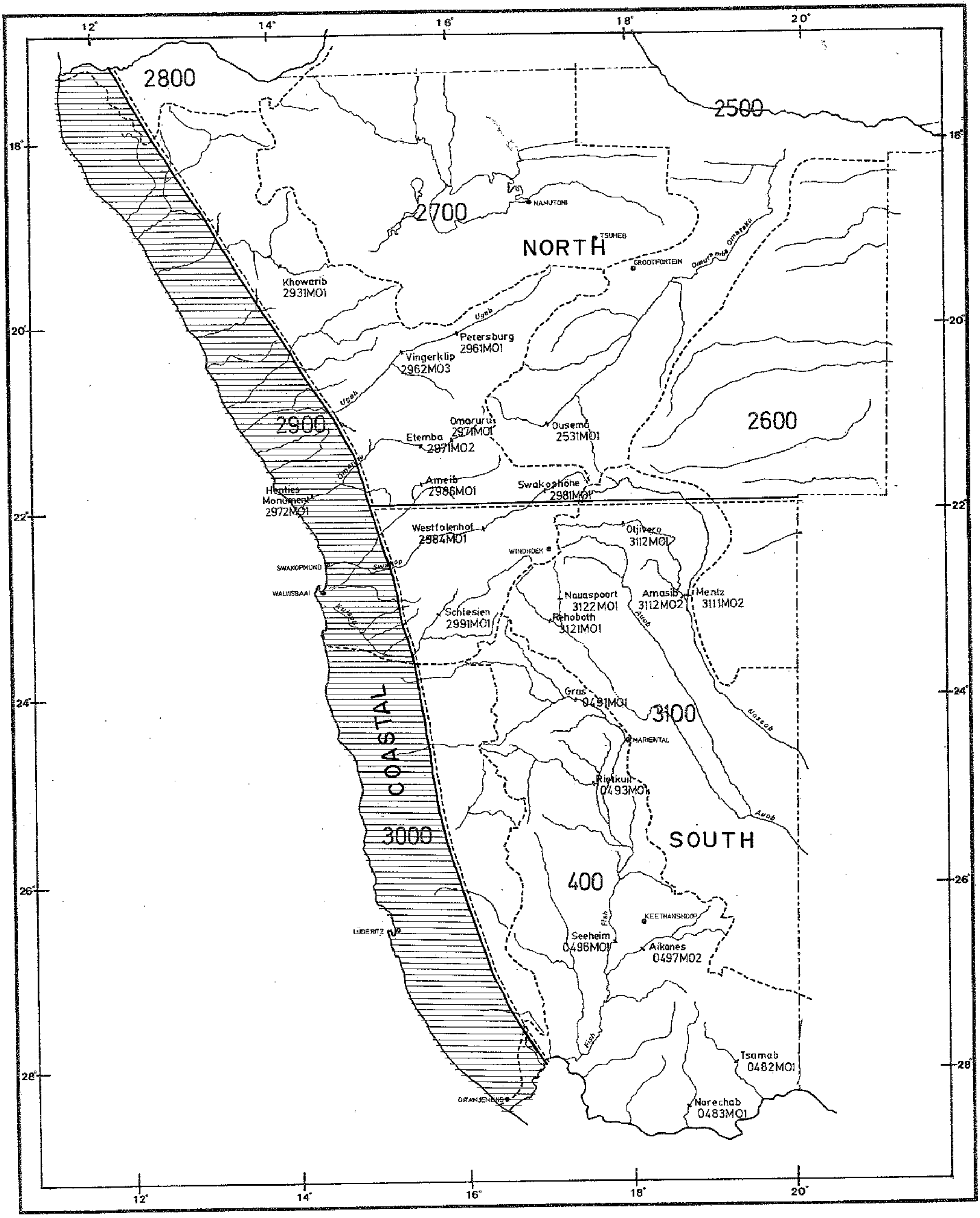

Figure A.3 Regional sub-division for large-area storm design

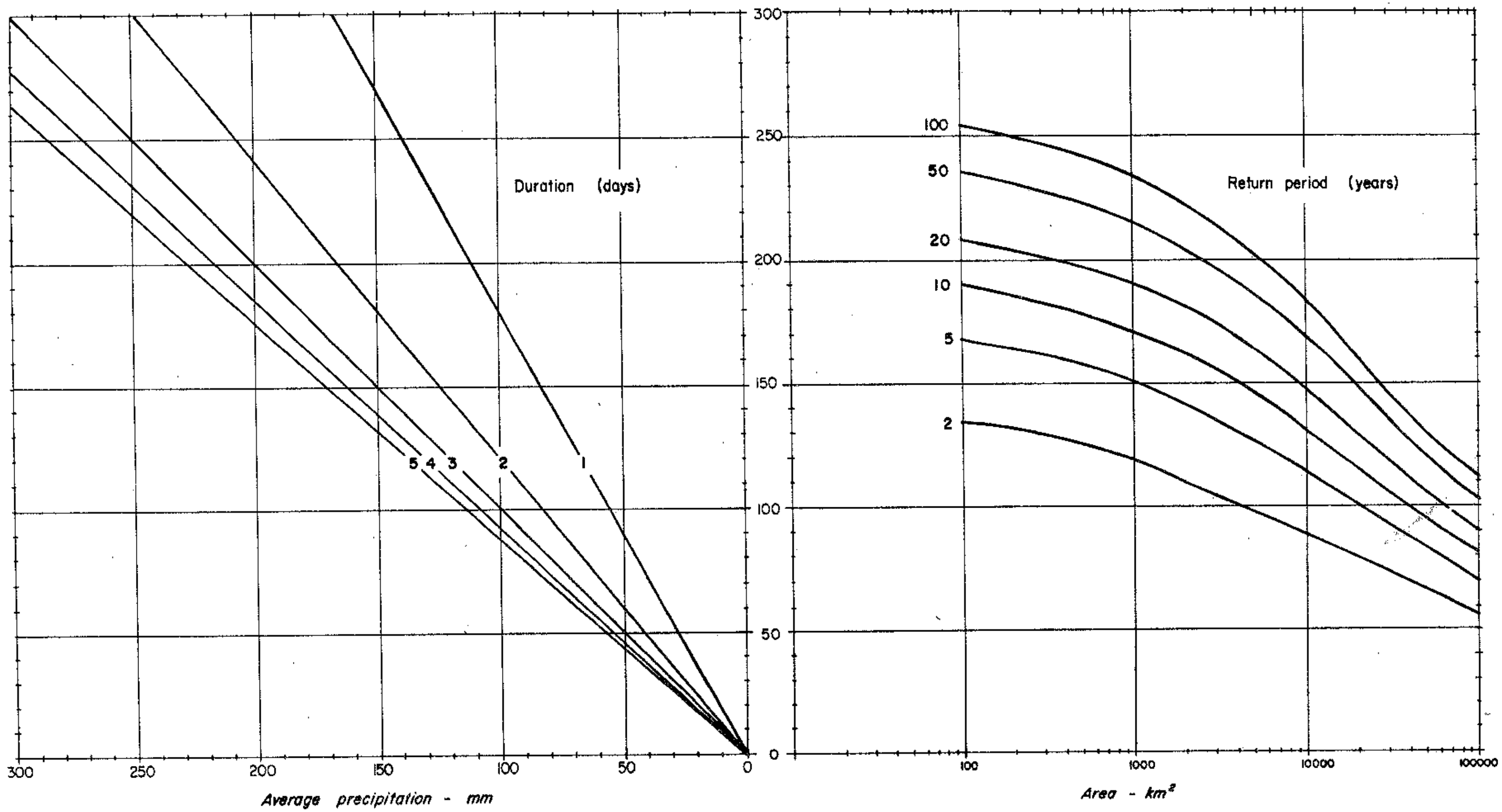

Figure A.4 Depth-duration-frequency-area relationship: North Region

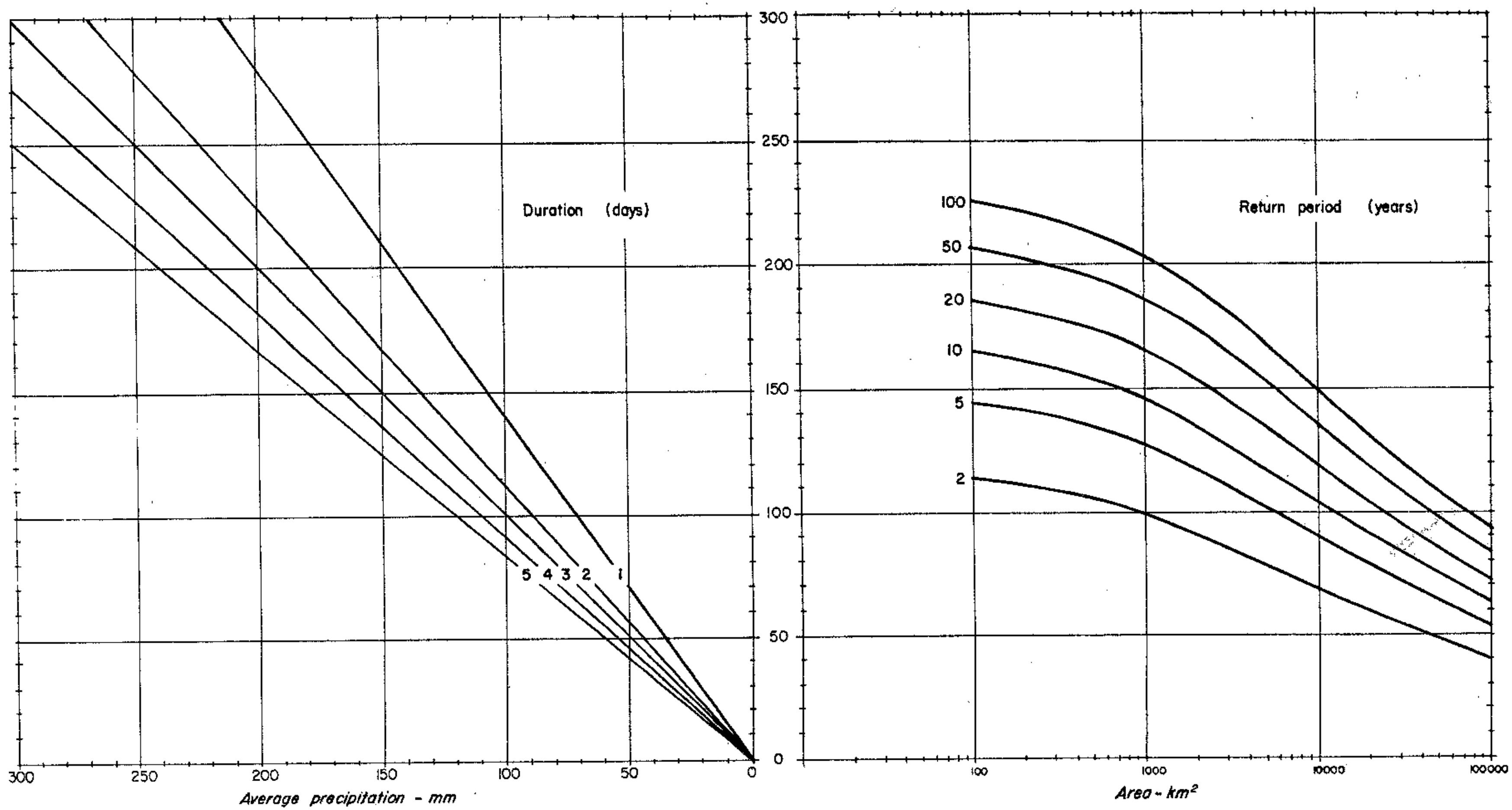

Figure A.5 Depth-duration-frequency-area relationship: South Region

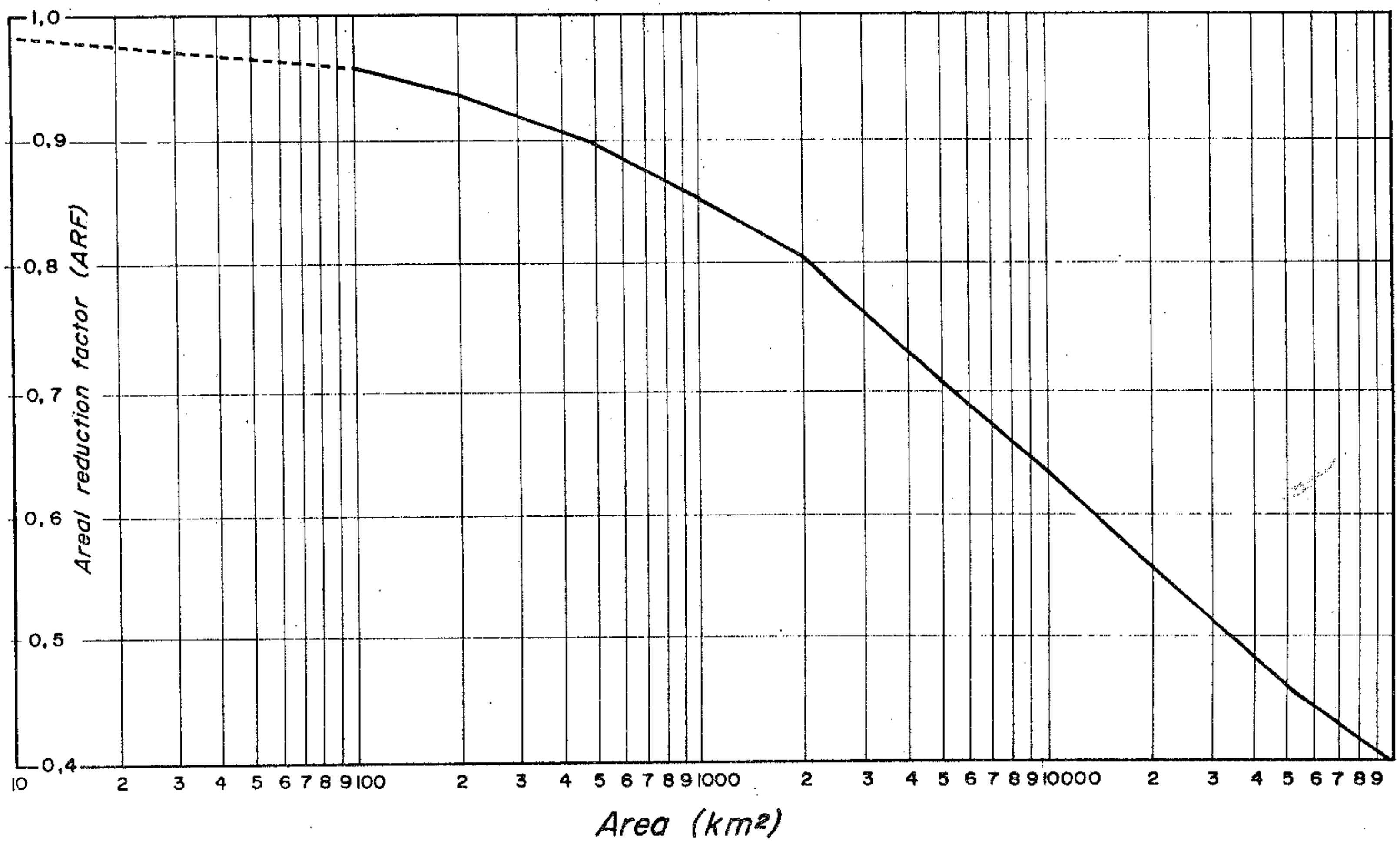

Figure A.6 Areal reduction factor

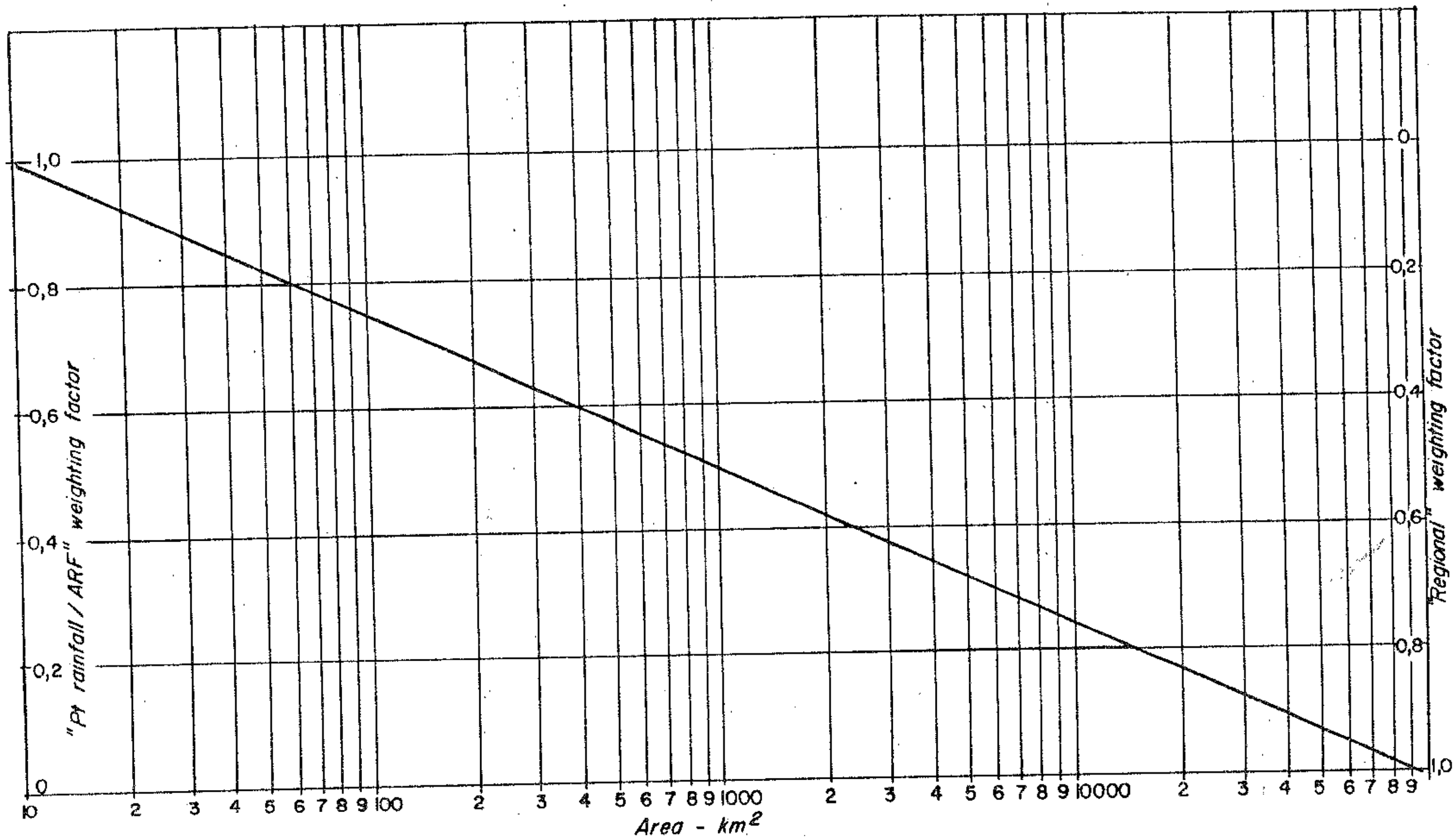

Figure A.7 Weighting factors: "Point rainfall/ARF" method and "Large area" method

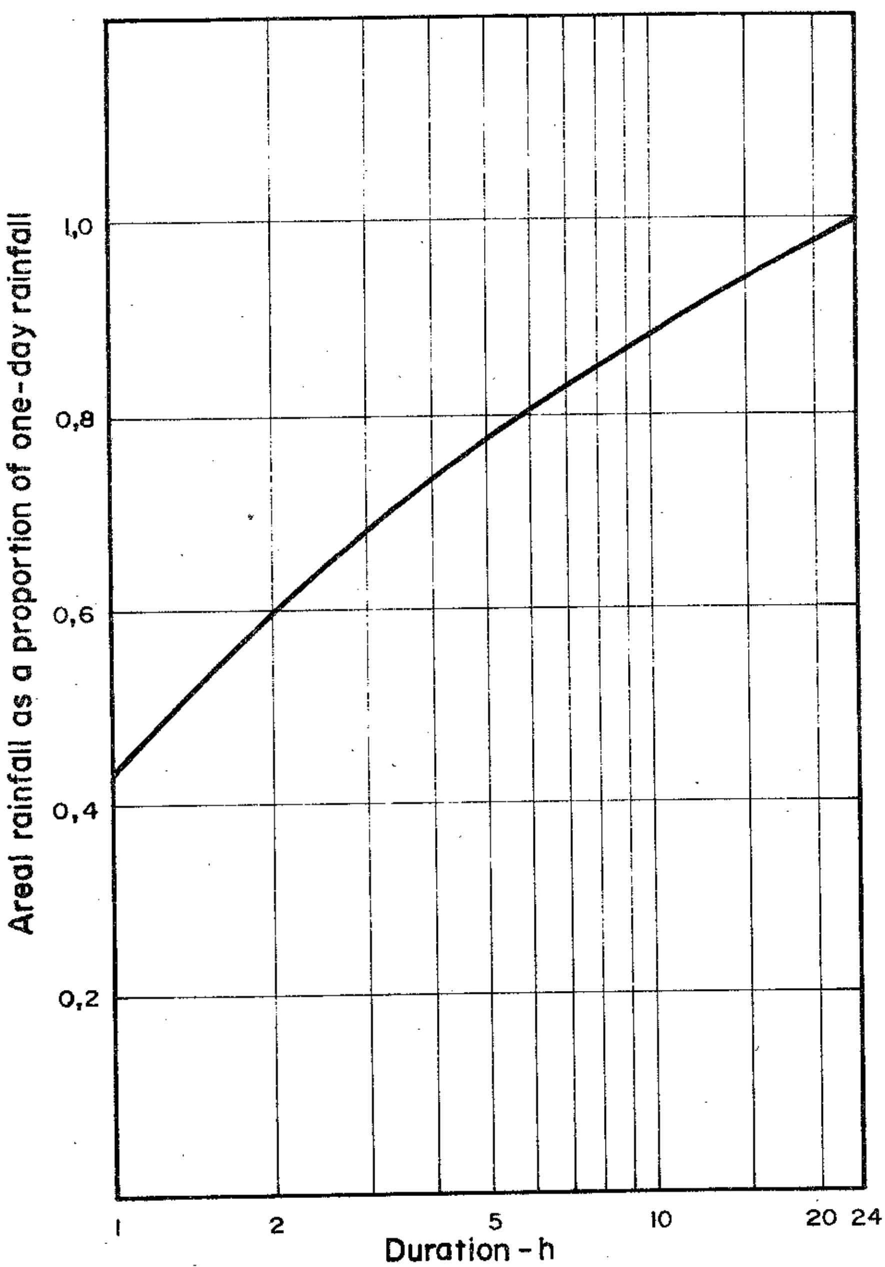

Figure A.8 Large-area storms: disaggregation of one-day rainfalls to shorter durations

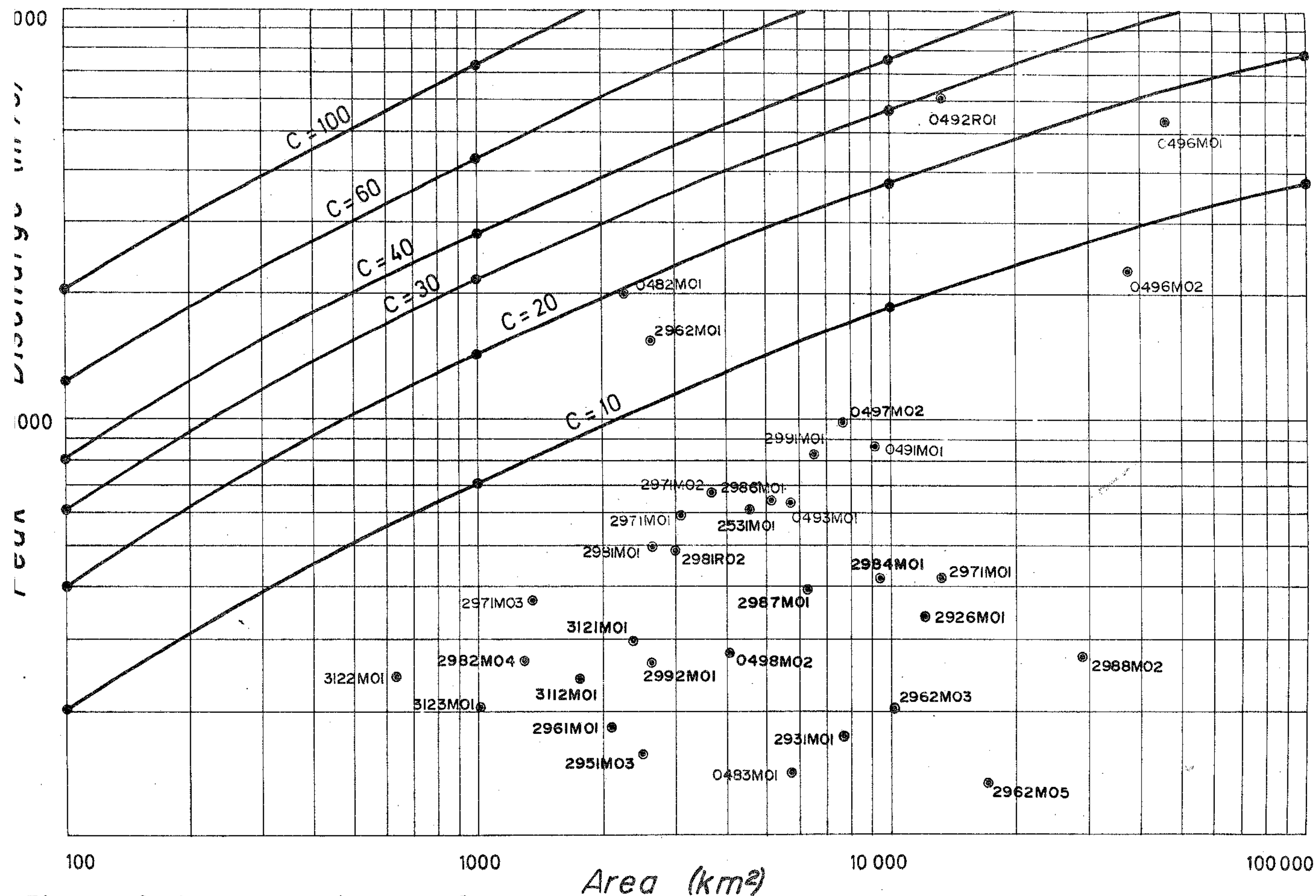

Figure A.9 Creager experience envelopes

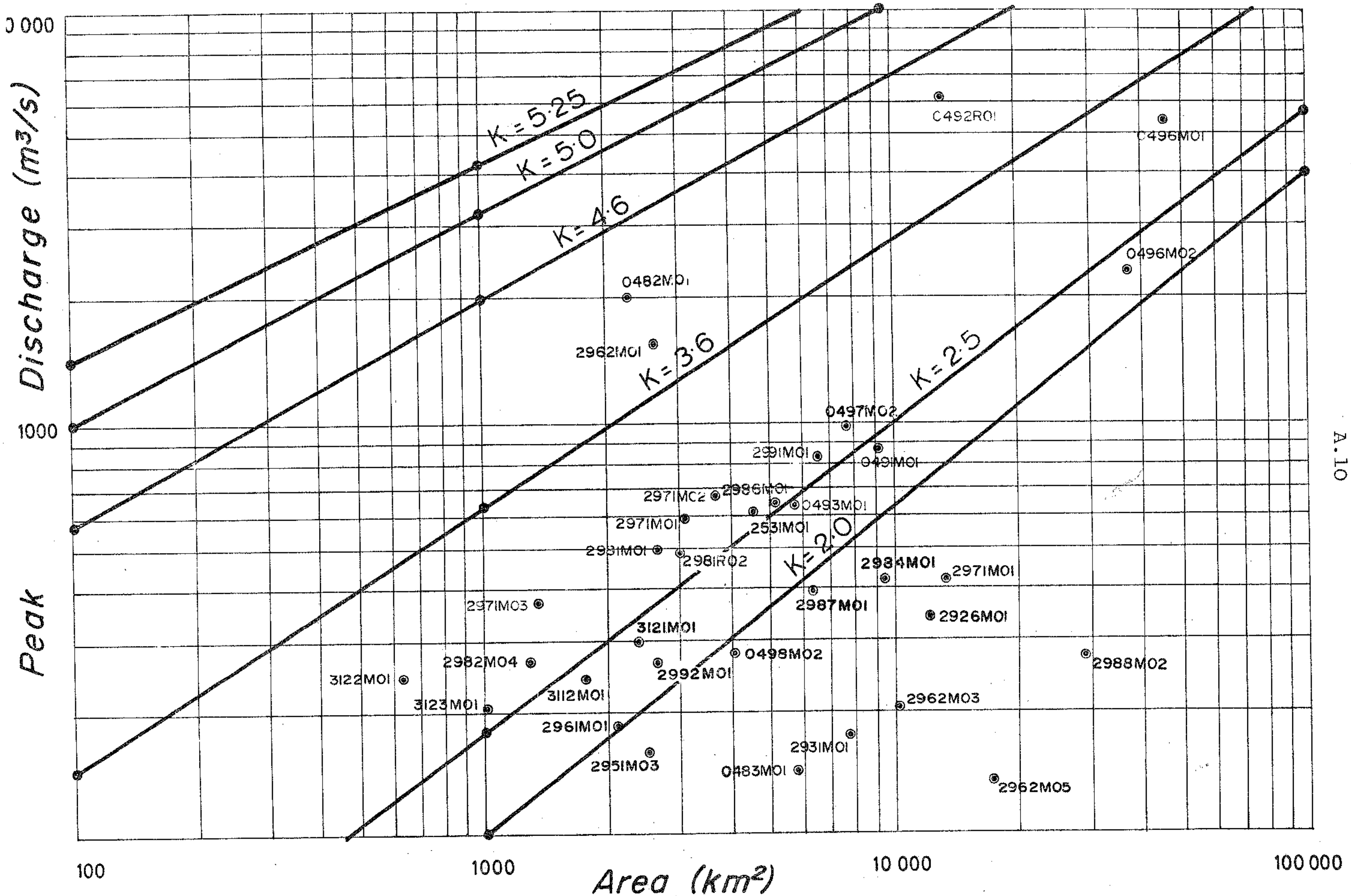

Figure A.10 Francou-Rodier experience envelopes

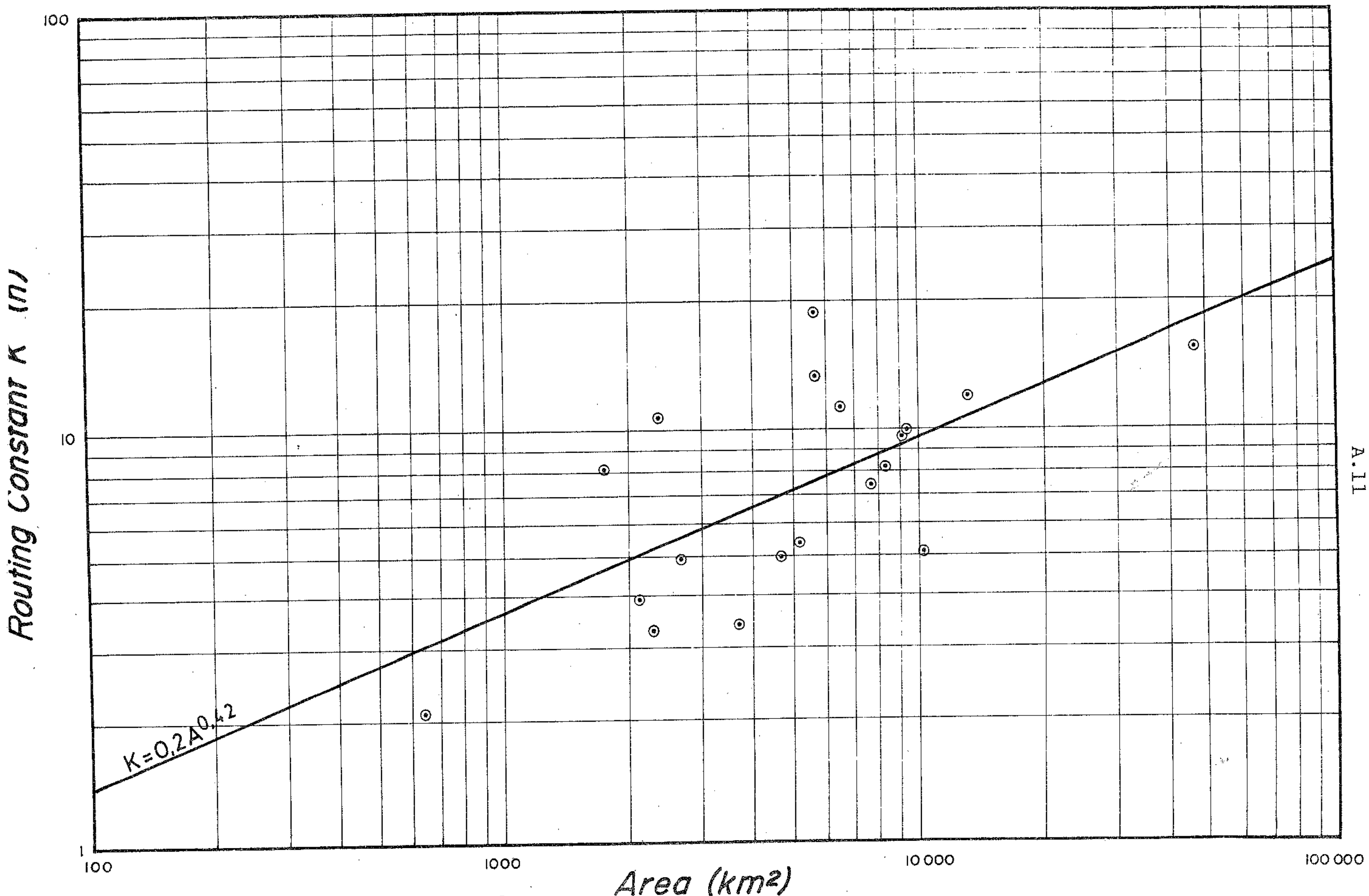

Figure A.11 Routing K as a function of area

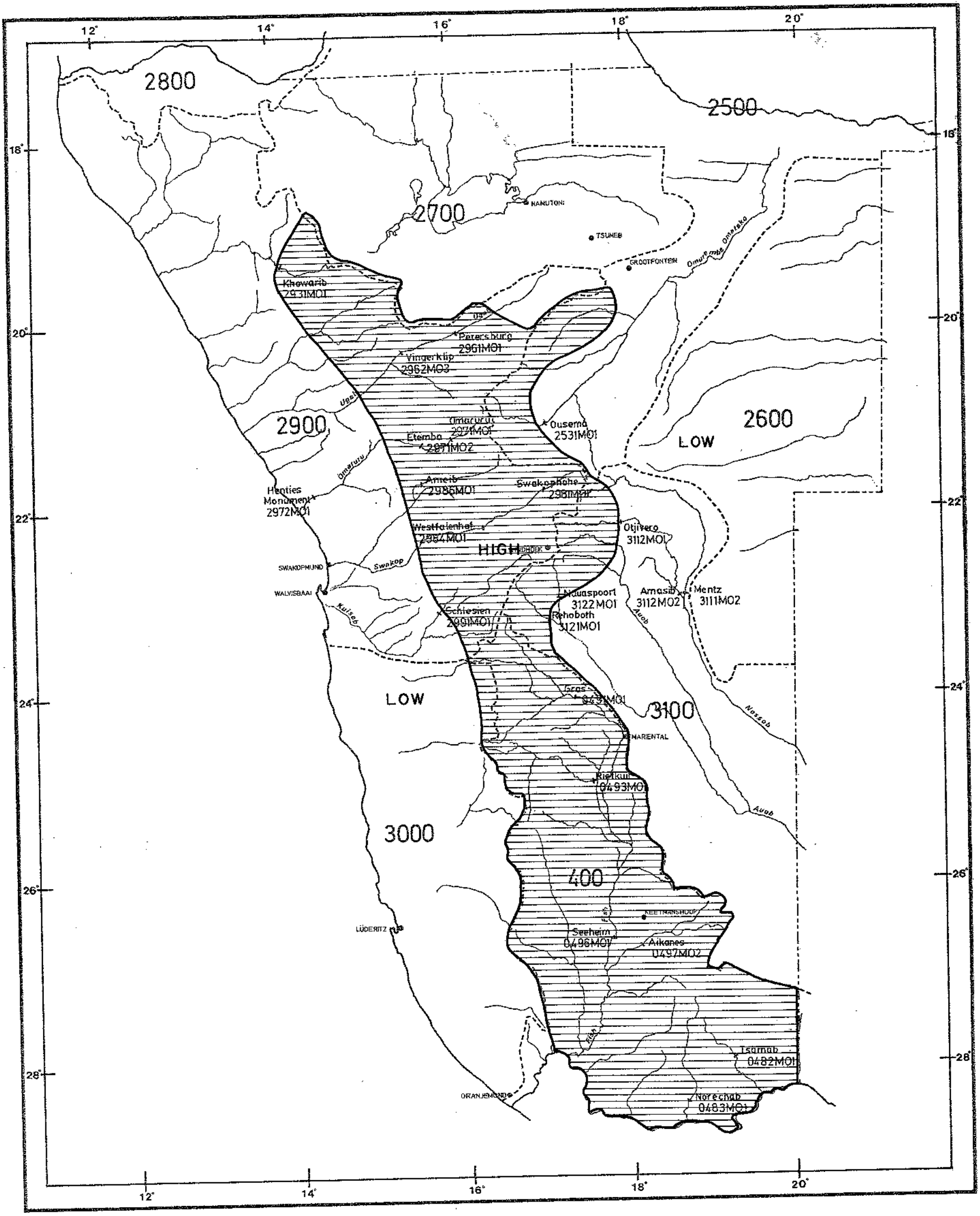

Figure A.12 Regional sub-division of flood potential

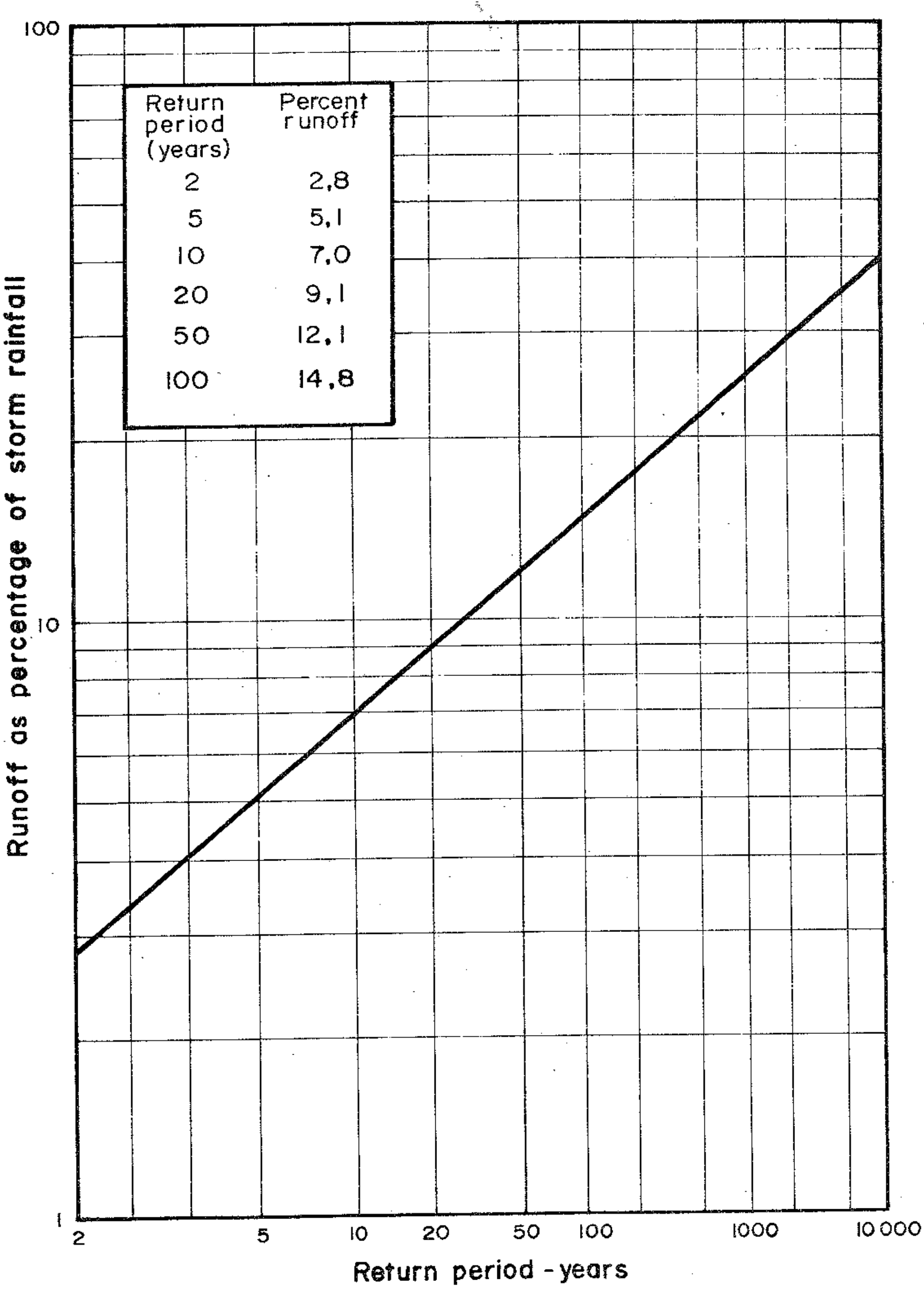

Figure A.13 Relationship between return period and percentage runoff

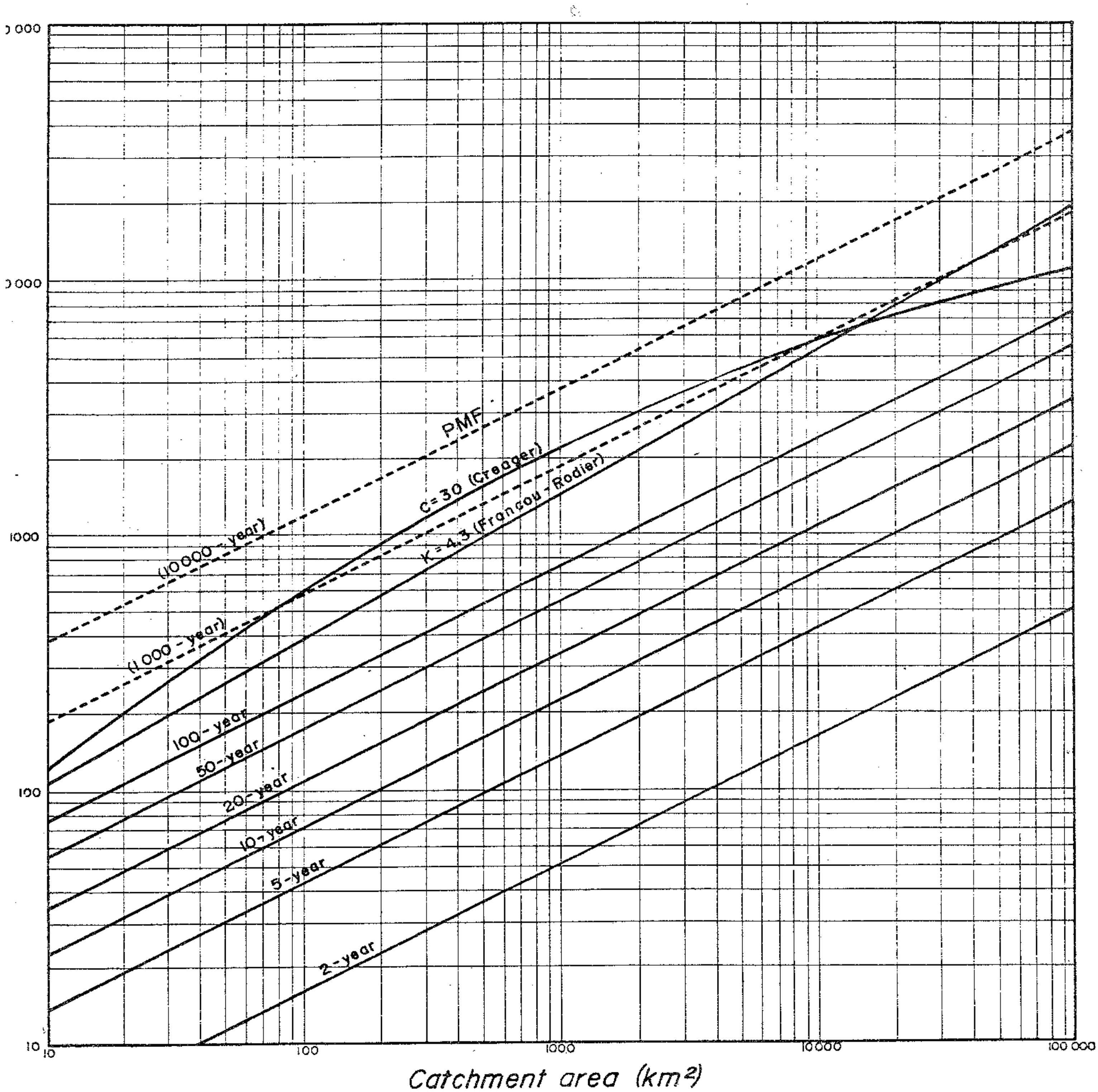

Figure A.14 Flood peak-frequency-catchment area relationship

Fig. A15 Recommended runoff coefficients, C, in the Rational Formula : $Q = CIA$

A. URBAN AREAS

Cover	C
LAWNS	
sandy, flat < 2%	0,05 - 0,10
sandy, steep > 7%	0,15 - 0,20
heavy soil, flat < 2%	0,13 - 0,17
heavy soil, steep > 7%	0,25 - 0,35
RESIDENTIAL	
single family area	0,30 - 0,50
apartment dwelling	0,50 - 0,70
INDUSTRIAL	
light industry areas	0,50 - 0,80
heavy industry areas	0,60 - 0,90
BUSINESS	
central	0,70 - 0,95
suburban	0,50 - 0,70
STREETS	0,70 - 0,95

Note: Time of concentration, $T_c = 0{,}96 \dfrac{L^{1,2}}{H^{0,2} . A^{0,1}}$ hours

L in km

H in m

A in km^2

Fig. A15 - cont.

B. RURAL AREAS

Component		Category	C
C_s	surface slope in %	< 3	0,01
		3 - 10	0,06
		10 - 30	0,12
		> 30	0,22
C_p	Permeability of soil	Very permeable (dolomite, gravel, coarse sand)	0,03
		Permeable (sandy soils)	0,06
		Semi-permeable (silt, clayey sand)	0,12
		Impermeable (clay, turf)	0,21
C_v	Vegetation	Dense bush	0,03
		Cultivated land, thin bush	0,07
		Grass land	0,17
		Bare surface	0,26

Notes:

i) $C' = C_s + C_p + C_v$

ii) Influence on C of return period T

T (years)	C
$\leq$ 20	0,67C'
50	0,83C'
100	1,00C'

iii) Time of concentration, $T_c = \left[\frac{0,87 . L^3}{H}\right]^{0,385}$ hours

L in km

H in m

APPENDIX B

B.1 PROGRAM DESCRIPTION

Program SWAHYD (see source listing, Table B.1) translates a stage hydrograph (in digital form) into a discharge hydrograph and calculates the flood volume. It then plots the stage- and discharge hydrographs on a CALCOMP plotter.

The program can accommodate up to 200 separate water level readings for any flood and up to 999 floods per gauging station.

The input data, as transcribed from records at the Directorate of Water Affairs, Windhoek, consists of two main categories, viz:

1. The Stage-Discharge rating equations
2. Time and Stage readings.

The Stage-Discharge rating equations

Table B.2 is a listing of the stage-discharge rating equations used in this report. It also provides, for every gauging station, the period of the record and the range of H (water level) for which each of the Q-equations is applicable.

NOTE:

Where the rating equations were not provided, they were derived by plotting discharge (Q) versus stage (H) on log-log paper. From the resulting straight lines, rating equations were obtained for different threshold values of H and the associated period of record for which the Q-equations were valid. This technique, more the exception than the rule, was applied at WESTFALENHOF.

The general form of the Q-equations, as used in program SWAHYD, is

```
Q = PARM(IA,1)*(H+PARM(IA,2))**PARM(IA,3)
```

for which

Q = discharge in m^3/s

H = water level in metres

IA = 1, 2 or 3, corresponding to the three threshold values for H

PARM = the three parameters in the Q-equation

e.g. At NORECHAB

If H = 0,9 metres, $Q = 51,5637H^{1,5590}$

Here IA = 1 and PARM(1,1) = 51,5637
PARM(1,2) = 0,0
PARM(1,3) = 1,5590

If $2,2 < H \leq 2,4$, $Q = 204,3730(H-1,10)^{2,9537}$

Here IA = 3 and PARM(3,1) = 204,3730
PARM(3,2) = -1,10
PARM(3,3) = 2,9537

The time and stage readings are coded in the form

DATE TIME WATER LEVEL

where

DATE is given by DAY, MONTH, YEAR

TIME is given as clock-time or time in minutes

WATER LEVEL is given as H metres.

Some criteria adopted by the program

In the computation of the discharge with respect to the three threshold values, the following inequalities were used:

i. if $H \leq$ THRESH(1) use the first Q-equation
ii. if THRESH(1) $< H \leq$ THRESH(2) use the second Q-equation
iii. if THRESH(2) $< H \leq$ THRESH(3) use the third Q-equation
iv. if $H >$ THRESH(3) use the third Q-equation and print the message "DISCHARGE RATING EXTRAPOLATED" alongside the result for Q.

Flood volume calculation

The flood volume for each flood is calculated in SWAHYD as follows:

if Q(L) and T(L) are the discharge and time at level L and Q(L-1) and T(L-1) are the discharge and time at level L-1
then $\Delta T = T(L) - T(L-1)$ in seconds
and VOL = Q(L)*ΔT/10**6 (million cubic metres)

B.2 INPUT DATA

Table B.3 is a listing of the input data for OTJIVERO.

The data as read by SWAHYD comprise 10 card types as described below in that order:

CARD 1: Header card

STATION NUMBER STATION NAME RIVER
FORMAT (2AH, HX, 16AH)

CARD 2: Total number of floods for this station (NS)
FORMAT(I3)

CARDS 3, 4 and 5: The threshold values for H and the Q-equation parameters

CARD 3: First Q-equation

THRESH(1) PARM(1,1) PARM(1,2) PARM(1,3)

Card 4: Second Q-equation

THRESH(2) PARM(2,1) PARM(2,2) PARM(2,3)

CARD 5: Third Q-equation

THRESH(3) PARM(3,1) PARM(3,2) PARM(3,3)

All have the format

FORMAT (F6.2,3F9.4)

CARD 6: Maximum discharge and maximum depth.

For each flood measured at a station, the flood peak was recorded. By inserting the highest flood peak (HMAX) in its associated Q-equation, we obtain (by hand calculation) the maximum recorded discharge (QMAX). After rounding, the values of QMAX and HMAX inputted give us the scale for the hydrograph plots thus ensuring that all hydrographs for a station will be plotted to the same scale.

FORMAT (F7.2,F5.1)

CARD 7: Time switch ICODE

FORMAT (I1)

ICODE = 0 if recorded time for this flood is given in MINUTES

= 1 if recorded time for this flood is given in CLOCK TIME

CARD 8: The streamflow data

IDAY, IMON, IYR, IHR, IMIN, H

where IDAY = day

IMON = month

IYR = year

IHR = hour

IMIN = minutes

H = water level

FORMAT(3I2, IX, 2I2, F5.2)

<u>NOTE</u>: SWAHYD plots the hydrographs one day at a time. Thus, in order to make the hydrographs continuous, the data set contains times 2400 hours and 0000 hours having the same water level denoting the end/start of each day. If time is in minutes, 1440 is used instead of 2400.

<u>CARD 9</u>: Flood terminating card
Contains 999999 starting in column 1 and indicates the end of a particular flood

<u>CARD 10</u>: As CARD 7 to begin the next flood and so on.

B.3 <u>OUTPUT</u>

Table B.4 is the paper output for the flood of 7-9 April 1972 measured at OTJIVERO the data for which appear in Table B.3.

The table shows:

i. the station identification
ii. date
iii. clock-time
iv. water level in metres
v. discharge in m^3/s
vi. flood volume.

Figure B.1 is the CALCOMP plotter output for the same flood. It plots Discharge and Level versus Clock-time.

Table B.1 Source listing of program SWAHYD

```
      DIMENSION STCODE(2),STNAME(8),RIVER(8),IBUF(1000),T(200),H(200),
     *          Q(200),FXHRS(4),IDAY(200),IMON(200),IYR(200),IMIN(200),
     *          IDATE(200),XDIST(4),IHR(200),JTIME(200),THRESH(3),
     *          PARM(3,3),AMESS(8)
      DATA  FXHRS  /4H0000,4H0600,4H1200,4H1800/
      DATA  XDIST  /-0.175,2.825,5.825,8.825/
      DATA  AMESS  /4HDISC,4HHARG,4HE RA,4HTING,4H EXT,4HRAPO,
     *               4HLATE,4HD    /
      N=0
      VOL=0.0
      READ(5,2) STCODE,STNAME,RIVER
    2 FORMAT(2A4,4X,16A4)
      READ(5,1) NS
    1 FORMAT(I3)
      DO 105 I=1,3
      READ(5,106) THRESH(I),(PARM(I,J),J=1,3)
  106 FORMAT(F6.2,3F9.4)
  105 CONTINUE
      READ(5,107) QMAX,HMAX
  107 FORMAT(F7.2,F5.1)
   18 READ(5,19) ICODE
   19 FORMAT(I1)
      WRITE(6,3) STCODE,STNAME,RIVER
    3 FORMAT('1',10X,'STATION CODE: ',2A4,10X,'STATION: ',8A4,8X,
     *       'RIVER: ',8A4/11X,13('*'),19X,8('*'),41X,6('*')//)
      WRITE(6,4)
    4 FORMAT(T8,' DATE ',10X,'CLOCK TIME',10X,'WATER LEVEL(METRES)',
     *     10X,'DISCHARGE(CUMECS)'/T9,4('='),11X,10('='),10X,19('='),
     *     10X,17('=')//)
      ISW=0
      MSW=0
      DO 5 I=1,200
      T(I)=0.0
      H(I)=0.0
      Q(I)=0.0
    5 CONTINUE
      CALL PLOTS(IBUF,4000)
      CALL PLOT(2.0,2.0,-3)
      CALL NEWPEN(1)
      CALL SYMBOL(0.0,22.4,0.4,'STATION NO. ',0.0,+12)
      CALL SYMBOL(3.5,22.4,0.6,STCODE,0.0,+8)
      CALL SYMBOL(12.5,22.4,0.4,'STATION:',0.0,+8)
      CALL SYMBOL(15.5,22.4,0.6,STNAME,0.0,+30)
      CALL SYMBOL(26.5,22.4,0.4,'RIVER:',0.0,+6)
      CALL SYMBOL(29.5,22.4,0.6,RIVER,0.0,+30)
      CALL NEWPEN(2)
      CALL SYMBOL(26.5,21.3,0.3,'LEVEL (METRES)      +++++++',
     *            0.0,+24)
      CALL NEWPEN(3)
      CALL SYMBOL(26.5,20.9,0.3,'DISCHAREGE(CUMECS) -------',
     *            0.0,+24)
      CALL NEWPEN(1)
      DISUPC = QMAX/20.0
      CALL AXIS(0.0,0.0,'DISCHARGE IN CUMECS',+19,20.0,90.0,
     *          0.0,DISUPC,10.0)
      CALL PLOT(0.0,0.0,-3)
      CALL PLOT(2.5,0.0,-2)
      HUPC = HMAX/20.0
      CALL AXIS(0.0,0.0,'LEVEL IN METRES',+15,20.0,90.0,0.0,HUPC,10.0)
      READ(5,7) IDAY(1),IMON(1),IYR(1),IHR(1),IMIN(1),H(1)
```

```
  7 FORMAT(3I2,1X,2I2,F5.2)
 15 IDATE(1)=IDAY(1)*10**4 + IMON(1)*100 + IYR(1)
    DO 8 J=2,200
    READ(5,7) IDAY(J),IMON(J),IYR(J),IHR(J),IMIN(J),H(J)
    IDATE(J)=IDAY(J)*10**4 + IMON(J)*100 + IYR(J)
    IF (IDATE(J) .EQ. 999999) GO TO 999
    IF (IDATE(J) .NE. IDATE(J-1)) GO TO 9
  8 CONTINUE
  9 ID = IDAY(J)
    IM = IMON(J)
    IY = IYR(J)
    IH = IHR(J)
    IMI= IMIN(J)
    HT = H(J)
 16 KK = J-1
    DO 10 K=1,KK
    IF (ICODE.EQ.1) GO TO 11
    T(K)=(IHR(K)*100.+IMIN(K))*12./1440.
    GO TO 12
 11 T(K)=(IHR(K)*60.+IMIN(K))*12./1440.
 12 IF(H(K).LE.THRESH(3)) GO TO 13
    MSW=1
    IA=3
    GO TO 75
 13 IF (H(K).LE.THRESH(1)) IA=1
    IF (H(K).GT.THRESH(1) .AND. H(K).LE.THRESH(2)) IA=2
    IF (H(K).GT.THRESH(2) .AND. H(K).LE.THRESH(3)) IA=3
 75 Q(K)=PARM(IA,1)*(H(K)+PARM(IA,2))**PARM(IA,3)
    IF (ICODE.EQ.1) GO TO 30
    JTIME(K)=IHR(K)*100+IMIN(K)
    IHR(K)=JTIME(K)/60
    IMIN(K)=JTIME(K)-IHR(K)*60
 30 IF (MSW.EQ.1) GO TO 31
    IF (K.EQ.1)
   *WRITE(6,14) IDAY(K),IMON(K),IYR(K),IHR(K),IMIN(K),H(K),Q(K)
 14 FORMAT(T7,I2,'/',I2,'/',I2,12X,I2,':',I2,19X,F5.2,21X,F8.2)
    IF (K.NE.1) WRITE(6,25) IHR(K),IMIN(K),H(K),Q(K)
 25 FORMAT(T27,I2,':',I2,19X,F5.2,21X,F8.2)
    GO TO 10
 31 IF(K.EQ.1)
   *WRITE(6,76)IDAY(K),IMON(K),IYR(K),IHR(K),IMIN(K),H(K),Q(K),
   *            (AMESS(IP),IP=1,8)
 76 FORMAT(T7,I2,'/',I2,'/',I2,12X,I2,':',I2,19X,F5.2,21X,F8.2,9X,8A4)
    IF(K.NE.1)WRITE(6,77)IHR(K),IMIN(K),H(K),Q(K),(AMESS(IP),IP=1,8)
 77 FORMAT(T27,I2,':',I2,19X,F5.2,21X,F8.2,9X,8A4)
    MSW=0
 10 CONTINUE
    JJ=J
    IF(ISW.EQ.1) JJ=KK
    IF( ICODE.EQ.1) GO TO 45
 43 JTIME(J)=IHR(J)*100+IMIN(J)
    IHR(J)=JTIME(J)/60
    IMIN(J)=JTIME(J)-IHR(J)*60
 45 IHR(J)=IHR(J)+24
    DO 200 L=2,JJ
    DELT = (IHR(L)*3600+IMIN(L)*60)-(IHR(L-1)*3600+IMIN(L-1)*60)
    VOL = VOL + Q(L-1)*DELT/10**6
200 CONTINUE
    T(KK+1)=0.0
    T(KK+2)=1.0
```

```
      H(KK+1)=0.0
      H(KK+2)=HUPC
      G(KK+1)=0.0
      G(KK+2)=DISUPC
      CALL GRID(0.0,0.0,3.0,2.0,4,10)
      CALL PLOT(0.0,0.0,+3)
      DO 300 K=1,4
      CALL SYMBOL(XDIST(K),-0.3,0.2,FXHRS(K),0.0,+4)
  300 CONTINUE
      FD=IDAY(1)*1.
      FM=IMON(1)*1.
      FY=IYR(1)*1.
      CALL NUMBER(5.1,-0.9,0.3,FD,0.0,-1)
      CALL SYMBOL(5.55,-0.9,0.0,'/',0.0,+1)
      CALL NUMBER(5.775,-0.9,0.3,FM,0.0,-1)
      CALL SYMBOL(6.225,-0.9,0.3,'/',0.0,+1)
      CALL NUMBER(6.45,-0.9,0.3,FY,0.0,-1)
      CALL NEWPEN(2)
      CALL LINE(T,H,KK,1,0,0)
      CALL LINE(T,H,KK,1,-1,3)
      CALL NEWPEN(3)
      CALL LINE(T,G,KK,1,0,0)
      CALL NEWPEN(1)
      CALL PLOT(12.0,0.0,-3)
      IF(ISW.EQ.1) GO TO 17
      IDAY(1)=ID
      IMON(1)=IM
      IYR(1) =IY
      IHR(1) =IH
      IMIN(1)=IMI
      H(1)   =HT
      GO TO 15
  999 ISW=1
      GO TO 16
   17 CALL PLOT(65.0,0.0,999)
      WRITE(6,47) VOL
   47 FORMAT(//////////,T20,'FLOOD VOLUME =',F8.2,1X,
     *      '* 10**6 CUBIC METRES'/T20,12('*'))
      VOL=0.0
      N=N+1
      IF(N.LT.NS) GO TO 18
      STOP
      END
```

Table B.2 Stage-discharge rating equations

Station Ref. No.	Station name	River	Period of record where Q-equations applicable	Threshold values for stage height H (m) from	to	Discharge rating equations (Q in m³/s)
0482M01	Tsamab	Ham	Entire record	0	1,1	$Q = 53{,}0726^{1{,}5557}$
				1,1	3,01	$Q = 2{,}2464(H+1{,}22)^{3{,}9380}$
0483M01	Norechab	Hom	Entire record	0	0,9	$Q = 51{,}5637H^{1{,}5590}$
				0,9	2,2	$Q = 131{,}3175(H-0{,}44)^{1{,}5691}$
				2,2	2,4	$Q = 204{,}3730(H-1{,}10)^{2{,}9537}$
0491M01	Gras	Fish	Entire record	0	0,5	$Q = 140{,}9466H^{1{,}5169}$
				0,5	1,4	$Q = 203{,}0754(H-0{,}07)^{1{,}7713}$
				1,4	4,0	$Q = 182{,}4844H^{1{,}8208}$
0493M01	Rietkuil	Hutup	From 27.11.1973 to date	0	0,7	$Q = 26{,}7014H^{2{,}5338}$
				0,7	2,4	$Q = 25{,}4192(H+0{,}03)^{2{,}6547}$
				2,4	4,1	$Q = 85{,}2274(H-0{,}52)^{1{,}8183}$
0496M01	Seeheim	Fish	From 11.08.70 to end 1973/74 season	0	0,4	$Q = 421{,}1949H^{1{,}5094}$
				0,4	1,9	$Q = 474{,}53(H-0{,}01)^{1{,}6041}$
				1,9	2,5	$Q = 327{,}5622(H+0{,}12)^{1{,}9535}$
0497M02	Aikanes	Löwen	From 27.01.76 to date	0	1,1	$Q = 61{,}7368H^{1{,}5274}$
				1,1	1,6	$Q = 56{,}1164(H+0{,}03)^{2{,}0048}$
				1,6	3,0	$Q = 67{,}3416(H-0{,}07)^{1{,}8745}$
2531M01	Ousema	Omuramba Omatako	From 11.08.70 to date	0	0,6	$Q = 44{,}7898H^{1{,}5927}$
				0,6	1,5	$Q = 53{,}6477(H+0{,}05)^{2{,}3270}$
				1,5	2,5	$Q = 47{,}1306H^{2{,}7474}$
2961M01	Petersburg	Ugab	Entire record	0	1,7	$Q = 16{,}9155H^{1{,}5339}$
				1,7	3,0	$Q = 30{,}9808(H-0{,}59)^{1{,}5363}$
				3,0	3,5	$Q = 1{,}44168E-13(H+10{,}91)^{13{,}0144}$
2962M03	Vingerklip	Ugab	Entire record	0	2,0	$Q = 62{,}5357H^{1{,}5611}$
				2,0	2,6	$Q = 0{,}0001(H+8{,}91)^{5{,}8794}$
				2,6	3,0	$Q = 32{,}3204(H+0{,}31)^{1{,}9412}$
2971M02	Etemba	Omaruru	From 1970 to date	0	0,5	$Q = 81{,}2941H^{1{,}5490}$
				0,5	1,5	$Q = 81{,}2088(H+0{,}01)^{1{,}5897}$
				1,5	4,0	$Q = 85{,}1681(H-0{,}03)^{1{,}5706}$
2972M01	Henties Monument	Omaruru	From October 1973 to date	0	0,5	$Q = 39{,}0913H^{1{,}5758}$
				0,5	1,2	$Q = 4{,}6669(H+0{,}81)^{4{,}5155}$
				1,2	4,0	$Q = 118{,}0201(H-0{,}22)^{1{,}7748}$

Station Ref.No.	Station name	River	Period of record where Q-equations applicable	Threshold values for stage height H (m) from	to	Discharge rating equation (Q in m^3/s)
2981M01	Swakophöhe	Swakop	Entire record	0	0,42	$Q = 21{,}7940H^{1,5849}$
				0,42	1,31	$Q = 20{,}5883(H+0{,}07)^{1,8353}$
				1,31	4,4	$Q = 31{,}1673(H-0{,}21)^{1,8544}$
2984M01	Westfalen-hof	Swakop	Before 1970	0	0,99	$Q = 62{,}7336H^{1,5263}$
				0,99	4,0	$Q = 62{,}9506H^{1,6249}$
			From 14.08.70	0	1,1	$Q = 80{,}3039H^{1,5792}$
			to date	1,1	3,0	$Q = 59{,}8064(H+0{,}18)^{1,8858}$
				3,0	4,0	$Q = 29{,}8724(H+0{,}17)^{2,5234}$
2986M01	Ameib	Kahn	From 14.08.70	0	0,6	$Q = 96{,}2624H^{1,5821}$
			to date	0,6	1,1	$Q = 92{,}1442\ (H+0{,}05)^{1,7528}$
				1,1	3,0	$Q = 70{,}9113(H+0{,}20)^{1,9468}$
2991M01	Schlesien	Kuiseb	From start to	0	0,5	$Q = 37{,}53H^{1,5419}$
			30.08.75	0,5	6,0	$Q = 50{,}2376(H+0{,}03)^{2,0755}$
			From 1975/76	0	0,5	$Q = 37{,}53H^{1,5419}$
			season only	0,5	2,6	$Q = 56{,}0683(H-0{,}02)^{1,9879}$
				2,6	4,5	$Q = 28{,}5612(H+0{,}39)^{2,3406}$
			From 1976/7	0	0,63	$Q = 30{,}127H^{2,5789}$
			season to date	0,63	1,6	$Q = 104{,}1154(H-0{,}37)^{1,7975}$
				1,6	6,0	$Q = 74{,}1619(H-0{,}18)^{2,0286}$
3111M02	Mentz	Black Nossob	Entire record	0	0,5	$Q = 2{,}6976H^{1,7299}$
				0,5	1,19	$Q = 25{,}3703(H-0{,}10)^{3,7812}$
				1,19	4,00	$Q = 56{,}8406(H-0{,}41)^{1,9369}$
3112M01	Otjivero	White Nossob	Entire record	0	0,3	$Q = 371{,}4976H^{3,1268}$
				0,3	1,4	$Q = 42{,}9138(H+0{,}03)^{1,6669}$
				1,4	3,0	$Q = 5{,}3817(H+1{,}31)^{2,7045}$
3112M02	Amasib	White Nossob	From start to	0	0,5	$Q = 24{,}1808H^{2,6799}$
			30.09.75	0,5	1,0	$Q = 57{,}4177(H-0{,}29)^{1,6650}$
				1,0	1,6	$Q = 116{,}4875(H-0{,}85)^{0,7566}$
			From 01.10.75	0	0,3	$Q = 9{,}1348H^{1,5456}$
			to 30.09.76	0,3	1,1	$Q = 10{,}9368(H+0{,}31)^{3,9149}$
				1,1	2,3	$Q = 62{,}1096(H-0{,}3)^{1,8587}$

Station Ref.No.	Station name	River	Period of record where Q-equations applicable	Threshold values for stage height H (m) from	to	Discharge rating equation (Q in m^3/s)
3121M01	Rehoboth	Haris	Entire record	0	0,6	$Q = 17,5045H^{1,6046}$
				0,6	1,3	$Q = 15,5402(H+010)^{1,9210}$
				1,3	3,0	$Q = 10,8886(H+0,14)^{2,7018}$
3122M01	Nauaspoort	Usib	Entire record	0	0,6	$Q = 18,9434H^{1,7099}$
				0,6	2,8	$Q = 18,0524(H+0,09)^{2,0857}$
				2,8	4,0	$Q = 1,14811E\text{-}08(H+11,1)^{8,8787}$

Table B.3 Listing of SWAHYD input data for Otjivero

```
3112M01     OTJIVERO                              WHITE NOSSOB
  3
  0.30 371.4976   0.0000   3.1258
  1.40  42.9138   0.0300   1.6669
  3.00   5.3817   1.3100   2.7045
400.00   3.0
1
250371 0230 0.09
250371 0245 0.19
250371 0300 0.21
250371 0400 0.22
250371 0600 0.24
250371 0700 0.28
250371 0800 0.27
250371 0900 0.25
250371 1000 0.23
250371 1200 0.21
250371 1500 0.20
250371 2100 0.18
250371 2300 0.38
250371 2315 0.59
250371 2330 0.66
250371 2345 0.73
250371 2400 0.78
260371 0000 0.78
260371 0015 0.82
260371 0030 0.84
260371 0045 0.87
260371 0100 0.93
260371 0115 0.97
260371 0130 1.03
260371 0145 0.99
260371 0200 0.98
260371 0215 0.97
260371 0230 0.95
260371 0245 0.97
260371 0300 0.86
260371 0315 0.78
260371 0330 0.73
260371 0345 0.70
260371 0400 0.68
260371 0415 0.65
260371 0430 0.61
260371 0445 0.59
260371 0500 0.56
260371 0530 0.52
260371 0600 0.49
260371 0700 0.45
260371 0800 0.43
260371 0900 0.39
260371 1100 0.37
260371 1200 0.35
260371 1400 0.32
260371 1700 0.29
260371 1900 0.27
260371 2000 0.26
260371 2100 0.25
260371 2200 0.23
260371 2400 0.21
270371 0000 0.21
270371 0200 0.19
270371 0400 0.18
270371 0500 0.17
270371 0700 0.16
270371 0800 0.15
270371 1100 0.13
270371 1300 0.12
270371 1500 0.11
270371 1800 0.09
270371 2400 0.07
280371 0000 0.07
280371 0600 0.05
280371 1200 0.03
280371 1800 0.01
999999
```

```
1
070472 0400 0.04
070472 0410 0.22
070472 0420 0.55
070472 0430 0.69
070472 0440 0.76
070472 0450 0.90
070472 0500 1.15
070472 0510 1.45
070472 0520 1.67
070472 0530 1.79
070472 0540 1.83
070472 0550 1.90
070472 0600 2.04
070472 0610 2.30
070472 0620 2.43
070472 0630 2.57
070472 0640 2.62
070472 0650 2.66
070472 0700 2.68
070472 0745 2.74
070472 0800 2.78
070472 0815 2.73
070472 0830 2.66
070472 0845 2.57
070472 0900 2.41
070472 0930 2.21
070472 1000 2.00
070472 1030 1.81
070472 1100 1.66
070472 1130 1.53
070472 1200 1.43
070472 1230 1.33
070472 1300 1.23
070472 1330 1.15
070472 1400 1.07
070472 1430 0.99
070472 1500 0.91
070472 1600 0.82
070472 1700 0.78
070472 1800 0.77
070472 1900 0.69
070472 2000 0.64
070472 2100 0.63
070472 2200 0.65
070472 2300 0.67
070472 2400 0.65
080472 0000 0.65
080472 0100 0.60
080472 0200 0.56
080472 0300 0.53
080472 0400 0.49
080472 0500 0.48
080472 0600 0.47
080472 0700 0.44
080472 0800 0.40
080472 0900 0.42
080472 1000 0.44
080472 1100 0.43
080472 1200 0.44
080472 1300 0.43
080472 1400 0.41
080472 1500 0.40
080472 1600 0.37
080472 1700 0.35
080472 1800 0.34
080472 1900 0.32
080472 2000 0.31
080472 2300 0.30
080472 2400 0.28
090472 0000 0.28
090472 0200 0.27
090472 0400 0.26
090472 0600 0.25
090472 1400 0.24
090472 1600 0.23
090472 1800 0.22
090472 2000 0.21
090472 2200 0.20
090472 2400 0.19
999999
```

```
1
250274 1700 0.38
250274 1900 0.36
250274 2130 0.40
250274 2230 0.44
250274 2400 0.46
260274 0000 0.46
260274 0400 0.49
260274 0430 0.56
260274 0500 0.64
260274 0515 0.73
260274 0530 0.83
260274 0600 0.88
260274 0700 0.84
260274 0730 0.82
260274 0830 0.93
260274 0900 1.02
260274 0915 1.10
260274 0930 1.16
260274 0945 1.23
260274 1000 1.29
260274 1015 1.36
260274 1025 1.46
260274 1030 1.44
260274 1035 1.42
260274 1100 1.50
260274 1115 1.60
260274 1130 1.67
260274 1200 1.70
260274 1230 1.72
260274 1300 1.79
260274 1400 1.82
260274 1500 1.77
260274 1530 1.69
260274 1600 1.60
260274 1610 1.51
260274 1630 1.42
260274 1700 1.45
260274 1710 1.50
260274 1800 1.41
260274 1830 1.34
260274 1900 1.26
260274 2000 1.17
260274 2100 1.11
260274 2300 1.16
260274 2400 1.22
270274 0000 1.22
270274 0100 1.21
270274 0300 1.15
270274 0700 1.10
270274 0900 1.03
270274 1000 0.97
270274 1100 0.91
270274 1200 0.83
270274 1400 0.73
270274 1600 0.65
270274 1800 0.59
270274 2000 0.49
270274 2400 0.44
999999
```

Table B.4 Output from program SWAHYD

STATION CODE: 3112M01 | STATION: OTJIVERO | RIVER: WHITE NOSSOB

DATE	CLOCK TIME	WATER LEVEL(METRES)	DISCHARGE(CUMECS)
25/ 3/71	2:30	0.09	0.20
	2:45	0.19	2.06
	3: 0	0.21	2.82
	4: 0	0.22	3.26
	6: 0	0.24	4.29
	7: 0	0.28	6.94
	8: 0	0.27	6.19
	9: 0	0.25	4.87
	10: 0	0.23	3.75
	12: 0	0.21	2.82
	15: 0	0.20	2.42
	21: 0	0.18	1.74
	23: 0	0.38	9.71
	23:15	0.59	19.34
	23:30	0.66	23.12
	23:45	0.73	27.16
	24: 0	0.78	30.20
26/ 3/71	0: 0	0.78	30.20
	0:15	0.82	32.73
	0:30	0.84	34.02
	0:45	0.87	36.00
	1: 0	0.93	40.09
	1:15	0.97	42.91
	1:30	1.03	47.29
	1:45	0.99	44.35
	2: 0	0.98	43.63
	2:15	0.97	42.91
	2:30	0.95	41.49
	2:45	0.97	42.91
	3: 0	0.86	35.34
	3:15	0.78	30.20
	3:30	0.73	27.16
	3:45	0.70	25.40
	4: 0	0.68	24.25
	4:15	0.65	22.56
	4:30	0.61	20.39
	4:45	0.59	19.34
	5: 0	0.56	17.81
	5:30	0.52	15.84
	6: 0	0.49	14.43
	7: 0	0.45	12.63
	8: 0	0.43	11.76
	9: 0	0.39	10.11
	11: 0	0.37	9.32
	12: 0	0.35	8.55
	14: 0	0.32	7.46
	17: 0	0.29	7.74
	19: 0	0.27	6.19
	20: 0	0.26	5.50
	21: 0	0.25	4.87
	22: 0	0.23	3.75
	24: 0	0.21	2.82
27/ 3/71	0: 0	0.21	2.82
	2: 0	0.19	2.06
	4: 0	0.18	1.74
	5: 0	0.17	1.46
	7: 0	0.16	1.21
	8: 0	0.15	0.99
	11: 0	0.13	0.63
	13: 0	0.12	0.49
	15: 0	0.11	0.37
	18: 0	0.09	0.20
	24: 0	0.07	0.09
28/ 3/71	0: 0	0.07	0.09
	6: 0	0.05	0.03
	12: 0	0.03	0.01
	18: 0	0.01	0.00

FLOOD VOLUME = 1.60 * 10**6 CUBIC METRES

STATION CODE: 3112M01
****

STATION: OTJIVERO
****

RIVER: WHITE NOSSOB
******

DATE	CLOCK TIME	WATER LEVEL(METRES)	DISCHARGE(CUMECS)
7/ 4/72	4: 0	0.04	0.02
	4:10	0.22	3.26
	4:20	0.55	17.31
	4:30	0.69	24.82
	4:40	0.76	28.97
	4:50	0.90	34.02
	5: 0	1.15	56.55
	5:10	1.45	83.82
	5:20	1.67	103.14
	5:30	1.79	114.76
	5:40	1.83	118.81
	5:50	1.90	126.11
	6: 0	2.04	141.55
	6:10	2.30	173.26
	6:20	2.43	190.66
	6:30	2.57	210.58
	6:40	2.62	218.00
	6:50	2.66	224.05
	7: 0	2.68	227.12
	7:45	2.74	236.47
	8: 0	2.78	242.84
	8:15	2.73	234.90
	8:30	2.66	224.05
	8:45	2.57	210.58
	9: 0	2.41	187.91
	9:30	2.21	161.82
	10: 0	2.00	137.02
	10:30	1.81	116.78
	11: 0	1.66	102.21
	11:30	1.53	90.56
	12: 0	1.43	82.19
	12:30	1.33	71.65
	13: 0	1.23	63.08
	13:30	1.15	56.55
	14: 0	1.07	50.30
	14:30	0.99	44.35
	15: 0	0.91	38.71
	16: 0	0.82	32.73
	17: 0	0.78	30.20
	18: 0	0.77	29.58
	19: 0	0.69	24.82
	20: 0	0.64	22.01
	21: 0	0.63	21.47
	22: 0	0.65	22.56
	23: 0	0.67	23.68
	24: 0	0.65	22.56
8/ 4/72	0: 0	0.65	22.56
	1: 0	0.60	19.87
	2: 0	0.56	17.81
	3: 0	0.53	16.32
	4: 0	0.49	14.43
	5: 0	0.48	13.97
	6: 0	0.47	13.51
	7: 0	0.44	12.19
	8: 0	0.40	10.51
	9: 0	0.42	11.34
	10: 0	0.44	12.19
	11: 0	0.43	11.76
	12: 0	0.44	12.19
	13: 0	0.43	11.76
	14: 0	0.41	10.92
	15: 0	0.40	10.51
	16: 0	0.37	9.32
	17: 0	0.35	8.55
	18: 0	0.34	8.18
	19: 0	0.32	7.46
	20: 0	0.31	7.11
	23: 0	0.30	8.61
	24: 0	0.28	6.94
9/ 4/72	0: 0	0.28	6.94
	2: 0	0.27	6.19
	4: 0	0.26	5.50
	6: 0	0.25	4.87
	14: 0	0.24	4.29
	16: 0	0.23	3.75
	18: 0	0.22	3.26
	20: 0	0.21	2.82
	22: 0	0.20	2.42
	24: 0	0.19	2.06

FLOOD VOLUME = 7.17 * 10**6 CUBIC METRES
****

STATION CODE: 3112M01
****

STATION: OTJIVERO
****

RIVER: WHITE NOSSOB
******

DATE	CLOCK TIME	WATER LEVEL(METRES)	DISCHARGE(CUMECS)
25/ 2/74	17: 0	0.38	9.71
	19: 0	0.36	8.93
	21:30	0.40	10.51
	22:30	0.44	12.19
	24: 0	0.46	13.07
26/ 2/74	0: 0	0.46	13.07
	4: 0	0.49	14.43
	4:30	0.56	17.81
	5: 0	0.64	22.01
	5:15	0.73	27.16
	5:30	0.83	33.37
	6: 0	0.88	36.67
	7: 0	0.84	34.02
	7:30	0.82	32.73
	8:30	0.93	40.09
	9: 0	1.02	46.55
	9:15	1.10	52.61
	9:30	1.16	57.35
	9:45	1.23	63.08
	10: 0	1.29	68.17
	10:15	1.36	74.30
	10:25	1.46	84.65
	10:30	1.44	83.00
	10:35	1.42	81.38
	11: 0	1.50	87.99
	11:15	1.60	96.72
	11:30	1.67	103.14
	12: 0	1.70	105.97
	12:30	1.72	107.89
	13: 0	1.79	114.76
	14: 0	1.82	117.79
	15: 0	1.77	112.77
	15:30	1.69	105.03
	15: 0	1.60	96.72
	16:10	1.51	88.84
	16:30	1.42	81.38
	17: 0	1.45	83.82
	17:10	1.50	87.99
	18: 0	1.41	80.58
	18:30	1.34	72.53
	19: 0	1.26	65.61
	20: 0	1.17	58.15
	21: 0	1.11	53.39
	23: 0	1.16	57.35
	24: 0	1.22	62.25
27/ 2/74	0: 0	1.22	62.25
	1: 0	1.21	61.42
	3: 0	1.15	56.55
	7: 0	1.10	52.61
	9: 0	1.03	47.29
	10: 0	0.97	42.91
	11: 0	0.91	38.71
	12: 0	0.83	33.37
	14: 0	0.73	27.16
	16: 0	0.65	22.56
	18: 0	0.59	19.34
	20: 0	0.49	14.43
	24: 0	0.44	12.19

FLOOD VOLUME =   8.64 * 10**6 CUBIC METRES
****

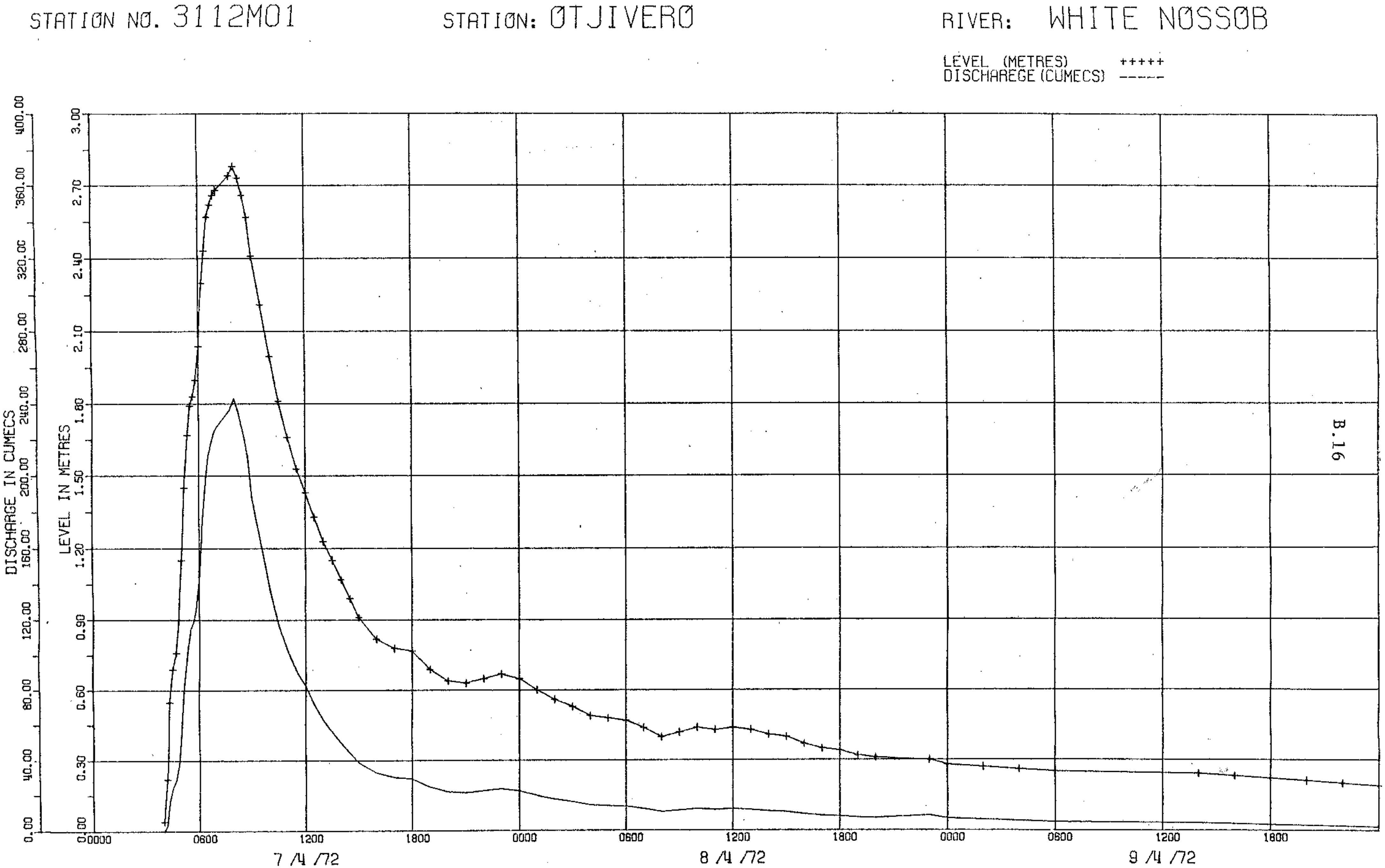

Figure B. 1 CALCOMP plot of stage and discharge hydrographs