INSTITUTE of
HYDROLOGY

GABORONE DAM RAISING

HYDROLOGICAL ANALYSIS

This report is prepared for

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INTRODUCTION

Gaborone reservoir is situated on the Notwani river draining a catchment area of approximately 4300 km^2. Figure 1 indicates the outline of the catchment area and raingauges in the locality. The objectives of the current hydrological study are

(i) to estimate the floods which are likely to pass through Gaborone reservoir with return periods of 20 to 500 years and

(ii) to investigate the 10, 20 and 50 year return period yields expected from the reservoir considering several different dam heights.

1.1 AVAILABLE RECORDS

Rainfall

The available monthly and daily rainfall records are summarized in Table 1 together with the mean annual rainfall calculated from 1922 to 1979. The catchment mean annual rainfall was calculated from a weighted mean of the point rainfall data where the weights were based on the location of the rainfall station with respect to the catchment. Thus the catchment mean annual rainfall was estimated as 541 mm.

Runoff

Suitable runoff data have not been collected for inflows to Gaborone reservoir but daily reservoir levels have been recorded together with six hourly readings for the short period from 27 February to 31 March 1976. In a previous report (Ref. 1) the inflows to Gaborone reservoir were derived from the rises in reservoir level immediately after rainfall plus any spillage. Generally the reservoir water levels decrease in a regular fashion due to evaporation and demand so that an inflow event can easily be isolated and quantitatively assessed. The equation used to estimate spill over the crest was

$$Q = 1.656 \times 270.8 \, (h - \text{spillway level})^{1.5} \qquad (1)$$

where Q = Spill in cumecs

h = gauged water level in m.

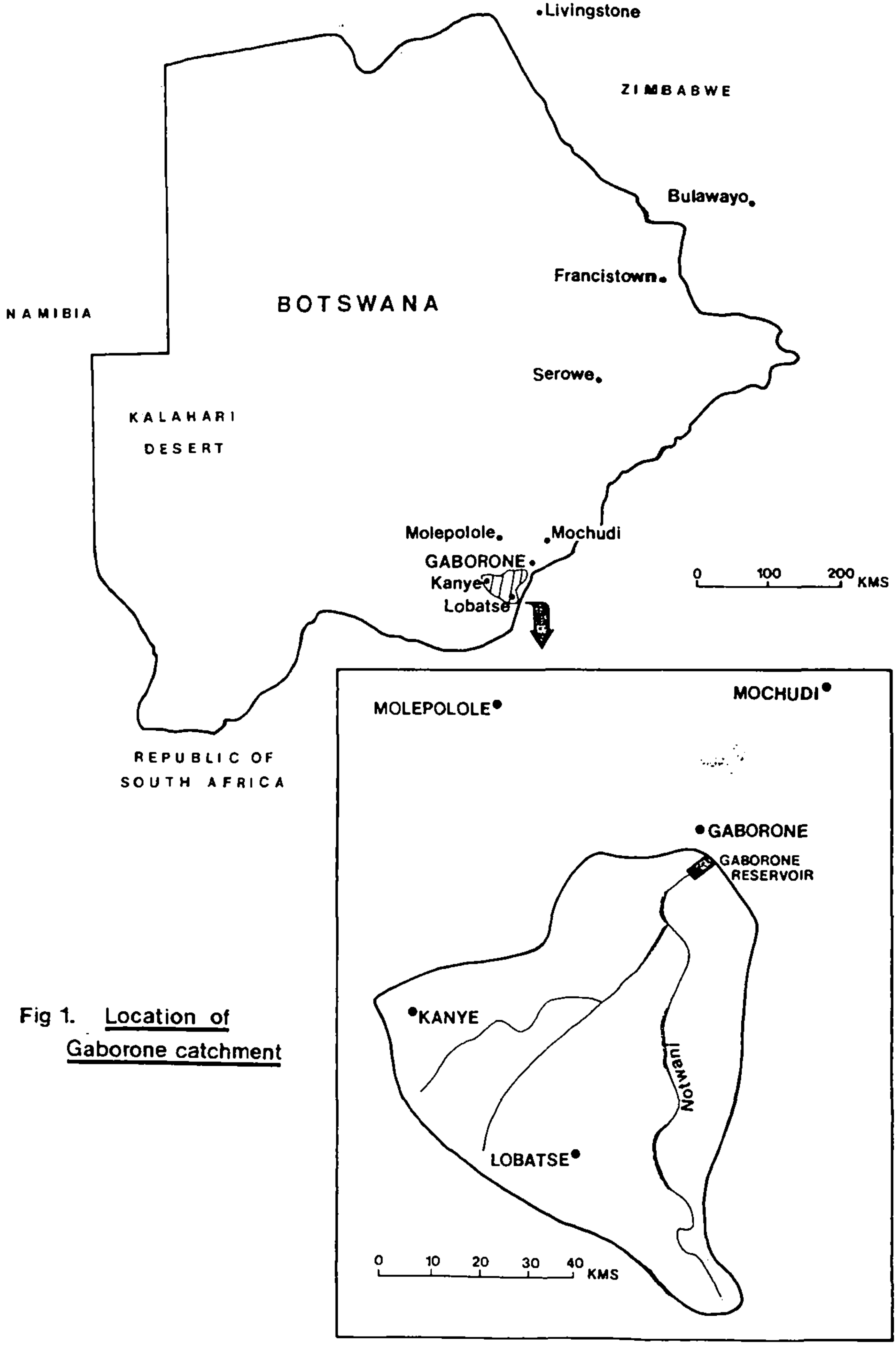


Fig 1. Location of Gaborone catchment

TABLE I - GABORONE DAM CATCHMENT RAINFALL RECORDS

	Start Date	Finish Date	No. of years of daily data	MAR (mm)	Weight
Gaborone *	1922	1979	41	536.9	
Lobatse	1922	1979	60	578.0	
Kanye	1922	1979	56	525.1	
Molepolole	1922	1979	57	502.9	
Mochudi *	1909	1979	66	500.3	
TOTAL			280		

* Daily rainfall data are not available for all the years.

TABLE 2: GABORONE DAM DISCHARGE MEASUREMENTS

(spillway at 15.019 m)

Date	Water level on gauge (m)	Mean crest velocity (m/s)	Spillway discharge (m^3/s)	Method
10.3.77	15.062	0.52	3.95	Velocity head
12.3.77	15.245	1.17	46.9	Velocity head
		0.86	42.0	Velocity head
		1.02	51.1	Current meter
13.3.77	15.200	0.93	38.6	Current meter
15.4.77	15.105	0.57	10.8	Velocity head

The available data included 4 spillway discharge measurements in March and April 1977 (from current metering) tabulated in Table 2.

These data were subjected to a regression analysis which calculated the best fit equation for the data as

$$Q = 1.602 \times 270.8 \, (h - 15.021)^{1.47} \qquad (2)$$

Values of Q, calculated for the maximum range of h expected, varied by 12% or less using equations (1) and (2); thus the regression confirmed the earlier equation.

The inflows to the reservoir were updated to include 1978/79 values (Table 3) and from this series the mean annual runoff is estimated as 34 million m^3 (compared with 35 million m^3 Ref (1)).

Evaporation

Open water evaporation estimates have been calculated for the period 1956 to 1968 (Ref. (2)) and the accuracy of these estimates was tested using a simple water balance carried out for an "average year".

The slope of the recession curve of the reservoir water level data indicates the rate of losses from the reservoir which includes demand, evaporation and seepage. The average monthly losses were calculated from the reservoir levels, and evaporation and demand estimates were deducted from these to determine the extent of seepage. The seepage values, thus calculated, were negligible compared to the evaporation indicating that either the seepage is negligible or that the evaporation is overestimated and accounts for seepage (as both terms are water level related). In either case the monthly evaporation estimates, together with monthly demand figures, are adequate to indicate the losses from the reservoir.

TABLE 3

INFLOWS TO GABORONE RESERVOIR (million m^3)

	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	TOTAL
1959-60			0,9	0,3		0,62	2,62						4,4
1960-61		0,93	18,25		5,7	1,32	29,8						56,00
1961-62							6,25	1,1					7,35
1962-63		3,23	3,96										7,19
1963-64	2,0	2,0											4,0
1964-65	3,7	9,9											13,6
1965-66			0,738	10,209	73,257			1,722				1,722	87,648
1966-67	0,369	2,030	9,325	41,792	35,160	11,487	71,790	4,620	0,160		0,492	0,246	177,471
1967-68		1,599		1,599		6,396	1,230	0,123	1,599			1,353	13,899
1968-69	0,984	0,615	0,492	0,123	0,369		0,123	-					2,706
1969-70		0,062	0,061			0,123							0,246
1970-71		0,615	1,230	1,722	4,920	0,738	0,123						9,348
1971-72		11,931	0,738	31,029									43,698
1972-73					0,861		10,578						11,439
1973-74	5,781	3,444	10,726	14,530	19,610	13,330	1,240	0,240					68,901
1974-75		1,107	0,369	0,369		4,305	7,153	2,900					16,203
1975-76	0,416		2,165	3,747	9,619	31,103	4,566	0,564					52,180
1976-77	2,082	0,108	0,083	0,432	2,482	19,912	16,811						41,910
1977-78			0,45	31,23	22,92	5,97	-						60,570
1978-79			0,20			-							0,200
											AVERAGE		33,948

Inflows for 1959/60 to 1964/65 are taken from Lund, Stage II Report, and are flows at Notwane dam.

FLOOD ANALYSIS

2.1 INTRODUCTION

This section of the report provides estimates of the 20, 50, 100, 200 and 500 year unrouted flood hydrographs for the catchment upstream of Gaborone Dam (Figure 1).

Flood estimates of these return periods may, in general, be obtained by a number of methods. Of these the simplest is that which uses annual peak flows abstracted from continuous flow records from a river gauging station at the required location. These annual maximum flows are ranked and plotted using an assumed theoretical frequency distribution. To be able to use this method without excessive extrapolation many years of streamflows are required at the single station or a number of stations in an area. This, however, is commonly not the case and other methods must be employed.

On a worldwide basis rainfall stations are more plentiful and their records longer than for river gauging stations. From local rainfall records it is normally possible to derive rainfall intensity/duration/frequency relationships and use the statistical properties of the rainfall to estimate floods of the required return period. For this to be possible a method of converting rainfall to river flow is required. Catchment unit hydrographs, which define the response to a unit net input of rainfall, have gained acceptance by most hydrologists as a useful tool in flood estimation. Theoretically it requires only one flood to be recorded at a gauging station together with a continuous (autographic) trace of storm rainfall to enable the derivation of a useful unit hydrograph. However it is preferable to take a number of events and obtain an average unit hydrograph. In the absence of the necessary continuous rainfall and flow data, synthetic unit hydrographs may be constructed using catchment properties (eg stream length, channel slope).

An important aspect of the derivation of flood flows from rainfall is the choice of percentage of rainfall effective in contributing to flood flows. If recorded flood and rainfall events are available then loss rates may be computed from these data and used in the design storm. Alternatively these data may be used to assess the percentage runoff for each storm. In general US practice has been to use the concept of a loss rate, which may be defined as an initial and a continuing loss rate. The Flood Studies Report (FSR) (Ref 3) found it reasonable, after tests of the alternatives, to use a runoff coefficient as a basis for design; this approach allows the runoff coefficient to be based on typical events and to increase with total storm rainfall, but is less conservative in design than the concept of a fixed soil infiltration. The choice of runoff coefficient in Britain may depend on the relatively low rainfall intensities and high infiltration rates prevailing, but this approach was thought more realistic for use in Botswana than the estimation of loss rates obtained from a moderate storm which would result in a very high runoff percentage in the design case.

In the absence of long term flow records, it was considered that the most satisfactory method of deriving flood estimates on this catchment was by the combined use of a unit hydrograph to determine the nature of catchment response and rainfall intensity/duration/frequency relationships to produce rainstorms of the desired severity. Although this study has made extensive use of the methods of analysis described in the FSR, whenever possible local data have been used to modify relationships from the United Kingdom.

The recommended design peak flows are summarized in Table 7 of this report.

2.2 DATA USED IN FLOOD ANALYSIS

Daily rainfall totals (measured at 8 a.m. and credited to the previous day) were available from 5 gauges in the Gaborone area (Figure 1). Using the catchment weightings given in Table 1 the Gaborone catchment annual average rainfall was computed as 541 mm. These rainfall data were used both to construct a local annual maximum daily rainfall series and as individual daily totals in unit hydrograph derivation.

Hydrological Research Unit Report (HRU) No 1/69 (Ref. 4) was used to extend the rainfall analysis to periods other than one day. Areal reduction factors were also taken from the same report.

Daily (8 a.m.) Gaborone reservoir levels were used in the unit hydrograph analysis. During the 1976 flood season these data were available at 6 hour time intervals.

2.3 RAINFALL ANALYSIS

Annual maximum daily rainfall series

In the process of abstracting annual maximum daily rainfalls for each of the five raingauges it was evident that annual maxima for each gauge did not necessarily fall on the same day. This was to be expected since the rainfall in this region is typified by local convective storms. The five raingauges were therefore considered to be independently sampling the same population, and their records added sequentially to provide an effective 280 year data set. This extended record was ranked and plotted using Gringorten plotting positions with a Gumbel reduced variate (Figure 2):

Probability, P $\frac{I - 0.44}{N + 0.12}$ (Gringorten formula)

where I = rank position
N = total number of points

Gumbel reduced variate, $Y = -\log_n(-\log_n P)$

Return period, $T = \frac{1}{1 - P}$

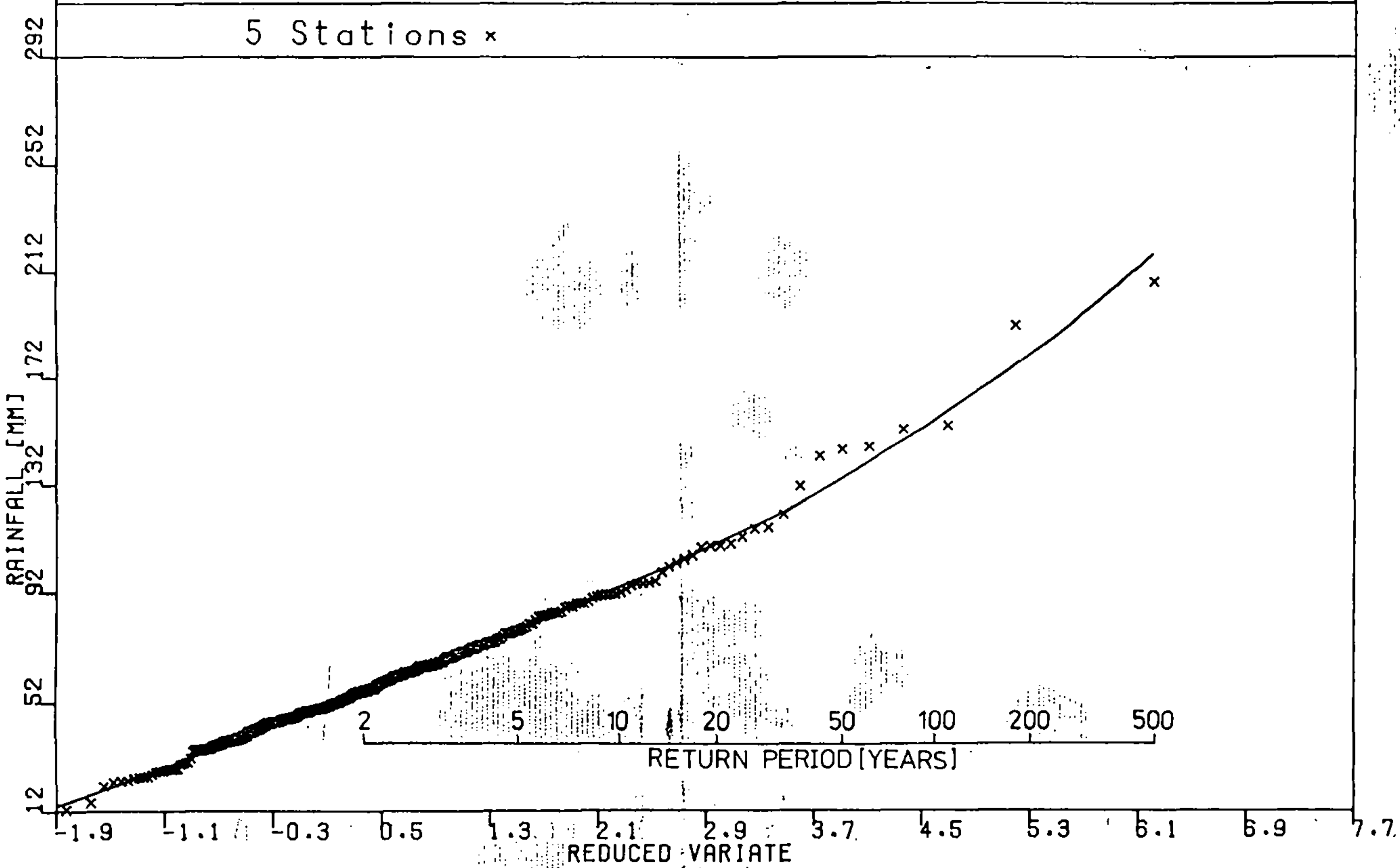
Using this lengthened data set it was possible to estimate maximum daily rainfalls of higher return periods with greater accuracy than with the gauges treated individually.

From Figure 2 it can be seen that the relationship is linear up to a 20 year return period and of increasing slope thereafter. A best fit curve was drawn by eye through these points and maximum rainfalls for 20, 50, 100, 200 and 500 year return periods abstracted (Table 4). Using 20 years as a basic return period, rainfall growth factors for the other return periods were computed (Table 4).

TABLE 4: MAXIMUM DAILY RAINFALLS

Return period years	Max Daily rainfall mm	Growth Factor
20	108	1.00
50	136	1.26
100	160	1.48
200	184	1.70
500	219	2.03

Fig 2 Maximum Annual Daily Rainfalls



Rainfalls so far considered relate to point measurements. In computing catchment rainfalls it is necessary to apply areal reduction factors to account for the fact that point intensities are higher than those occurring, with the same probability of exceedance, over larger areas. HRU report No 1/69 Figure 5.13 gives areal reduction factors for South Africa and these have been used in this study.

Figure 3 of the current report shows this information for a catchment area of 4300 km^2 (ie Gaborone catchment). The 1 day, 20 year return period catchment rainfall was then computed:-

1 day 20 year return period point rainfall	= 108 mm
1 day areal reduction factor	= 0.625
1 day 20 year return period catchment rainfall	= 108 x 0.625 = 67.5 mm

Rainfall intensity/duration analysis

In the synthesis of the design rainstorm, rainfall intensities of storms with durations other than one day are required. HRU report No 1/69 (Ref 4) provides two analyses for South Africa. Firstly, using daily rainfall totals, storm durations of one day and more were studied on a regional basis. The Gaborone catchment is closest to and partly in region 10 (annual rainfall subdivision 500 - 1000 mm). Secondly, storm durations of less than one day were studied using records from autographic gauges on a country wide basis. Point rainfall depths are related to mean annual rainfall, duration (15 minutes to 24 hours), recurrence interval and rainfall season.

From these two studies information relating to the Gaborone catchment has been abstracted and is shown in Table 5. The long duration analysis gives the 1 day 20 year return period rainfall as 67.5 mm which is in agreement with the figure calculated locally (above). From the short duration analysis a 24 hour rainfall total of 68.8 mm is obtained. It is normally accepted that rainfall totals occurring within any 24 hour period are higher than those totals falling in a fixed calendar day. Since no conversion of daily to 24 hour totals is given in HRU Report No 1/69 (Ref 4), figures from the long duration analysis have been adjusted by the ratio $\frac{68.8}{67.5}$ to give agreement at the 24 hour time interval.

Figure 4 shows the 20 year return period rainfall intensity/duration graph for the Gaborone catchment. The discontinuity at 24 hours is due

Fig. 3. Areal reduction factor/duration relationships for Gaborone catchment

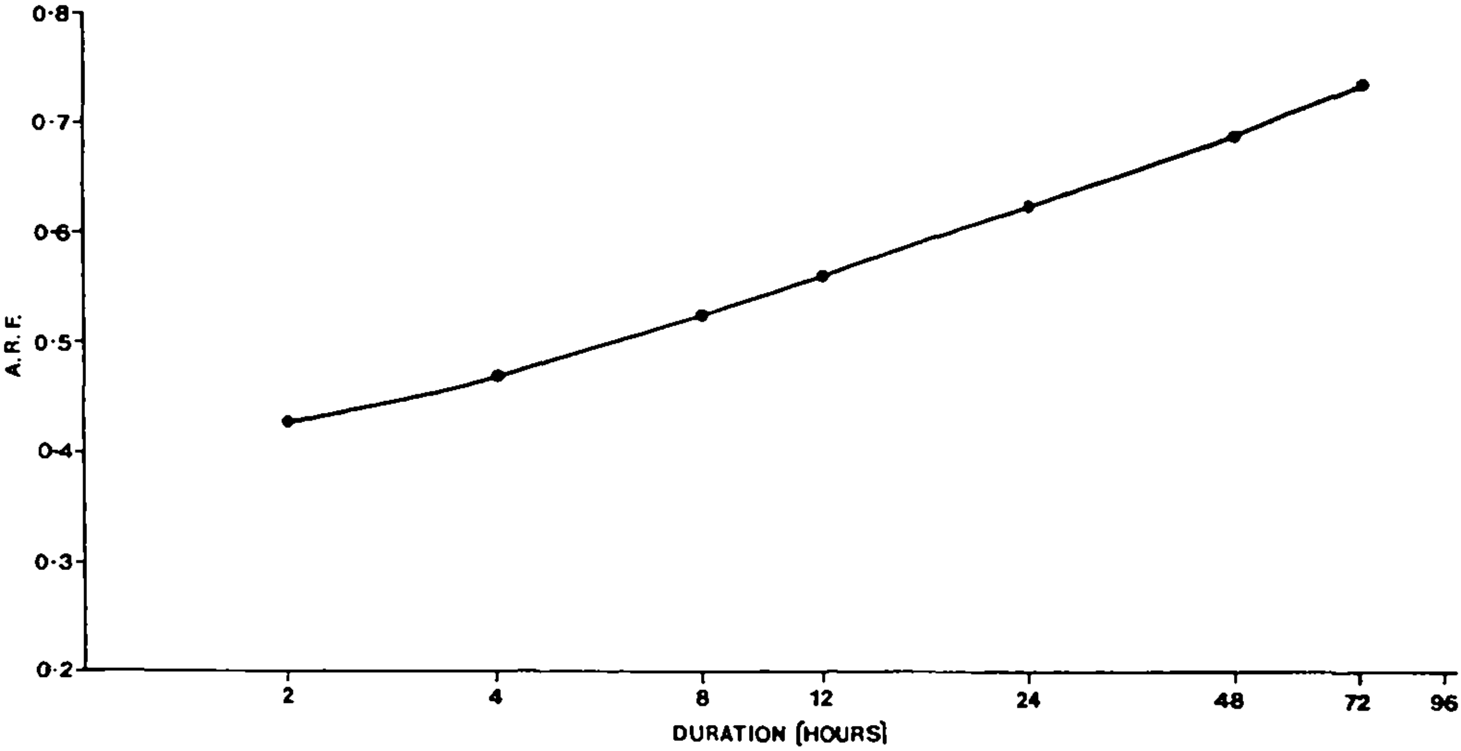


Fig. 4. Rainfall intensity/duration relationships for Gaborone catchment

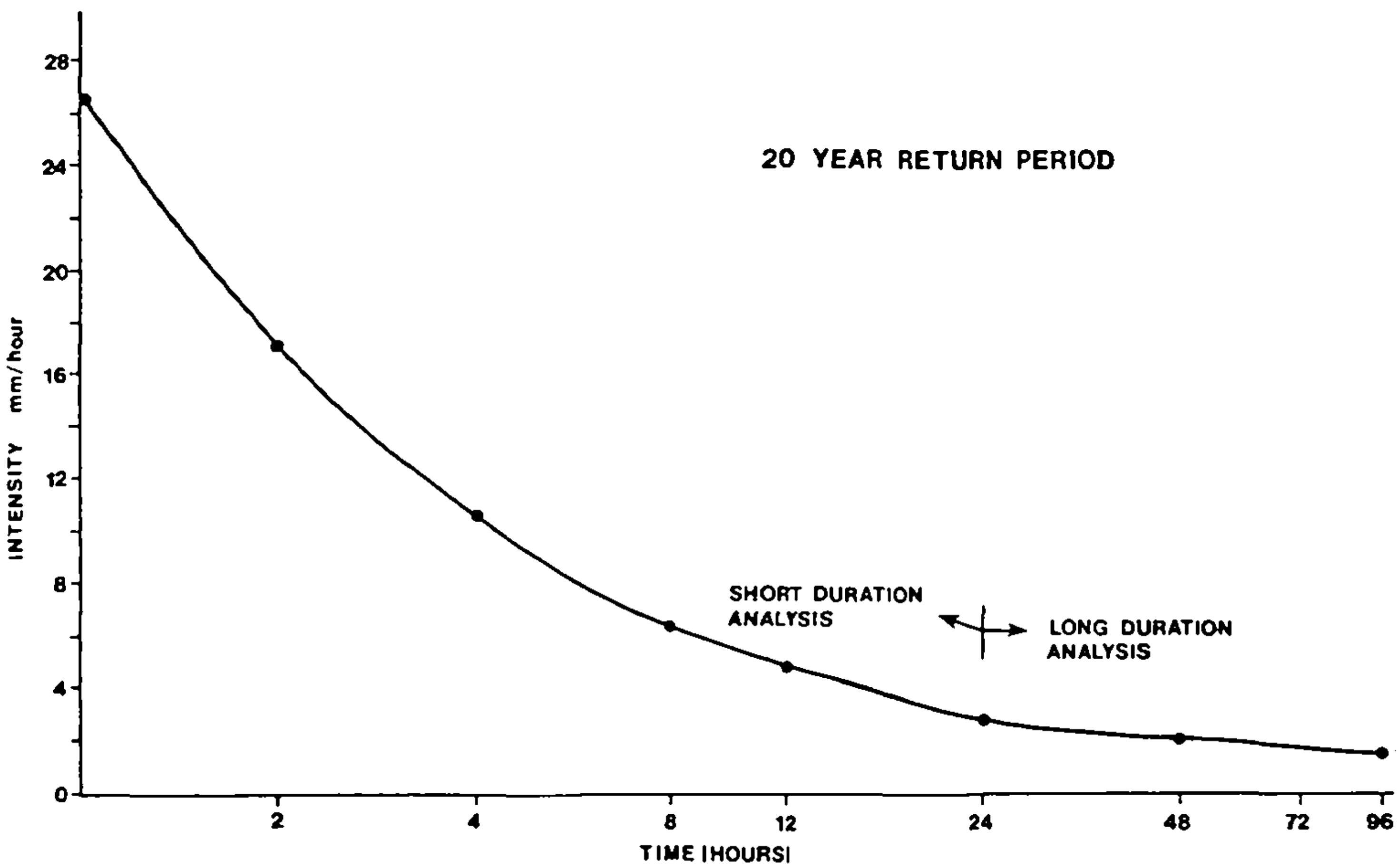


TABLE 5: 20 YEAR RETURN PERIOD RAINFALLS FOR GABORONE CATCHMENT

Analysis type	Duration hours	Depth (mm)	Areal reduction factor	Areal depth (mm)	Adjusted depth (mm)	Intensity (mm/hour)
S	2	81	.43	34.8		17.4
S	4	91	.47	42.8		10.7
S	8	100	.525	52.5		6.58
S	12	107	.56	59.9		5.0
S	24	110	.625	68.8		2.87
L	48			110.0	112.1	2.34
L	72			133.0	135.6	1.89
L	96			150.0	152.9	1.59
L	120			159.0	162.1	1.36

S = short duration analysis

L = long duration analysis

TABLE 6: FLOOD EVENTS USED IN THE UNIT HYDROGRAPH ANALYSIS

Event number	Peak discharge (m^3/s)	Initial CWI (mm)	Average Base flow (m^3/s)	Percentage runoff
	125	140.3	3.06	3.4
2	102	128.7	2.3	8.5
4	72	125.0	2.4	9.8
6	66	131.8	6.0	3.9
7	32	127.0	0.73	4.3
Average		130.6	2.9	

TABLE 7: FLOOD ESTIMATE SUMMARY

Return period (years)	Peak flow (m^3/s)	Flood volume (million m^3)
20	528	151
50	765	218
100	997	283
200	1259	357
500	1707	483

to the two separate methods of analysis used in HRU Report No. 1/69 (Ref 4). Although this fact is noted by the authors of the report, it means that the rainstorms constructed for the flood analyses reflect this break in slope. However it does not have a significant effect on the size of the design flood estimates.

2.4 UNIT HYDROGRAPH DERIVATION

Gaborone dam inflows

The unit hydrograph for a catchment is most reliably obtained from an analysis of recorded flood and rainfall data from the catchment itself. This process also yields, for each flood event studied, the percentage runoff, ie the proportion of rain effective in producing the flood hydrograph. It is recommended by the FSR that at least five large events should be used in the analysis. Although there are no records for inflows into Gaborone reservoir, daily readings of reservoir level at the spillway were available. During the 1976 flood season reservoir stage measurements were taken at 6 hr intervals. For a catchment of this size a time resolution of 24 hours for both rainfall and flow data would normally be regarded as too large for an accurate unit hydrograph derivation. However, it was considered preferable to use these natural data with their limitations rather than to revert to a synthetic unit hydrograph derived from catchment characteristics (ie length of main river, bed slope).

Before this unit hydrograph analysis could proceed it was necessary to obtain the inflow hydrograph of several large flood events responsible for the changes of stage recorded at the dam. From plots of these stage data, seven floods were chosen from the three year period 1976 to 1978. The two floods in 1976 had the advantage of stage readings at a 6 hour time interval:-

Flood number	Start date	Flow data interval (hrs)
1	26/2/76	6
2	18/3/76	6
3	9/3/77	24
4	31/3/77	24
5	22/1/78	24
6	19/2/78	24
7	9/3/78	24

Inflow discharges were derived from reservoir level records by the inverse of a routing procedure. The following information was used in the computations:

(1) The reservoir level/storage characteristic curve

(2) Average wet season evaporation and demand. During floods these have a relatively minor effect but were included for completeness

(3) The regression based spillway rating equation

This inverse routing procedure proved to be an unstable process. Errors in estimation of inflow in one time step arising from small errors in the record of reservoir levels, often resulted in a compensating correction in the subsequent step. However since this instability was of an oscillatory nature it was possible to remove it almost completely by a two point moving average. A final smoothing by hand of the flood hydrographs was necessary because of some residual instability and also because of the coarseness of the data interval. From these smoothed hydrographs, flow values were abstracted at 6 hour time intervals.

Analysis of rainfall and runoff data

Rainfall having an influence on flood events within the Gaborone catchment was taken to be represented by the mean of the Kanye, Gaborone and Lobatse gauges up to the day preceding the start of rise of the flood hydrograph. Antecedent precipitation for the five days preceding each event (P_{d-1}, P_{d-2} etc) was taken from the mean of the same three gauges and used to calculate the antecedent precipitation index (API5) thus:

$$API5_d = 0.5(P_{d-1} + 0.5\ P_{d-2} + (0.5)^2 P_{d-3} + (0.5)^3 P_{d-4} + (0.5)^4 P_{d-5})$$

Runoff was separated according to the method recommended in the FSR; the recession before the hydrograph rising limb was extended to below the peak and from there joined to the recession at a distance 4 x LAG after the peak. LAG is defined as the time from the centroid of total rainfall to peak flow or weighted peak flow for a multi-peaked event. Percentage runoff is that percentage of the storm rainfall required to produce the total separated or quick response runoff. The average non separated flow during the event is a measure of baseflow during that event. In the absence of any soil moisture deficit (SMD) data, SMD has had to be ignored throughout this study, both in the unit hydrograph derivation process and synthesis of the design flood hydrograph.

The antecedent state of the catchment for each flood event is indexed using the concept of a Catchment Wetness Index (CWI) from the FSR. This is computed as:

$$CWI = API5 - SMD + 125 \text{ (mm)}$$

The constant 125 is added for convenience to keep the index positive. For this analysis the index reduced to

$$CWI = API5 + 125$$

The net rainfall profile for each event has been deduced using the concept of a loss rate curve as defined in the FSR. The loss rate curve is an extension of the infiltration curve originally due to Horton (Ref 5) and is assumed to include the effect of all forms of loss in addition to infiltration. The FSR links the loss rate to the inverse of CWI in such a way as to ensure the volume of effective rain equals the response runoff. Thus as the storm progresses CWI increases and loss rate decreases.

The analysis was applied to the data from each of the seven events and is illustrated by Figures 5 to 11.

Unit hydrograph derivation

Having separated effective flood producing rainfall and flood flows, unit hydrographs were derived from each flood event using matrix inversion with smoothing. Of the seven flood events five produced useful unit hydrographs and these are shown with the peaks aligned on Figure 12. Event number 3 was rejected because of timing problems with the rainfall data (Figure 7) and event number 5 (Figure 9) was rejected because its double peak produced a double peaked unit hydrograph. Figure 12 shows that one unit hydrograph (from event 1) has a considerably higher peak than the rest. Differences in unit hydrograph shape can be attributed to dissimilar spatial variation of rainfall from event to event and, in this study, partly to the coarseness of the data interval. This is the reason why it is necessary to analyse a group of floods and obtain the mean or median unit hydrograph.

Fig.5 Inflow hydrograph and rainfall

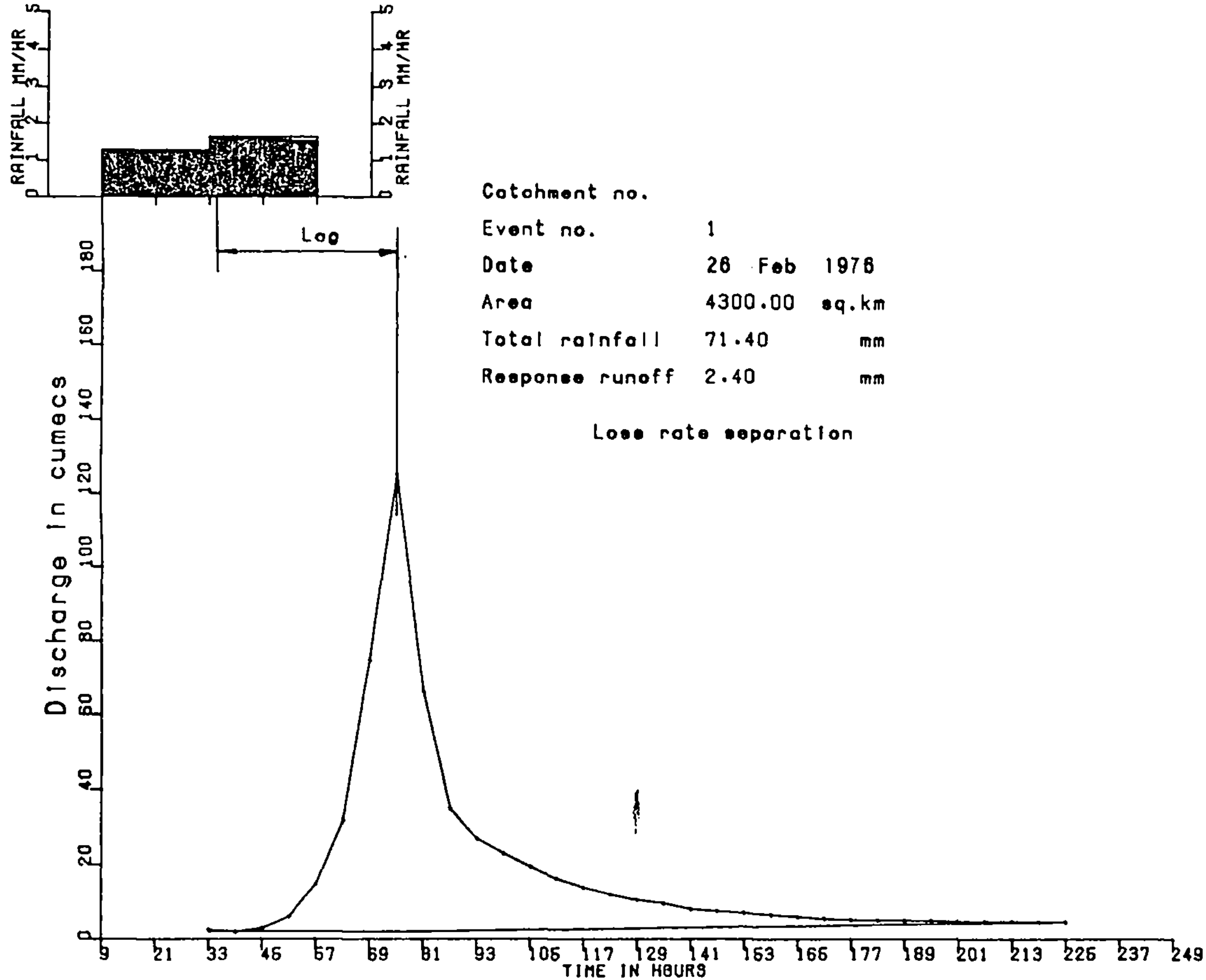


Fig.6 Inflow hydrograph and rainfall

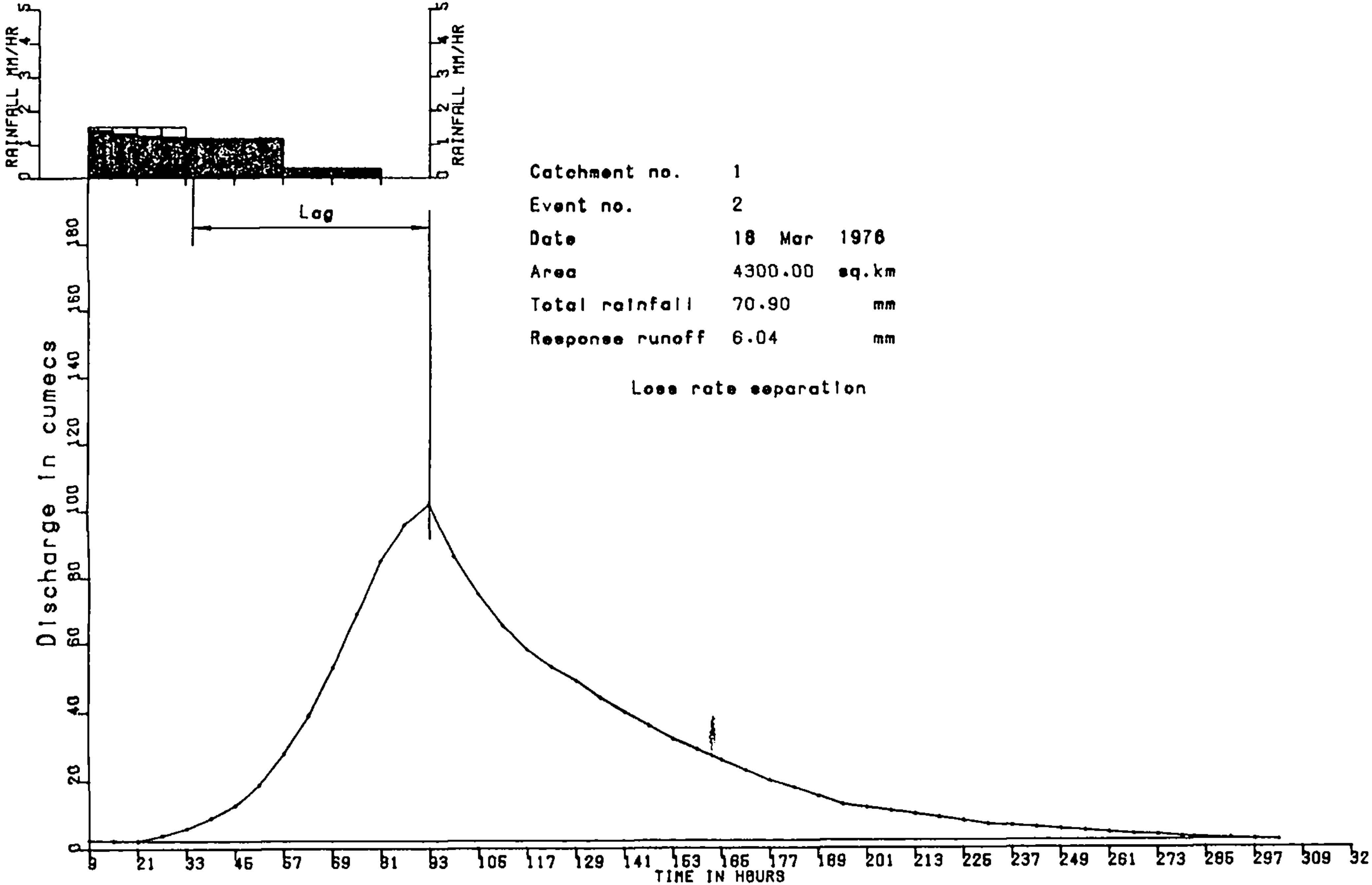
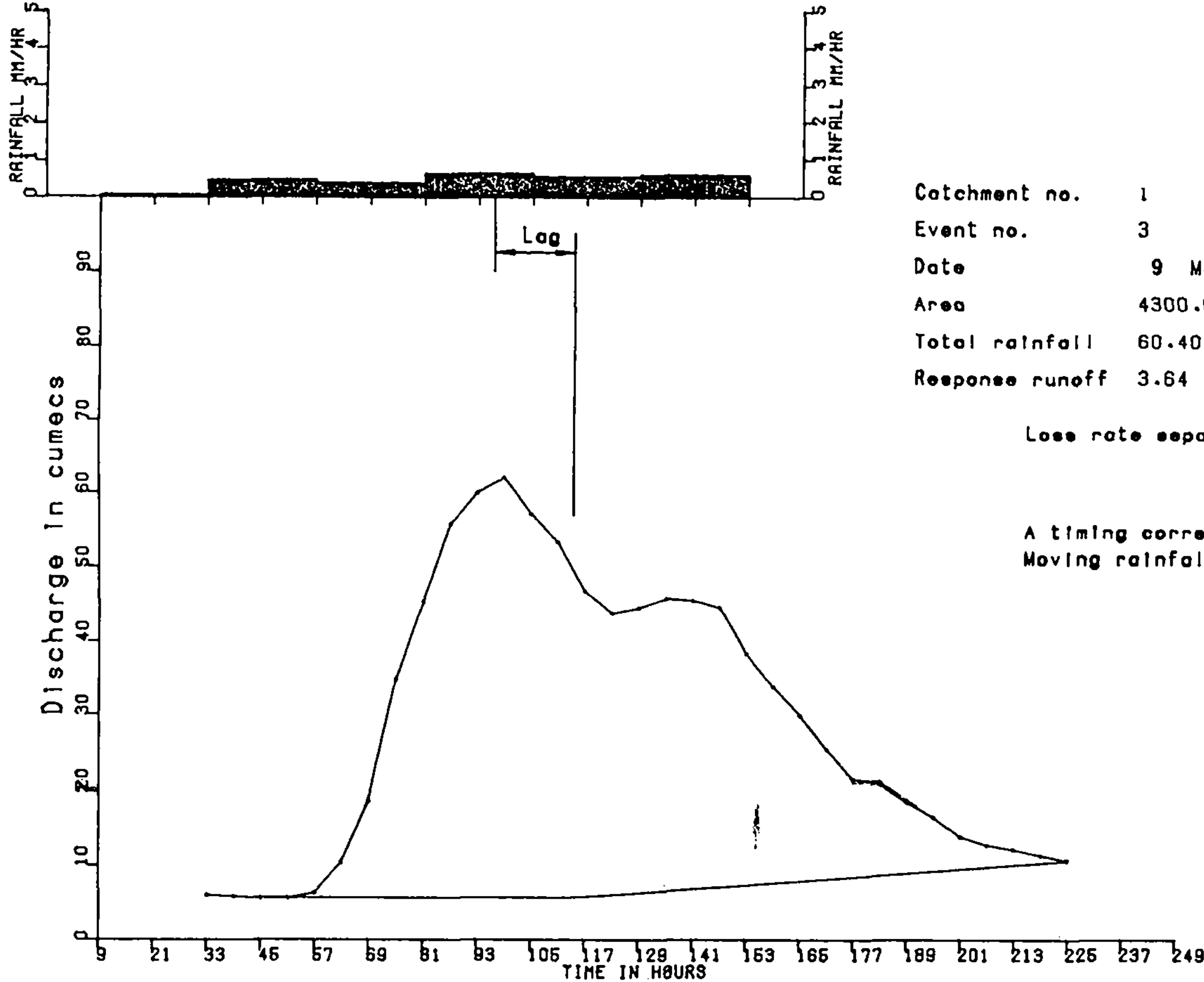


Fig.7 Inflow hydrograph and rainfall



Catchment no.	1
Event no.	3
Date	9 Mar 1977
Area	4300.00 sq.km
Total rainfall	60.40 mm
Response runoff	3.64 mm

Loss rate separation

A timing correction has been used
Moving rainfall left by 24.0hours

Fig.8 Inflow hydrograph and rainfall

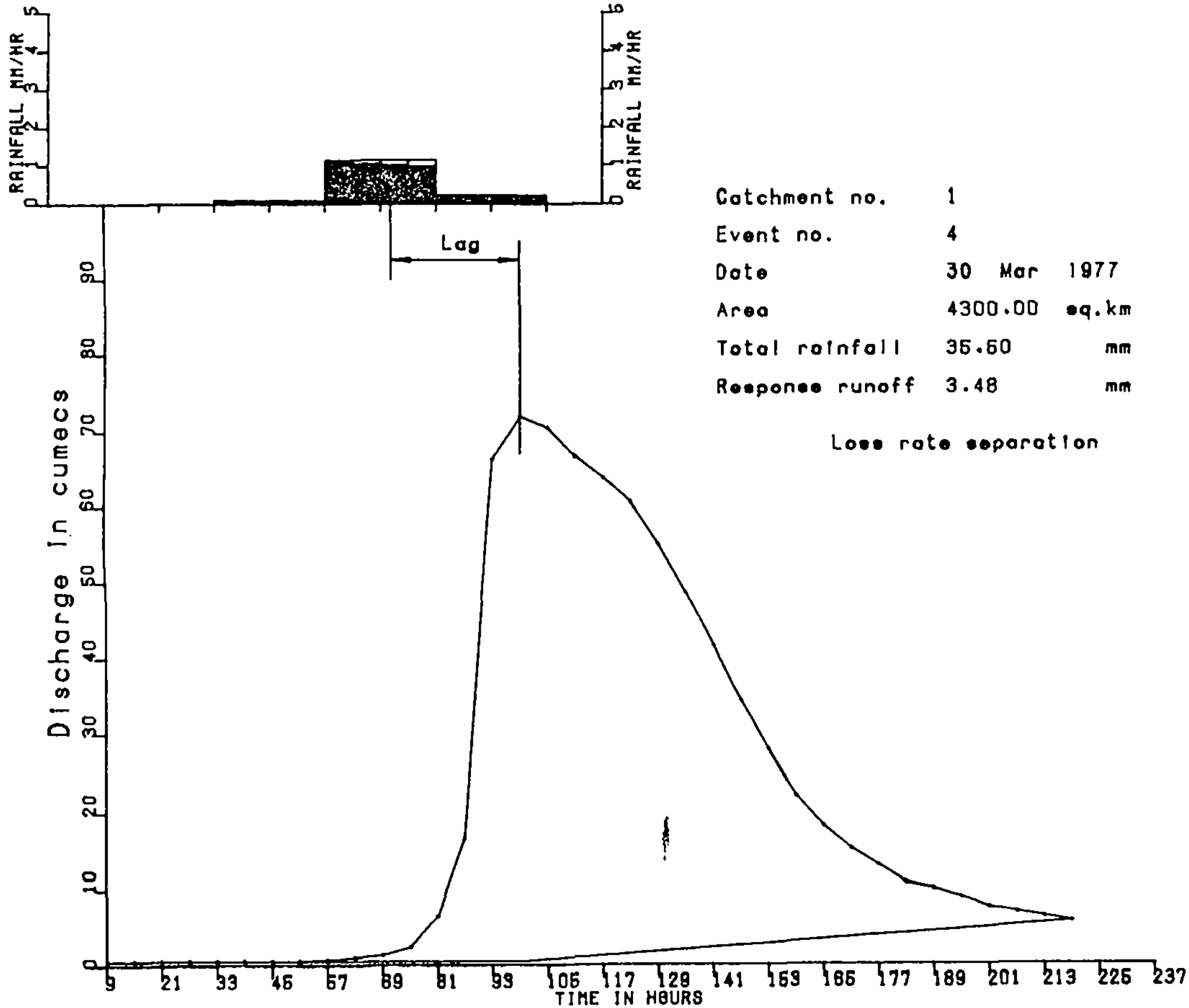


Fig.9 Inflow hydrograph and rainfall

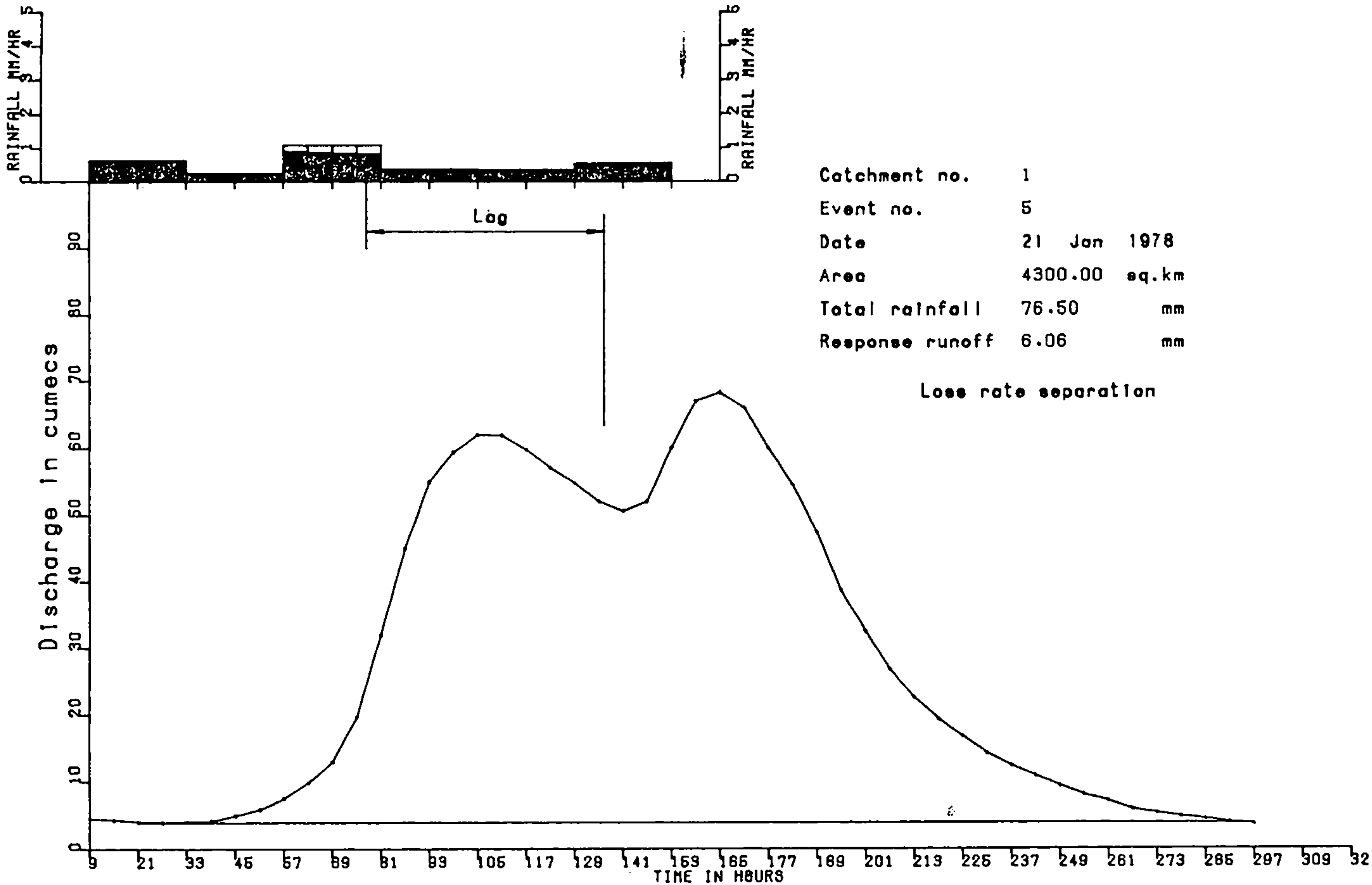


Fig.10 Inflow hydrograph and rainfall

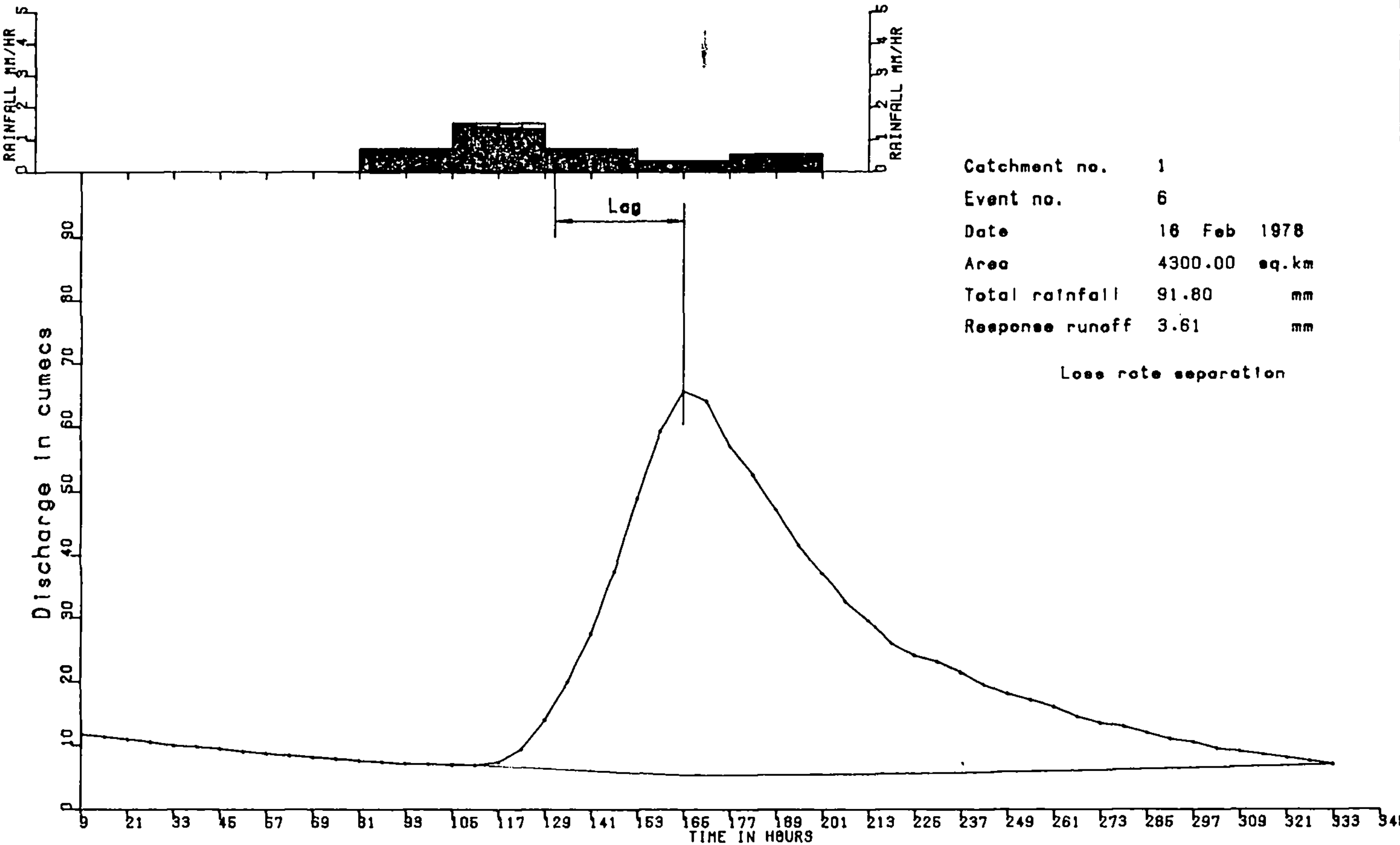


Fig.11 Inflow hydrograph and rainfall

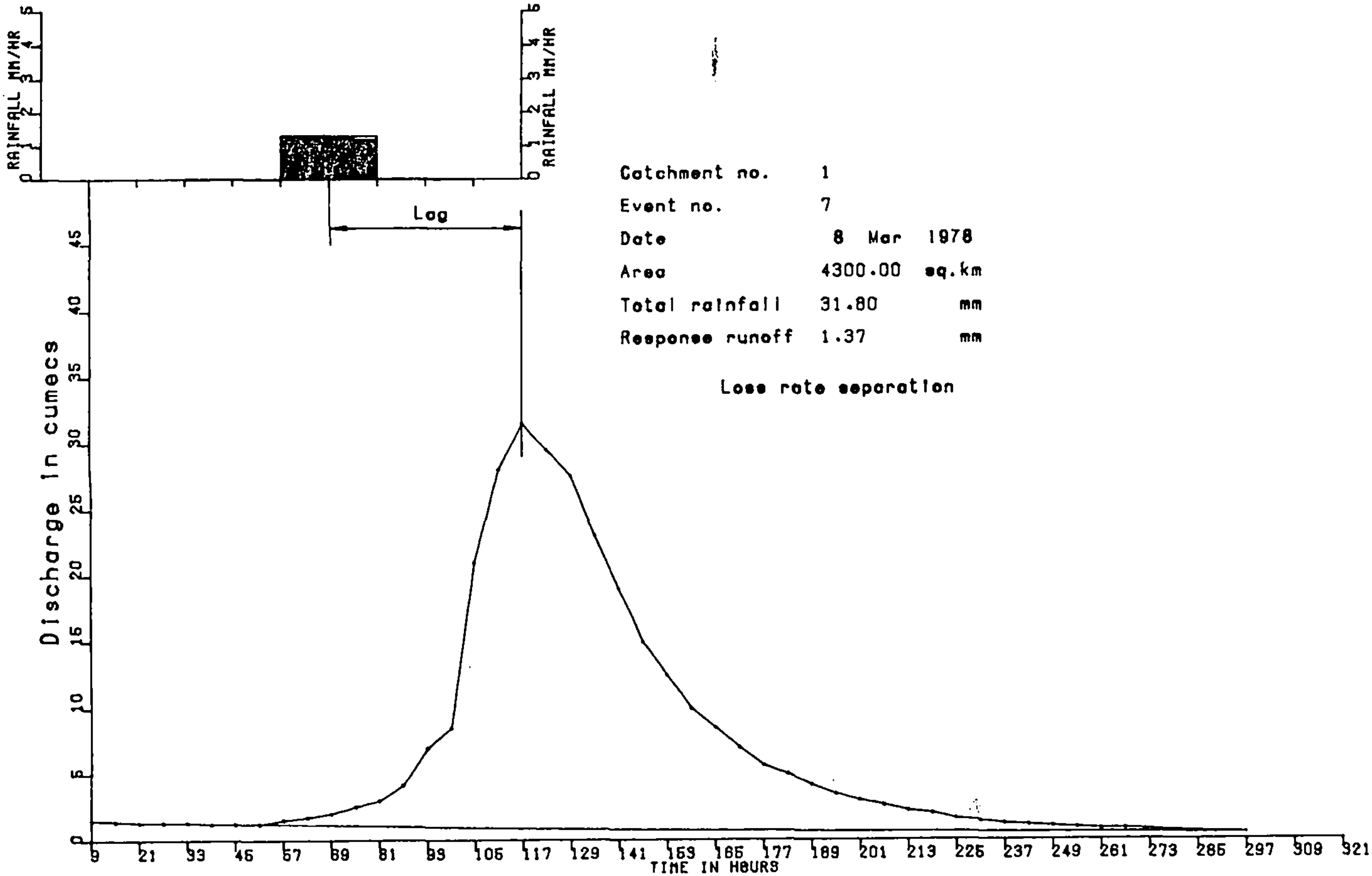
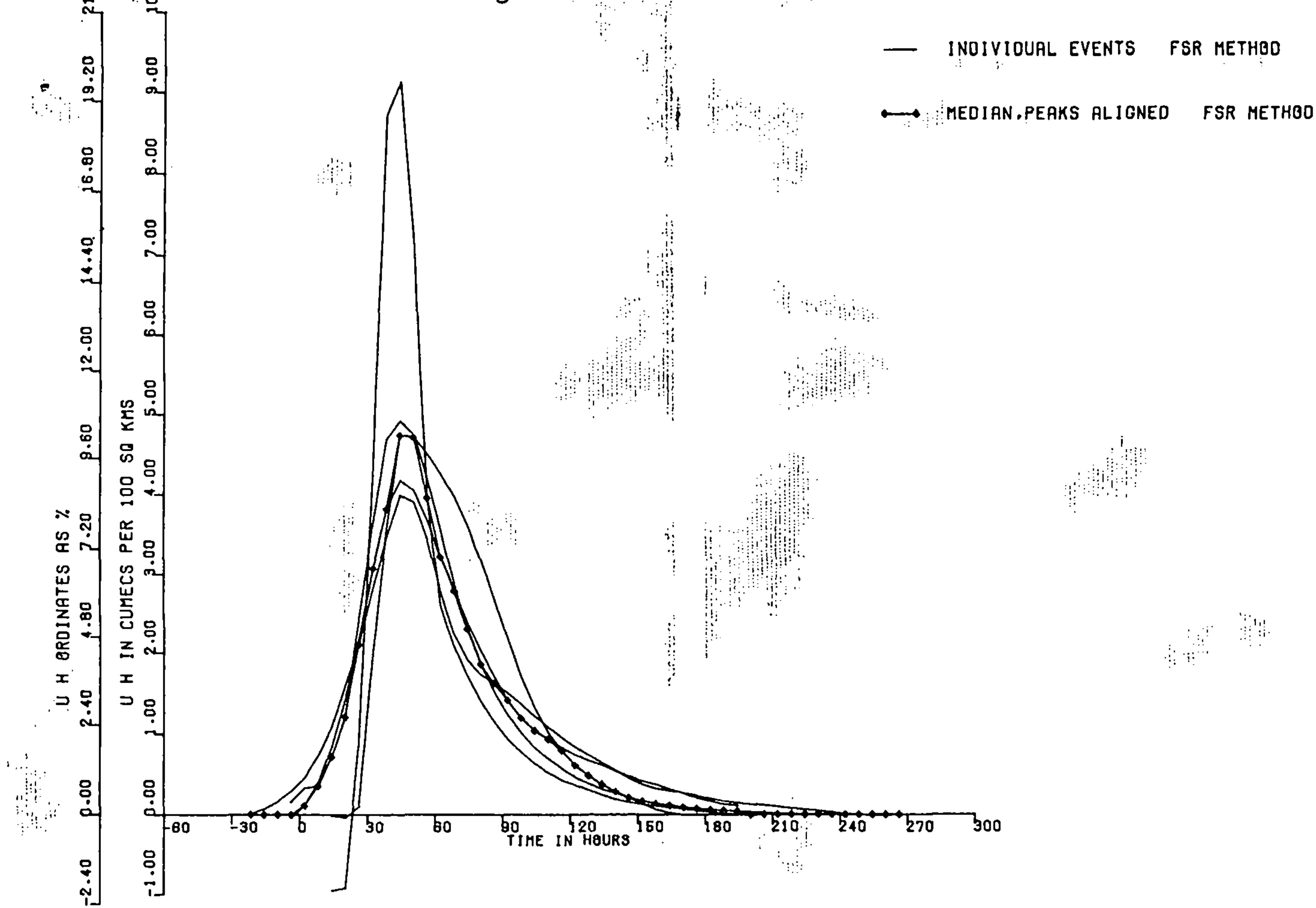


Fig 12 UNIT HYDROGRAPHS FOR CATCHMENT 1



The median was adopted as the design unit hydrograph for the Gaborone catchment (Figure 12) in preference to the mean because of its more clearly defined start time and smoother rising limb. This unit hydrograph is shown again on Figure 13. Additional information from the analysis of the five events is given in Table 6.

2.5 DESIGN PARAMETERS

In order to estimate the design floods using the unit hydrograph derived above it is necessary to choose the return period of the design storm, storm duration and profile, percentage runoff, baseflow and antecedent conditions for the catchment. These are considered in the following sections.

Rainfall return period

For this study it has been assumed that the storm and flood return period are equal (ie the 200 year return period storm is used to produce the 200 year return period flood). In practice the response depends on antecedent catchment conditions which vary from event to event, but the assumption is reasonable if median values of catchment conditions are assumed.

Rainfall duration

The FSR recommends the following equation for the duration of the design storm:

$$D = Tp\ (1 + SAAR/1000)$$

where SAAR = catchment average annual rainfall = 541 mm

T_p = time to peak of the unit hydrograph = 48 hours

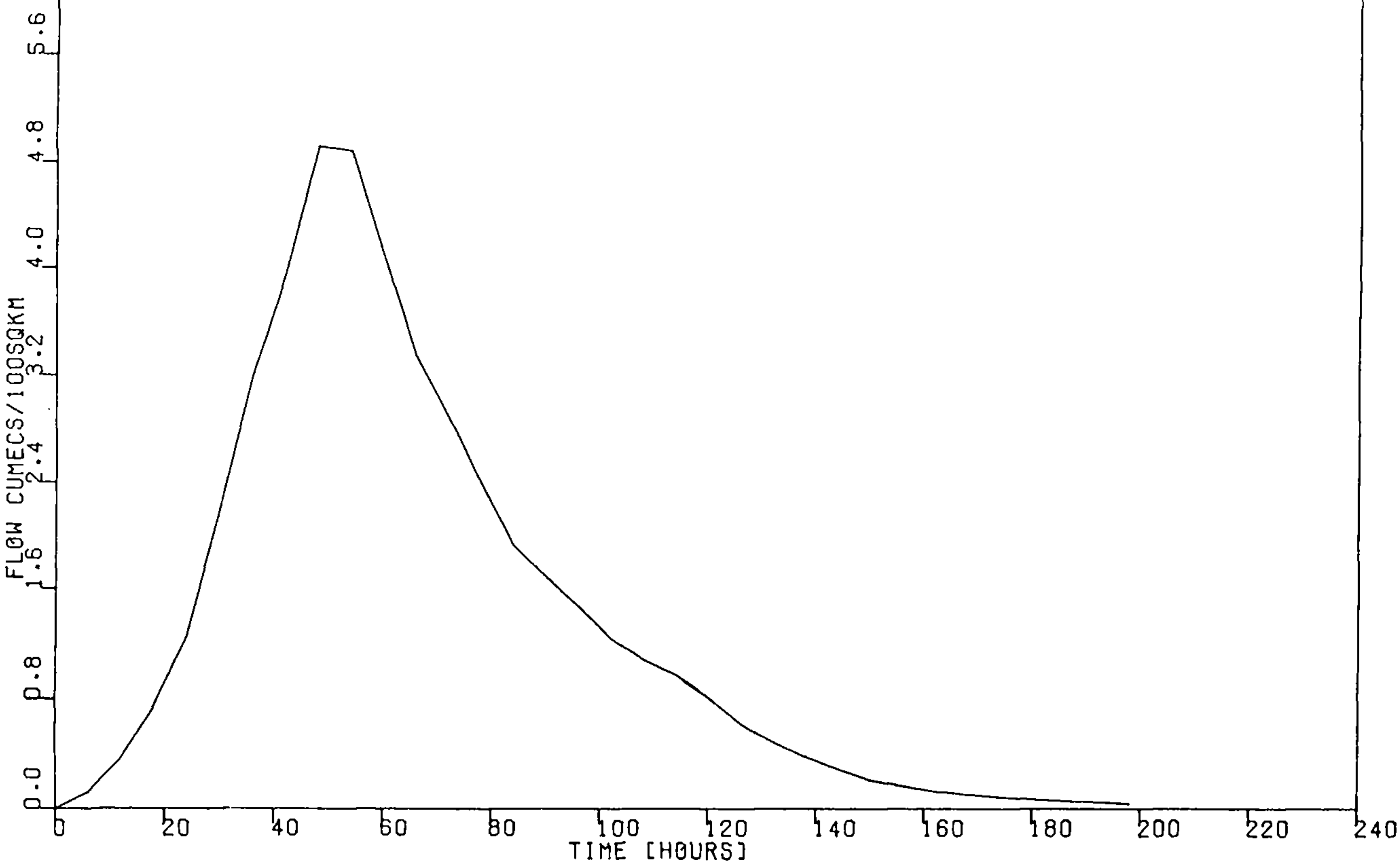
From this equation a storm duration of 74 hours is obtained. However, D should, for convenience, be an odd multiple of the data interval (6 hours). The nearest higher value is 78 hours. In fact the magnitude of the flood peak is relatively insensitive to storm duration since most of the rain falls within the central section.

Rainfall profile

Although HRU Report No 1/69 (Ref 4) gives rainstorm profiles for

Fig 13

Design Unit Hydrograph From 10mm Rainfall



durations up to 25 hours it was considered unwise to extrapolate this relationship to the required duration 78 hours. A nested profile was therefore adopted such that for all durations the rainfall intensities of the same return period occurred within the same storm. The 1 in 200 year storm of 78 hours duration was composed of the 1 in 200 year 18 hour fall etc. Although the average intensity during any part of the storm does not exceed 1 in 200 years, nesting the profile in this way tends to create a larger flood because of its peaky nature. Figures 14 and 15 show the 1 in 200 and 1 in 500 year rainfall profiles for the Gaborone catchment. The small increase in rainfall away from the storm centre is due to the break in slope of the rainfall intensity/duration graph at 24 hours (discussed earlier). However, this was not significant in the estimation of design floods.

Percentage runoff

The percentage of the rainfall contributing to large return period storms is a critical factor in the estimation of the magnitude of the design flood hydrograph. In the United Kingdom, FSR (Ref 3) practice is to relate percentage runoff to three factors. Firstly a standard percentage runoff (SPR) for the catchment is determined which defines the contribution due to the physiographic properties of the catchment (ie soil type, slope and vegetation). Secondly SPR is increased by the size of the rainstorm (ie more severe storms have a higher percentage runoff than others) and thirdly the percentage runoff is governed by how wet the catchment is prior to the flood event.

Although SPR in the United Kingdom ranges from 15% to 50% depending on soil type it is clear from the floods studied on the Gaborone catchment (Table 6) that runoff percentages are much lower (3.4% to 9.8%). Monthly percentage runoff, which might be anticipated to be lower than those for individual flood events, was computed from December 1965 to January 1980. Over this period the maximum observed percentage runoff was 7.8%. These low percentages, both monthly and on a flood event basis, are to be expected considering the catchment is situated in a semi-arid zone. In view of the limited amount of data available a standard percentage runoff of 10% has been adopted for the design flood. If more accurate rainfall and flow data were to become available it would be reasonable to review this conservative assumption.

Fig 14   1 in 200 year design rainstorm

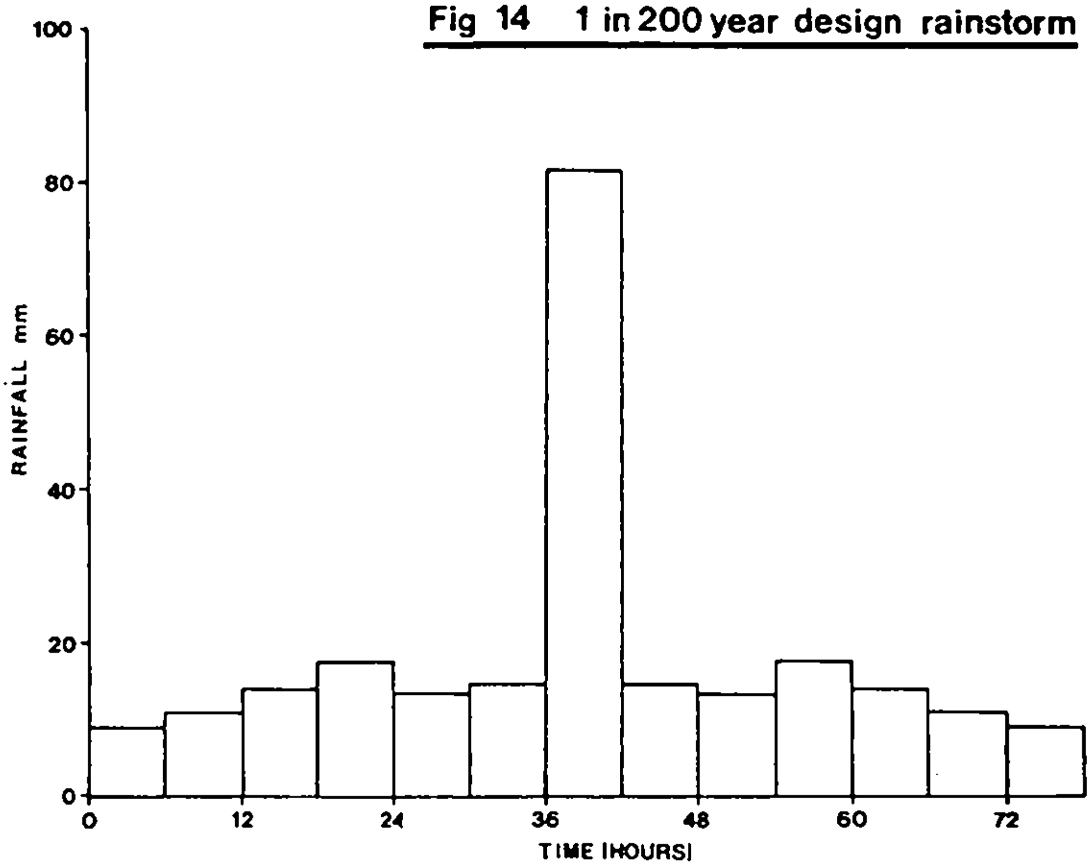
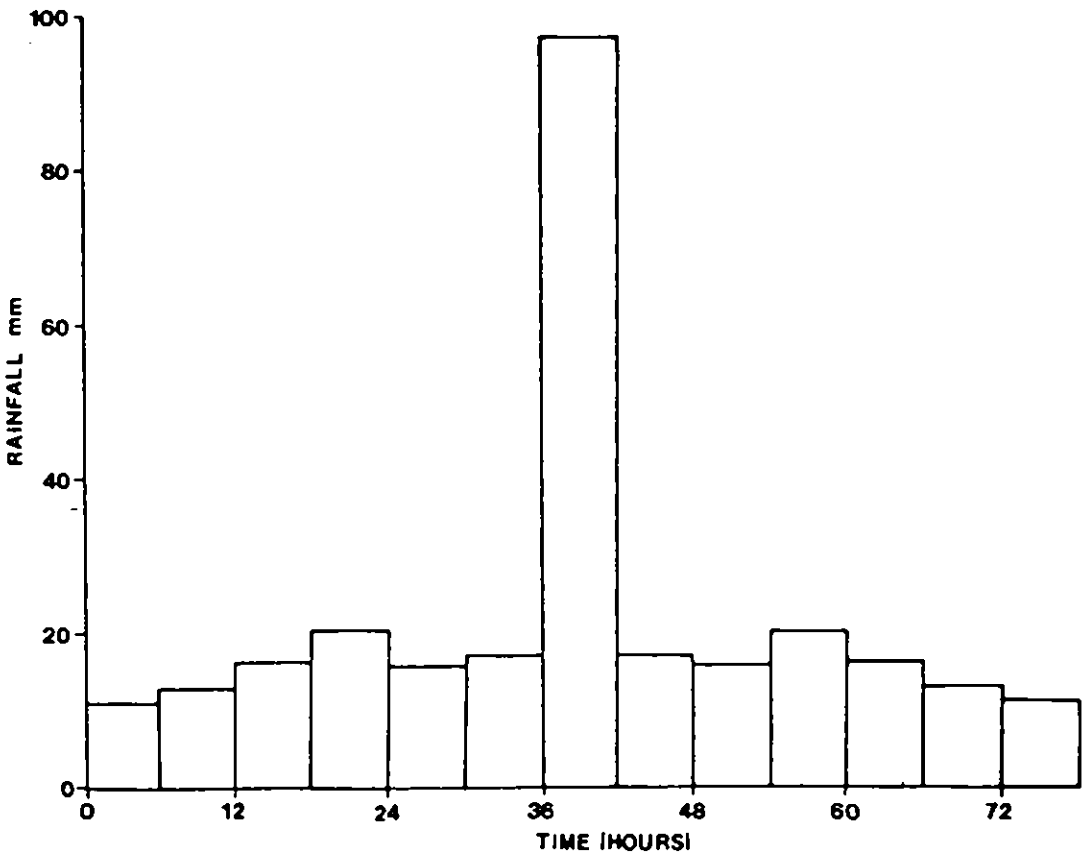


Fig 15   1 in 500 year design rainstorm



In the absence of local information to the contrary, the increase in percentage runoff due to size of rainstorm and the initial wetness of the catchment has been calculated using FSR (Ref 3) recommendations:

$$PR = SPR + 0.1(P-10) + 0.22\ (CWI - 125).$$

where P = total storm rainfall in mm

PR = storm percentage runoff

The average initial CWI from the five storms studied given in Table 6 (130.6) was taken to be representative of the state of the catchment preceding the design flood events.

Design storm percentage runoffs increased from 24% for the 1 in 20 year return period to 39% for the 1 in 500 year return period flood estimates.

Baseflow

The average baseflow of 2.9 m^3/s given in Table 6 for the five flood events used in the unit hydrograph analysis was taken to be representative of baseflow during the design floods. Baseflow is only a small proportion of the flood hydrograph and its value is therefore not critical to the flood estimates.

2.6 FLOOD ESTIMATE RESULTS

The design storms discussed above were multiplied by the appropriate percentage runoff and convoluted with the unit hydrograph. To this the baseflow was added to give estimates of the 20, 50, 100, 200 and 500 year return period floods (Tables 8, 9, 10, 11 and 12). Peak discharges and flood volumes are summarized in Table 7. Flood hydrographs are shown on Figure 16.

2.7 CONCLUSIONS AND RECOMMENDATIONS

In this analysis important assumptions have had to be made about rainfall profile and percentage runoff; these should be reviewed when more data become available. A plot of annual maximum daily inflows to Gaborone Dam from the period 1965 to 1979 indicates that, for the lower return period flood at least, estimates of floods are conservative. However, with the limitations of the present data in mind, it is considered advisable to use

Table 8 Gaborone Dam Botswana - 1 in 20 year flood

Nested rainfall profile

Area (Sq.Km.)	4300.00
Data interval (Hr)	6.00
Design duration (Hr)	78.00
Total rain (mm)	141.28
Percentage runoff	24.36
base flow (cumecs per sq.km)	.00067
CWI at start of storm	130.60

Convolution of unit hydrograph and net rain profile

Time	Total Rain mm	Net Rain mm	Unit hydrograph ordinate	Total hydrograph cumecs	
.00	5.32	1.30	.00	2.90	
6.00	6.58	1.60	.12	3.55	
12.00	8.24	2.01	.37	5.76	
18.00	10.13	2.47	.74	10.58	
24.00	7.92	1.93	1.25	19.41	
30.00	8.44	2.06	2.19	34.99	
36.00	48.02	11.70	3.19	58.46	
42.00	8.44	2.06	3.96	94.34	
48.00	7.92	1.93	4.92	144.87	
54.00	10.13	2.47	4.88	202.63	
60.00	8.24	2.01	4.10	263.49	
66.00	6.58	1.60	3.34	337.22	
72.00	5.32	1.30	2.90	406.92	
78.00			2.40	460.88	
84.00			1.93	517.79	
90.00			1.69	528.45	-Peak-
96.00			1.47	500.13	
102.00			1.24	465.72	
108.00			1.08	433.87	
114.00			.97	388.95	
120.00			.82	337.98	
126.00			.63	292.00	
132.00			.50	248.49	
138.00			.39	209.39	
144.00			.30	178.40	
150.00			.22	152.90	
156.00			.18	128.92	
162.00			.14	106.28	
168.00			.11	87.97	
174.00			.09	72.30	
180.00			.08	58.90	
186.00			.06	47.12	
192.00			.05	37.98	
198.00			.04	30.29	
204.00				24.41	
210.00				19.70	
216.00				16.03	
222.00				13.04	
228.00				10.97	
234.00				9.16	
240.00				6.28	
246.00				5.38	
252.00				4.66	
258.00				3.98	
264.00				3.49	
270.00				3.14	

Total Flood Volume (cubic metres) 150986136.000
Curvature around peak -1.063

Table 9 Gaborone Dam Botswana - 1 in 50 year flood

Nested rainfall profile

Area (Sq.Km.)	4300.00
Data interval (Hr)	6.00
Design duration (Hr)	78.00
Total rain (mm)	178.01
Percentage runoff	28.03
Base flow (cumecs per sq.km)	.00067
CWI at start of storm	130.60

Convolution of unit hydrograph and net rain profile

Time	Total Rain mm	Net Rain mm	Unit Hydrograph ordinate	Total Hydrograph cumecs	
.00	6.70	1.88	.00	2.90	
6.00	8.29	2.32	.12	3.84	
12.00	10.38	2.91	.37	7.05	
18.00	12.77	3.58	.74	14.03	
24.00	9.98	2.80	1.25	26.84	
30.00	10.63	2.98	2.19	49.43	
36.00	60.50	16.96	3.19	83.49	
42.00	10.63	2.98	3.96	135.48	
48.00	9.98	2.80	4.92	208.76	
54.00	12.77	3.58	4.88	292.51	
60.00	10.38	2.91	4.10	380.76	
66.00	8.29	2.32	3.34	487.67	
72.00	6.70	1.88	2.90	588.73	
78.00			2.40	666.97	
84.00			1.93	749.49	
90.00			1.69	764.94	-Peak-
96.00			1.47	723.86	
102.00			1.24	673.99	
108.00			1.06	627.81	
114.00			.97	562.67	
120.00			.82	486.77	
126.00			.63	422.10	
132.00			.50	359.01	
138.00			.39	302.32	
144.00			.30	257.38	
150.00			.22	220.40	
156.00			.18	185.64	
162.00			.14	152.80	
168.00			.11	126.25	
174.00			.09	103.53	
180.00			.08	84.09	
186.00			.06	67.03	
192.00			.05	53.77	
198.00			.04	42.61	
204.00				34.09	
210.00				27.27	
216.00				21.95	
222.00				17.60	
228.00				14.61	
234.00				11.98	
240.00				7.80	
246.00				6.49	
252.00				5.45	
258.00				4.46	
264.00				3.76	
270.00				3.25	

Total Flood Volume (cubic metres) 217633430.000
Curvature around peak -1.570

Table 10 Gaborone Dam Botswana - 1 in 100 year flood

Nested rainfall profile

Area (Sq.Km.)	4300.00
Data interval (Hr)	6.00
Design duration (Hr)	78.00
Total rain (mm)	209.10
Percentage runoff	31.14
Base flow (cumecs per sq.km)	.00067
CWI at start of storm	130.60

Convolution of unit hydrograph and net rain profile

Time	Total Rain mm	Net Rain mm	Unit hydrograph ordinate	Total Hydrograph cumecs	
.00	7.87	2.45	.00	2.90	
6.00	9.73	3.03	.12	4.13	
12.00	12.20	3.80	.37	8.32	
18.00	15.00	4.67	.74	17.42	
24.00	11.73	3.65	1.25	34.13	
30.00	12.49	3.89	2.19	63.62	
36.00	71.06	22.13	3.19	108.06	
42.00	12.49	3.89	3.96	175.90	
48.00	11.73	3.65	4.92	271.52	
54.00	15.00	4.67	4.88	380.80	
60.00	12.20	3.80	4.10	495.95	
66.00	9.73	3.03	3.34	635.44	
72.00	7.87	2.45	2.90	767.32	
78.00			2.40	869.40	
84.00			1.93	977.07	
90.00			1.69	997.24	-Peak-
96.00			1.47	943.66	
102.00			1.24	876.57	
108.00			1.08	818.30	
114.00			.97	733.30	
120.00			.82	636.88	
126.00			.63	549.88	
132.00			.50	467.57	
138.00			.39	393.59	
144.00			.30	334.95	
150.00			.22	286.70	
156.00			.18	241.34	
162.00			.14	198.49	
168.00			.11	163.85	
174.00			.09	134.21	
180.00			.08	108.85	
186.00			.06	86.57	
192.00			.05	69.27	
198.00			.04	54.71	
204.00				43.60	
210.00				34.69	
216.00				27.75	
222.00				22.08	
228.00				18.18	
234.00				14.74	
240.00				9.29	
246.00				7.59	
252.00				6.22	
258.00				4.94	
264.00				4.02	
270.00				3.35	

Total Flood Volume (cubic metres)	283097536.000
Curvature around peak	-.049

Table 11 Gaborone Dam Botswana - 1 in 200 year flood

Nested rainfall profile

Area (Sq.Km.)	4300.00
Data interval (Hr)	6.00
Design duration (Hr)	78.00
Total rain (mm)	240.18
Percentage runoff	34.25
Base flow (cumecs per sq.km)	.00067
CWI at start of storm	130.60

Convolution of unit hydrograph and net rain profile

Time	Total Rain mm	Net Rain mm	Unit Hydrograph ordinate	Total Hydrograph cumecs	
.00	9.04	3.10	.00	2.90	
6.00	11.18	3.83	.12	4.45	
12.00	14.01	4.80	.37	9.75	
18.00	17.23	5.90	.74	21.25	
24.00	13.47	4.61	1.25	42.36	
30.00	14.35	4.91	2.19	79.61	
36.00	81.63	27.96	3.19	135.75	
42.00	14.35	4.91	3.96	221.45	
48.00	13.47	4.61	4.92	342.24	
54.00	17.23	5.90	4.88	480.29	
60.00	14.01	4.80	4.10	625.76	
66.00	11.18	3.83	3.34	801.98	
72.00	9.04	3.10	2.90	968.58	
78.00			2.40	1097.55	
84.00			1.93	1233.57	
90.00			1.65	1259.04	-Peak-
96.00			1.47	1191.35	
102.00			1.26	1109.12	
108.00			1.08	1032.99	
114.00			.97	925.61	
120.00			.82	803.81	
126.00			.63	693.90	
132.00			.50	589.91	
138.00			.39	496.45	
144.00			.30	422.38	
150.00			.22	361.43	
156.00			.18	304.12	
162.00			.14	249.99	
168.00			.11	206.22	
174.00			.09	168.78	
180.00			.08	136.74	
186.00			.06	108.60	
192.00			.05	86.75	
198.00			.04	68.36	
204.00				54.32	
210.00				43.07	
216.00				34.29	
222.00				27.12	
228.00				22.20	
234.00				17.86	
240.00				10.97	
246.00				8.82	
252.00				7.10	
258.00				5.47	
264.00				4.31	
270.00				3.47	

Total Flood Volume (cubic metres)	356676476.000
Curvature around peak	-2.588

Table 12 Gaborone Dam Botswana - 1 in 500 year flood

Nested rainfall profile

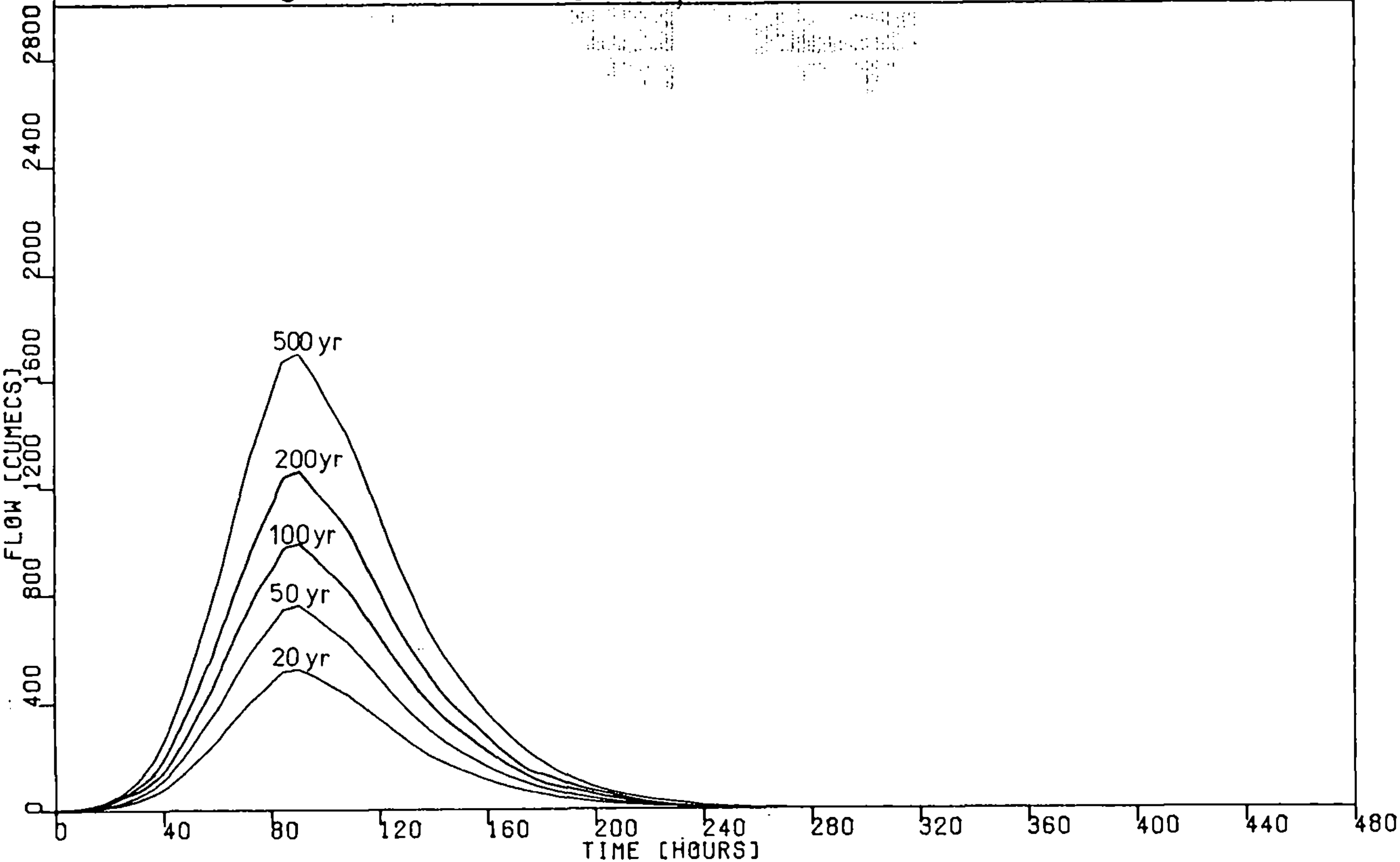
Area (Sq.Km.)	4300.00
Data interval (Hr)	6.00
Design duration (Hr)	78.00
Total rain (mm)	286.80
Percentage runoff	38.91
Base flow (cumecs per sq.km)	.00067
CWI at start of storm	130.60

Convolution of unit hydrograph and net rain profile

Time	Total Rain mm	Net Rain mm	Unit Hydrograph ordinate	Total Hydrograph cumecs	
.00	10.79	4.20	.00	2.90	
6.00	13.35	5.20	.12	5.00	
12.00	16.73	6.51	.37	12.19	
18.00	20.57	8.00	.74	27.79	
24.00	16.09	6.26	1.25	56.43	
30.00	17.13	6.67	2.19	106.96	
36.00	97.47	37.93	3.19	183.13	
42.00	17.13	6.67	3.96	299.40	
48.00	16.09	6.26	4.92	463.28	
54.00	20.57	8.00	4.88	650.56	
60.00	16.73	6.51	4.10	847.91	
66.00	13.35	5.20	3.34	1086.99	
72.00	10.79	4.20	2.90	1313.01	
78.00			2.40	1487.97	
84.00			1.93	1672.50	
90.00			1.69	1707.07	-Peak-
96.00			1.47	1615.23	
102.00			1.24	1503.66	
108.00			1.08	1400.40	
114.00			.97	1254.72	
120.00			.82	1089.46	
126.00			.63	940.36	
132.00			.50	799.28	
138.00			.39	672.49	
144.00			.30	572.00	
150.00			.22	489.30	
156.00			.18	411.56	
162.00			.14	338.12	
168.00			.11	278.74	
174.00			.09	227.95	
180.00			.08	184.48	
186.00			.06	146.31	
192.00			.05	116.65	
198.00			.04	91.70	
204.00				72.66	
210.00				57.39	
216.00				45.49	
222.00				35.77	
228.00				29.08	
234.00				23.20	
240.00				13.85	
246.00				10.93	
252.00				8.60	
258.00				6.39	
264.00				4.82	
270.00				3.67	

Total Flood Volume (cubic metres)	483134848.000
Curvature around peak	-3.511

Fig 16 Design Flood Hydrographs



the flood estimates as stated. The derivation of the catchment unit hydrograph has been based on relatively crude data. Nevertheless it was considered preferable to use this rather than a unit hydrograph derived from catchment characteristics alone.

The flood estimates quoted are the total inflow hydrograph into Gaborone reservoir. They must be routed through the proposed reservoir for spillway design purposes.

RESERVOIR YIELD ANALYSIS

Estimates of the 10 year, 20 year and 50 year return period yield are required for several proposed dam heights taking into consideration the likely amount of sedimentation. For this purpose revised area capacity tables have been derived assuming that $0.2 \times 10^6 m^3$ of sediment will be deposited annually, half of which will be added to the dead storage of the reservoir whilst the rest will be deposited evenly over the area under the top water level of the dam. These tables (calculated for the 1985 and 2035 conditions) are reproduced in Table 13.

In previous reservoir studies for Botswana the yields have been calculated by the method of Midgley and Pitman (6) based on the mean annual runoff (MAR) at the reservoir site and a drought region selected by climatological characteristics. Regional critical mass curves are used together with the reservoir geometry and evaporation to estimate the yield. This method is very useful in areas where the data are not sufficient for a reservoir operation study and where the proposed reservoir capacity is less than 200% of the mean annual runoff. In this case, however, the reservoir capacities of interest could be much higher and we believe that the best estimates of yields will be achieved by extending the inflow series and using reservoir simulation.

3.1 EXTENSION OF INFLOW DATA

As there are 20 years of runoff data and 57 years of rainfall data the runoff series was extended using the Pitman monthly model (Ref 7) and the monthly rainfall values for the catchment. The model parameters used by Pitman for the Gaborone catchment (Ref 8) were used as initial estimates. These were then optimised to fit the 20 years of inflow data by comparing the mean, standard deviation and seasonal distribution of the observed and predicted flows. A logarithmic transformation was applied before a comparison was made because the distribution of flows is highly skewed and a few very high flows would dominate the statistics of the data Table (14) shows the comparison of the observed and synthetic inflows with a difference of less than 5% in the mean and standard deviation of the logarithms and Table 15 lists the 20 years of synthetic flow from 1959 to 1978.

TABLE 13: REVISED AREA/CAPACITY TABLES FOR GABORONE RESERVOIR

Level (m)	1985		2035 Existing dam		2035 Raised dam	
	Area (km^2)	Capacity ($m^3 \times 10^6$)	Area (km^2)	Capacity ($m^3 \times 10^6$)	Area (km^2)	Capacity ($m^3 \times 10^6$)
981.7	1.4	0				
982	1.5	0.4				
983	2.0	2.2				
984	2.6	4.5	1.6	0	1.6	0
985	3.5	7.5	2.4	2.0	3.1	2.3
986	4.1	11.3	3.6	5.0	3.7	5.7
987	4.9	15.8	4.2	8.9	4.4	9.8
988	5.7	21.1	4.7	13.3	5.1	14.5
989	6.3	27.1	5.3	18.3	5.9	20.0
990	7.4	33.9	5.9	23.9	6.8	26.3
991	9.1	42.4	9.1	32.4	8.6	34.0
992	10.2	52.1	10.2	42.1	9.6	43.1
993	11.9	63.5	11.9	53.5	11.1	53.5
994	13.1	76.1	13.1	66.1	13.1	66.1
995	14.4	90.0	14.4	80.0	14.4	80.0
996	15.9	105.2	15.9	95.2	15.9	95.2
997	17.5	122.1	17.5	112.1	17.5	112.1
998	19.0	140.4	19.0	130.4	19.0	130.4
999	20.5	160.3	20.5	150.3	20.5	150.3
1000	22.1	181.7	22.1	171.7	22.1	171.7
1001	24.0	205.0	24.0	195.0	24.0	195.0
1002	26.0	230.0	26.0	220.0	26.0	220.0
1003	28.0	257.0	28.0	247.0	28.0	247.0
1004	30.0	286.0	30.0	276.0	30.0	276.0
1005	32.0	317.0	32.0	307.0	32.0	307.0
1006	34.0	350.0	34.0	340.0	34.0	340.0
1007	36.0	385.0	36.0	375.0	36.0	375.0
1008	38.0	422.0	38.0	412.0	38.0	412.0

Values extrapolated above 1000m.

TABLE 14

COMPARISON OF SYNTHETIC AND OBSERVED INFLOWS

	Mean of logs	Standard Deviation of logs	Mean Annual Inflow
Reservoir data	1.108	0.782	33.95
Model	1.065	0.769	35.65
Difference % Reservoir values	- 3.88	- 1.66	5.01

(All values in million m^3)

The validity of the simulated 20 year series was further tested by using it as inflows to a reservoir operation program and comparing the synthetic end of month water levels with observed water levels for the same period.

The comparison can be seen, from Figure 17, to be very good except during the period 1972 to 1974 when the rainfall data do not correspond well with inflows derived from the measured water levels. This is caused by the sparse nature of the rainfall values and the difficulties of accurately estimating spilling over the very wide dam. As the comparison becomes better when dealing with more recent data we conclude that the model is able to produce realistic estimates of reservoir inflows. The inflow series was therefore extended to a 57 year series, as shown in Table 16, using the rainfall data from 1922 to 1959 and derived inflows from 1959 to 1978.

Table 15.

SYNTHESIZED RUNOFF AT GAUGE GARS CATCHMENT AREA= 4300.50.KM M.A.P.= 540.MM

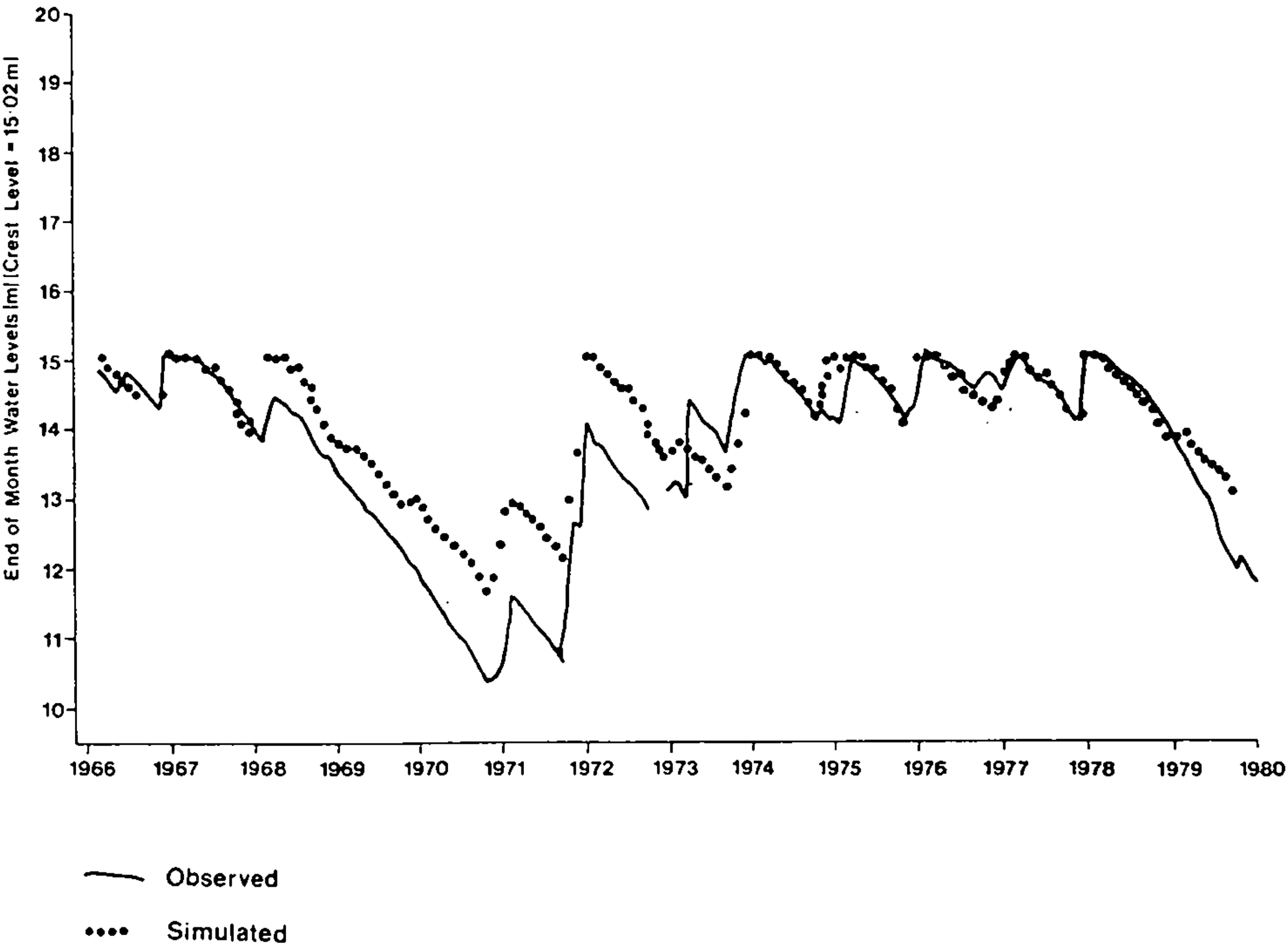
ELEVAP.	196.1	200.7	210.5	215.1	175.5	160.7	117.6	80.2	58.6	69.0	100.9	148.4	MM-R= .5
YEAR	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	TOTAL
1959	.00	.27	4.51	4.27	.03	.09	.12	.02	.00	.00	.00	.00	9.32
1960	.00	3.86	10.48	6.67	1.33	1.35	6.80	6.82	.06	.00	.00	.00	37.34
1961	.00	.01	.01	.02	.03	.01	.03	.03	.00	.00	.00	.00	.13
1962	.00	2.81	2.82	1.13	1.12	.00	.00	.00	.00	.00	.00	.00	7.88
1963	.64	.66	.24	.24	.24	.01	.01	.00	.00	.00	.00	.00	2.25
1964	.03	.03	.19	.48	.29	.00	.39	.39	.00	.00	.00	.00	1.81
1965	.00	.11	.11	.72	17.45	16.73	.00	.00	.00	.00	.00	.00	35.12
1966	.01	.01	4.25	37.63	43.09	11.26	40.72	39.17	.00	.00	.00	.00	176.14
1967	.02	.05	.21	.49	.35	12.35	14.42	2.03	.00	.00	.00	.00	30.01
1968	.00	.17	.36	.19	.13	.30	.38	.20	.00	.00	.00	.00	1.73
1969	.10	.13	1.11	1.42	.34	.00	.00	.00	.00	.00	.00	.00	3.11
1970	.00	.22	1.45	3.57	3.80	1.45	.05	.65	.00	.00	.00	.00	10.66
1971	.16	5.11	4.94	52.16	52.18	.23	.20	.00	.00	.00	.00	.00	115.00
1972	.00	.00	.15	.42	1.66	1.59	.00	.00	.00	.00	.00	.02	4.04
1973	2.48	3.24	3.96	20.52	20.24	4.22	1.48	.13	.00	.00	.00	.00	56.25
1974	.00	3.09	5.00	4.91	3.31	5.06	6.61	1.87	.02	.00	.00	.00	29.89
1975	.00	.08	10.01	20.55	19.38	14.06	5.31	.01	.00	.00	.00	.16	69.57
1976	.19	.69	1.27	2.40	3.12	14.48	14.06	.00	.00	.00	.00	.14	37.24
1977	.14	.00	.74	34.45	35.07	1.37	.01	.00	.00	.00	.00	.00	71.77
1978	.07	.07	.01	1.37	1.36	.00	.00	.00	.00	.00	.00	.00	2.88
% TOTAL	.55	2.96	7.38	27.72	29.16	12.06	12.90	7.23	.01	.00	.00	.05	100.00

MEAN(LOG)= 1.051216 STD. DEV.= .784470 MEAN ANNUAL RUNOFF= 35.11 MILLION CUBIC METRES

Table 16 Inflow Sequence Used For Reservoir Operation Trials

YEAR	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	TOTAL
1922/23	.0	.0	.8	19.2	25.3	7.0	.2	.0	.0	.0	.0	.0	52.5
1923/24	.0	.0	.1	.0	.0	2.1	2.1	.0	.0	.0	.0	.0	4.4
1924/25	.0	.0	18.1	18.6	2.3	9.5	11.9	5.0	.9	.0	.0	.0	66.3
1925/26	.0	.2	.2	2.2	2.2	.0	.0	.0	.0	.0	.0	.0	4.9
1926/27	.0	.8	.8	.2	.6	.6	.1	.0	.0	.0	.0	.0	3.2
1927/28	.1	.1	.4	4.4	6.3	2.6	.3	.0	.0	.0	.0	.0	14.4
1928/29	.0	.1	.5	1.2	.7	.4	.4	.0	.0	.0	.0	.0	3.3
1929/30	.0	3.8	4.6	2.5	1.9	1.3	1.5	.4	.0	.0	.0	.0	16.1
1930/31	.0	.2	1.0	2.8	2.0	.3	.4	.2	.0	.0	.0	.0	6.9
1931/32	.0	1.3	1.4	.2	1.4	1.6	1.4	1.2	.0	.0	.0	.0	8.5
1932/33	.0	.1	1.3	1.2	.0	.0	.0	.0	.0	.0	.0	.0	2.6
1933/34	.0	.5	10.0	27.2	18.2	.5	.0	.0	.0	.0	.0	.0	56.5
1934/35	.0	4.2	4.3	.1	6.9	8.0	1.1	.0	.0	.0	.0	.0	24.6
1935/36	.0	.0	.1	.2	3.2	16.4	13.3	.2	.2	.0	.0	.0	33.6
1936/37	.4	2.2	1.7	5.0	6.1	1.1	.0	.0	.0	.0	.0	.0	16.6
1937/38	.0	.0	6.1	8.5	.5	.2	.0	.0	.0	.0	.0	.0	17.2
1938/39	.0	.0	6.7	7.5	46.0	45.3	.1	.0	.0	.0	.0	.0	105.6
1939/40	.0	.4	1.5	1.7	.5	2.8	2.8	.0	.0	.0	.0	.2	10.0
1940/41	.2	.0	11.1	11.3	.3	.1	.0	.0	.0	.0	.0	.0	23.1
1941/42	.0	.0	.3	1.1	1.0	5.2	5.0	.0	.0	.0	.0	.0	12.6
1942/43	.3	.3	13.3	14.6	1.3	2.6	6.8	4.2	.0	.0	.0	1.0	44.3
1943/44	3.9	5.8	3.1	2.4	41.7	39.5	.0	.0	2.1	2.1	.0	.0	100.6
1944/45	.5	.6	.1	.0	.0	41.9	41.9	.0	.0	.0	.0	.0	84.9
1945/46	.0	.0	.0	11.0	17.8	7.2	.3	.0	.0	.0	.0	.0	36.4
1946/47	.1	.1	.8	1.5	.8	.5	.4	.0	.0	.0	.0	.0	4.2
1947/48	.0	.0	.2	1.5	1.3	.6	.8	.2	.0	.0	.0	.0	4.6
1948/49	.8	4.1	3.3	5.5	5.5	.0	.0	.0	.0	.0	.0	.0	19.3
1949/50	.0	.2	3.5	3.4	.1	.1	.1	.0	.0	.0	.0	.0	7.4
1950/51	.0	.0	15.0	15.9	1.0	.0	.6	.8	.2	.0	.0	.0	33.5
1951/52	.3	.3	.0	.0	.4	.5	.0	.0	.0	.0	.0	.0	1.5
1952/53	.0	1.5	4.1	2.6	.6	7.9	7.3	.0	.0	.0	.0	.0	24.1
1953/54	.0	.2	.2	.6	.9	.3	.1	.1	.0	.0	.0	.0	2.3
1954/55	.0	.3	1.5	10.3	65.3	56.1	.0	.0	.0	.0	.0	.0	133.5
1955/56	.2	.5	3.5	3.1	27.5	27.5	.0	.0	.0	.0	.0	.0	62.3
1956/57	.1	.1	.8	.9	1.0	1.9	1.0	.0	.0	.5	.5	.2	6.8
1957/58	.6	.9	1.0	13.6	13.4	.3	.0	.0	.0	.0	.0	.0	29.9
1958/59	.0	.7	6.2	6.2	.6	.0	.0	.1	.0	.0	.0	.0	13.8
1959/60	.0	.0	.9	.3	.0	.6	2.6	.0	.0	.0	.0	.0	4.4
1960/61	.0	.9	18.2	.0	5.7	1.3	29.8	.0	.0	.0	.0	.0	55.9
1961/62	.0	.0	.0	.0	.0	.0	6.2	1.1	.0	.0	.0	.0	7.3
1962/63	.0	3.2	4.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	7.2
1963/64	2.0	2.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	4.0
1964/65	3.7	9.9	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	13.6
1965/66	.0	.0	.7	10.2	73.2	.0	.0	1.7	.0	.0	.0	1.7	87.5
1966/67	.4	2.0	9.3	41.8	35.2	11.5	71.8	4.6	.2	.0	.5	.2	177.5
1967/68	.0	1.6	.0	1.6	.0	6.4	1.2	.1	1.6	.0	.0	1.4	13.9
1968/69	1.0	.6	.5	.1	.4	.0	.1	.0	.0	.0	.0	.0	2.7
1969/70	.0	.1	.1	.0	.0	.1	.0	.0	.0	.0	.0	.0	.3
1970/71	.0	.6	1.2	1.7	4.9	.7	.1	.0	.0	.0	.0	.0	9.2
1971/72	.0	11.9	.7	31.0	.0	.0	.0	.0	.0	.0	.0	.0	43.6
1972/73	.0	.0	.0	.0	.9	.0	10.6	.0	.0	.0	.0	.0	11.5
1973/74	5.8	3.4	10.7	14.5	19.6	13.3	1.2	.2	.0	.0	.0	.0	68.7
1974/75	.0	1.1	.4	.4	.0	4.3	7.2	2.9	.0	.0	.0	.0	16.3
1975/76	.4	.0	2.2	3.7	9.6	31.1	4.6	.6	.0	.0	.0	.0	52.2
1976/77	2.1	.1	.1	.4	2.5	19.9	16.8	.0	.0	.0	.0	.0	41.9
1977/78	.0	.0	.4	31.2	22.9	6.0	.0	.0	.0	.0	.0	.0	60.5
1978/79	.0	.0	.2	.0	.0	.0	.0	.0	.0	.0	.0	.0	.2
					8.4	6.8				.0			31.1
		2.3		9.2	15.9	12.7	11.7						36.3

Fig 17. Comparison of Simulated and Observed End of Month Reservoir Levels



3.2 ESTIMATION OF YIELDS

The yield of a particular return period may be estimated by the frequency of failure of a reservoir operated to meet this yield. For example, a yield which can be supplied for 990 years out of every 1000 (on average) would be a 100 year return period (T) yield; and the probability of occurrence of failure is 0.01, (i.e. 1/T). If the set of data is of several hundred years duration yields of 20 and 50 year return periods can be calculated with some confidence, but with only 57 years the estimates of these yields are subject to large errors, by this "failure rate" method. This method also ignores the extent of a failure, hence reducing the available information on which to base a design.

An alternative approach, which we have adopted, is to consider the reservoir capacity necessary to sustain the yield and to fit a statistical distribution to these capacities. The capacities required are known as the "deficient volumes" and are calculated as the volumes necessary to just sustain a yield through the worst droughts in the record of inflows. These deficient volumes (expressed as a percentage of the MAR) are ranked and plotted using a log normal plotting scheme with non-exceedance probability of the ith smallest storage given by Blom's plotting position

$$F_i = \frac{i - 0.375}{N + 0.25} \qquad N = \text{Total number of years}$$

thus a probability of failure is assigned, by the plotting position, to a storage for any particular yield.

The deficient volumes are usually calculated from the annual minimum water levels taken from one reservoir operation trial and considering a very large reservoir. In this case the evaporation is a very important factor and thus the yield which can be sustained through anything other than the worst drought in the sequence is grossly underestimated. To overcome this reservoir trials were carried out for several capacities for each yield and the return period at which each capacity just failed was calculated.

The analysis was carried out for yields in the range of 0.4 to 1.8 million m^3 per month and the results are plotted in Figures 18 and 19 for the sedimentation expected in 1985 and 2035 respectively. From these curves

Fig 18. Storage Yield Curves for 1985 Sedimentation Conditions

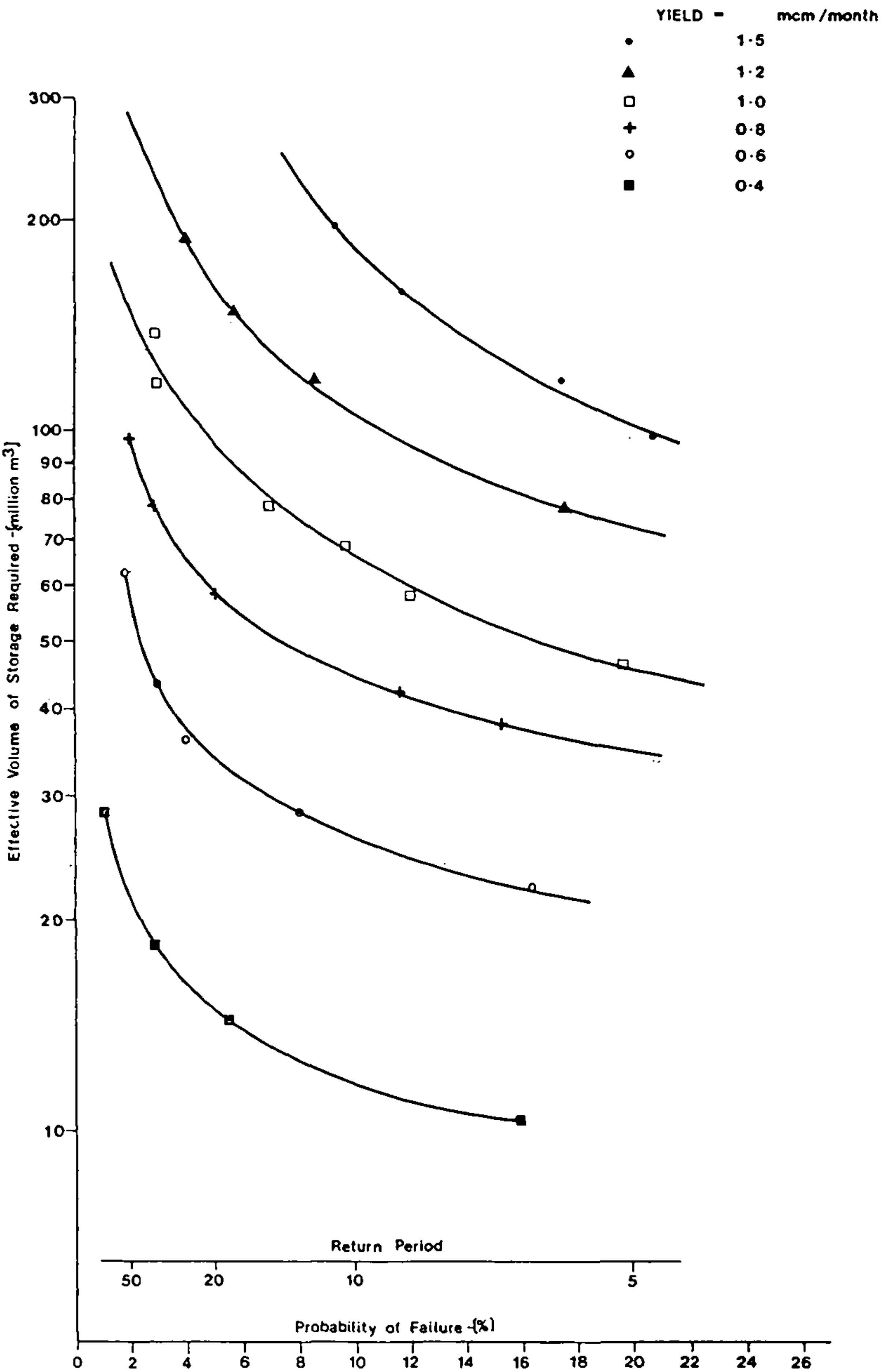
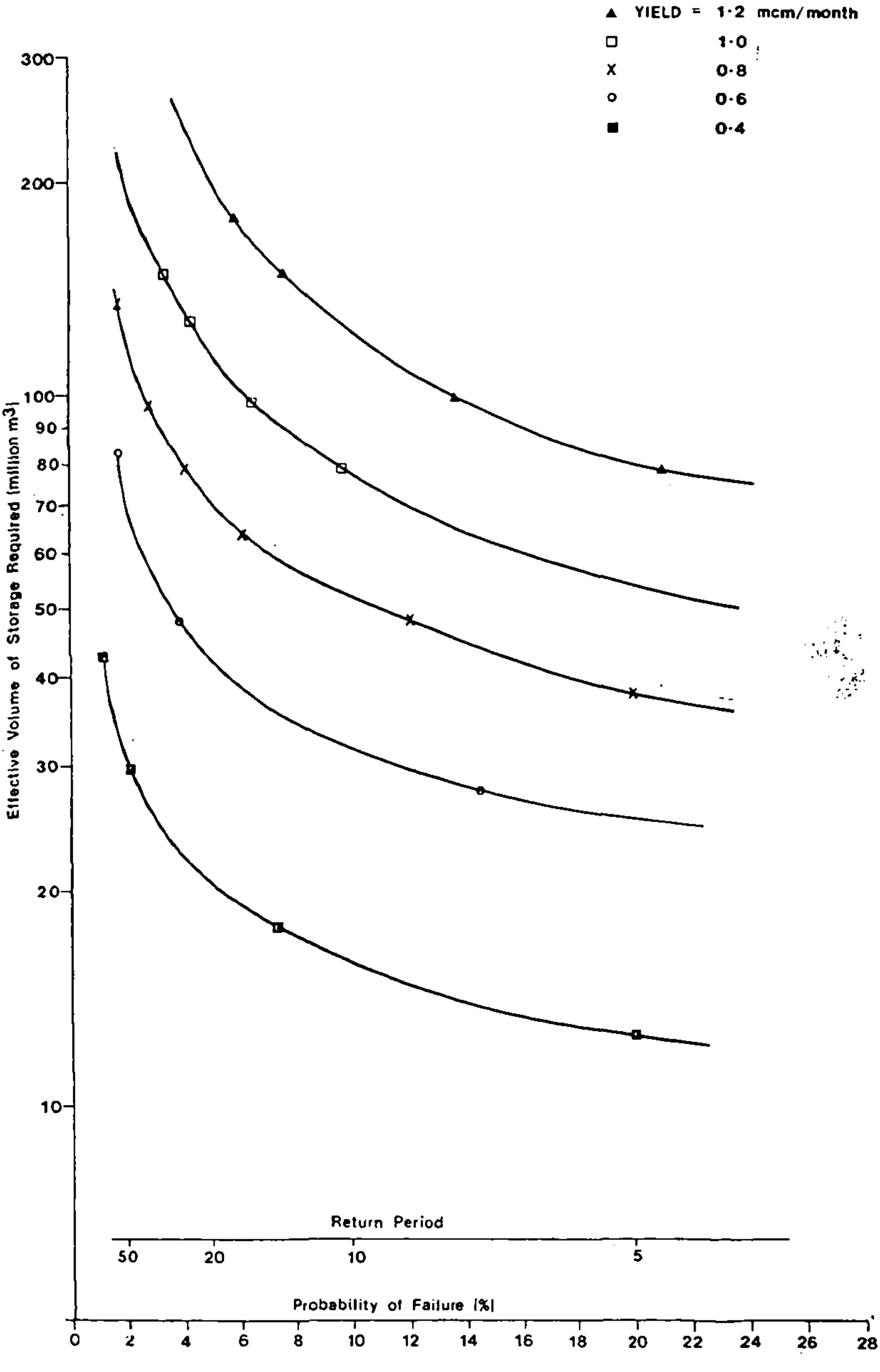


Fig 19. Storage Yield Curves for 2035 Sedimentation Conditions



the yield can be estimated for any capacity and for any probability of failure; thus Figures 20 and 21 delineate the expected 10, 20 and 50 year return period yields. The results for yield greater than 1.0 million m^3 per month are only indicative of the expected required capacities as the Gaborone Dam storage/area curves have had to be extrapolated for these calculations. The possible yield is very sensitive to the evaporation estimate and hence to the surface area assumed for the reservoir; for example, the evaporation calculated for the simple water balance in section (1.1 · *Evaporation*) was estimated as 11 million m^3 per year which was more than 5 times the historic yield. The yields for the future are greater than this but as larger capacities are considered the evaporation will still be important. Because of the crucial importance of the evaporation estimate it is necessary to use simulation as part of the analytical scheme for the yield calculations.

This analysis was compared with the failure rate method using the reservoir operation model with a capacity of 38 million m^3 sustaining yield of 0.6 million m^3. The reservoir failed twice during the 57 years of inflows, which indicates a return period of failure of between 19 and 28 years. From Fig 18 the probability of failure is 4% which corresponds to a 25 year return period, thus the failure rate and deficient volumes analyses do not conflict, but the latter provides more precise information concerning the return period of failure.

For this particular analysis the definition of a 20 year return period yield is one which can be provided, on average, 95% of the years; however, the inflow data indicate that inflows, of less than the mean annual value, tend to occur one after another and the first order serial correlation coefficient of the deficient volumes is 0.65. Therefore it is important not to consider independent years of inflows for the yield analysis, but to calculate the available yield from longer than one year duration droughts. The deficient volume analysis allows for this as it is based on historic droughts and not independent annual events.

The Immediate Effect of Raising the Dam

In the long term, the yield which may be expected for a given reservoir capacity is dependent only on the sequence of inflows. In the short term the likelihood of meeting that yield will tend to be less than the long term reliability if, at the time the forecast is made, the reservoir is not full.

Fig. 20. **Reservoir Capacity Required for 1985 Conditions**

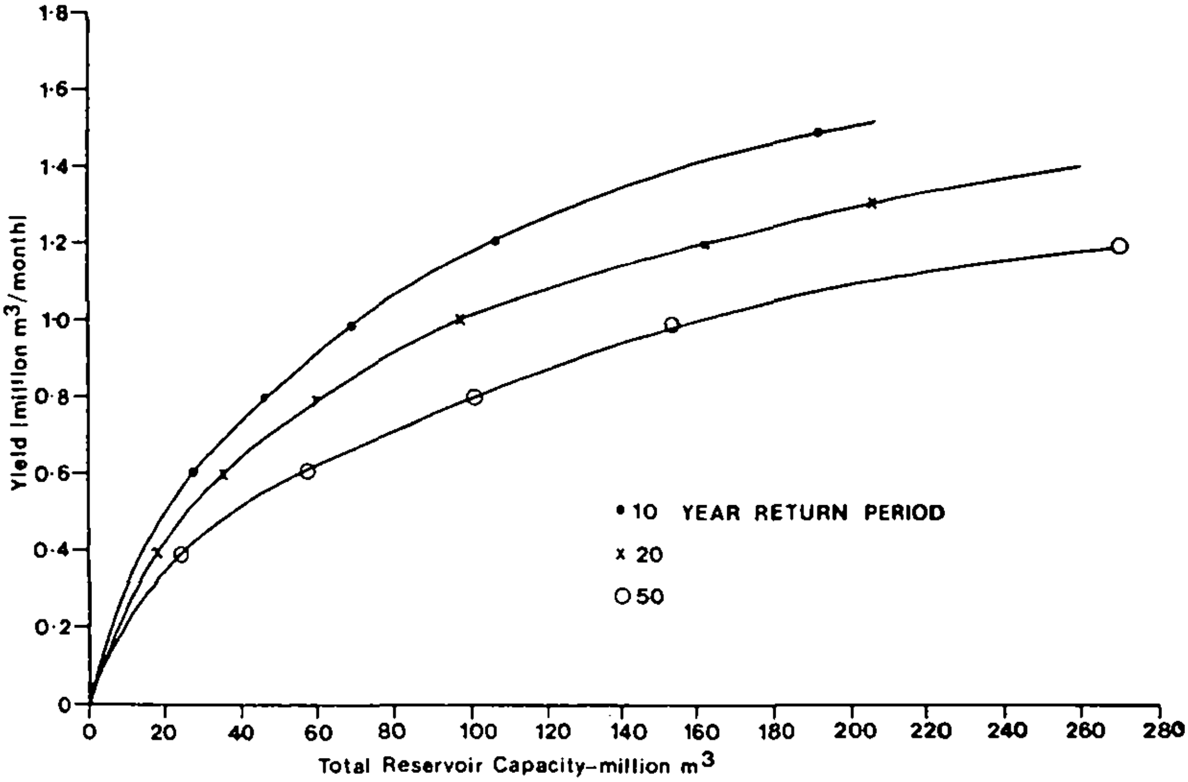


Fig. 21. **Reservoir Capacity Required for 2035 Conditions**

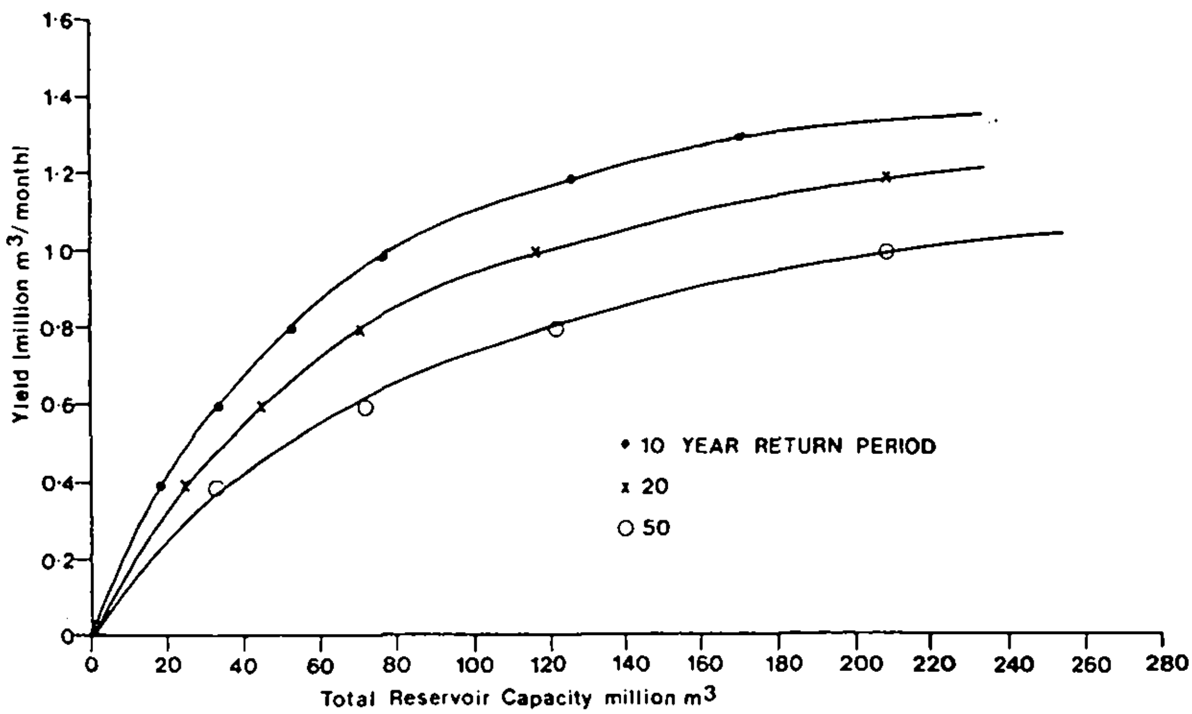


Figure 18 and 19 describe the average, long term behaviour of the reservoir but an approximate estimate of the safe, short term yield can be achieved by transforming the "Reservoir Capacity" axis to a "Current Contents" axis. The storage available at the end of a season can then be used to estimate the safe yield available until the reservoir contents are increased. In this way it is possible to determine either the increased risk of not providing a water supply at the design yield, or the safe yield which may be supplied retaining the original risk during the transition period immediately after raising the dam.

3.3 THE PROBABILITY OF FILLING OF THE RESERVOIR

To complete the picture of the performance of different sized reservoirs it is useful to compare the probability of filling for different capacities. This will also give some insight into the length of time which will have to elapse before the design yield can be met.

The probability of filling can be best estimated by Gould's probability matrix method (Ref 9) in which the reservoir capacity is divided into a number of equal states. A transition matrix is calculated from the available data such that the probability of the contents being in any particular state at the end of the season can be determined from the contents (or state) at the beginning of the season. The matrix is formed by determining the end of year state from any beginning of year state using a simple monthly water balance whereby

Change in Storage = Inflow - Evaporation - Demand

The frequency of occurrence of each end of year state is extracted from these results and collated in the transition matrix. This method can also be used to determine the probability of failure and spill, from any starting state, by simply counting the number of occurrences and expressing the total as a probability.

The analysis can be extended to more than one year by considering the joint probability of starting in a certain state and finishing in a state conditional on that starting state. For example, from Figure 22 the probability of ending the first year in state 2 from starting in state 1 is 0.09. Then the probability of ending the second year also in state 2 from starting the first year in state 1, is

$$0.09 \times 0.10 = 0.009$$

Fig 22. **Theoretical Diagrammatic Transition Matrix**

STARTING STATE

FINISHING STATE

	0	1	2
0	·12	·14	·15
1	·10	·11	·12
2	08	·09	·10

If this is continued the probabilities approach a limit, known as the steady state situation, which is independent of initial conditions. From the steady state likelihood of being in any one state and the probability of filling, from starting in that state, it is possible to determine the total probability of filling for any capacity and yield. Thus the steady state probability of filling was estimated for yields of 0.5 to 1.2 million m^3 per month and capacities of 15 to 250 million m^3. These probabilities are relatively insensitive to yield as they are concerned with the high inflow events so the results have been plotted in Figure 23 to show the range of values for the two extremes of yield. The results range from 5.5% likelihood of filling for a capacity of 250 million m^3 to 53.8% likelihood for 15 million m^3. The percentage probability quantifies the likelihood of filling in any one year.

The Short Term Probability of Filling

The Gould analysis carried out so far describes the long term, steady state likelihood of filling, but the likelihood immediately after raising the dam will be less than this as the contents are bound to be below the original dam level. The Gould method can be adapted to estimate the probability of spill of the reservoir from the contents at the beginning of the season. The results are shown in Figure 24 for a demand of 0.5 million m^3 per month and considering five different capacities (including the projected capacity for the present dam height which is 34.3 million m^3). Thus, from Figure 24, the probability of filling during the wet season can be determined from the end of dry season contents. For the purpose of this analysis the end of the dry season is taken as the 30th September. A more detailed study will be possible, to check the probability of filling of the reservoir, once the height of the proposed increase has been determined.

3.4 RELATIONSHIP BETWEEN RETURN PERIODS AND ACTUAL DROUGHTS

The estimate of a yield for a particular return period of failure is perhaps more easily placed in an historical context if a return period can be estimated for known historic droughts. Ref (10) is a paper concerning South African rainfall, area A of which includes part of the Gaborone catchment. The analysis has been carried out for rainfall records from 1910 to 1972 at 157 stations. This paper picks out three troughs of precipitation (that are synonymous with droughts) identifying the worst periods during recent decades. These are 1928 to 1932, 1948 to 1952 and 1968 to 1972. The decades separating these events exhibited higher than normal rainfall conditions. These drought periods can be readily observed in the rainfall records of Botswana and in the simulation inflows for Gaborone dam. However a period of low flows is also noticeable from 1961 to 65 which causes a more severe shortage than 1948 to 1952. A crude estimation of return period of these droughts has been made by ranking the droughts in order of severity and thus assigning corresponding return periods as the four most severe droughts in 57 years.

RANK	DROUGHT PERIOD	LIKELY RANGE OF RETURN PERIOD (YRS)
	1928 to 1932	> 29
	1968 to 1972	19 - 57
	1961 to 1965	14 - 29
	1948 to 1952	< 19

The extent, and hence damage, to be expected from a 20 year return period drought can thus be approximately deduced from droughts 2 and 3; however capacity and yield of a reservoir can alter the effect of individual droughts and therefore change the ranking; for instance a short severe drought may have a greater effect than a long moderate drought for a small reservoir and low yield whereas the opposite is true for a large reservoir and high yield, thus there is no unique solution for the ranking of droughts.

3.5 CONCLUSIONS AND RECOMMENDATIONS

The 10 year, 20 year and 50 year return period yields can be determined, for any reservoir capacity, from Figures 20 and 21. These results have been summarized, in terms of the amount of raising of the dam, in Tables 17 and 18 and the results plotted in Figures 25 and 26.

The slope of the curves in these figures indicate the gradually decreasing rate of yield available with increased spillway height. Eventually, as the capacity is increased, the available yield will reach a maximum, after which all the increase in capacity will be lost by evaporation. It is not possible to calculate this "ultimate" capacity with the available data because, as the capacity and yield become larger the results become more dependent on the starting conditions of the reservoir operation. Figures 25 and 26 have been drawn to cover the range of capacities which have been determined with

Fig 23. Long Term Probability of Filling

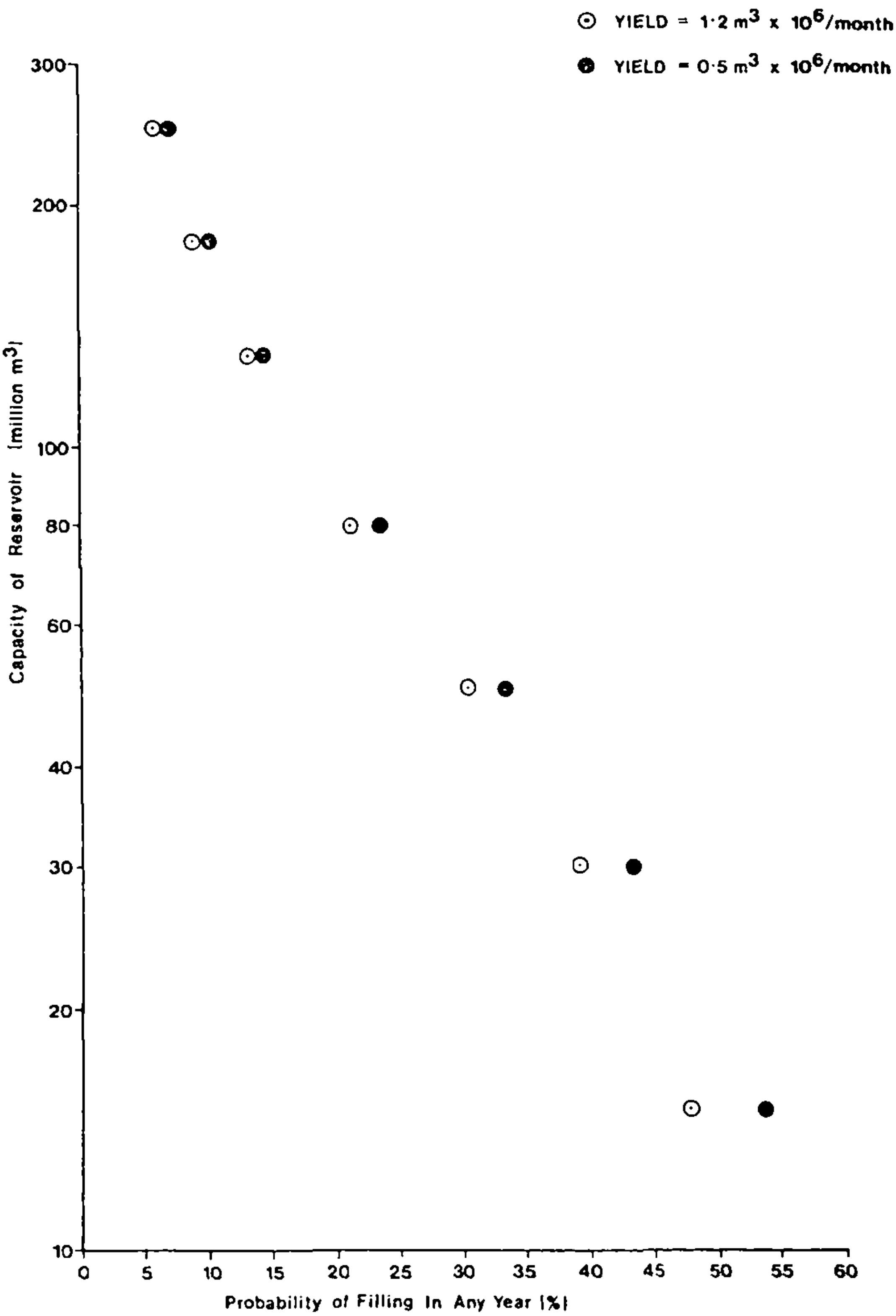


Fig 24. Short Term Probability of Filling

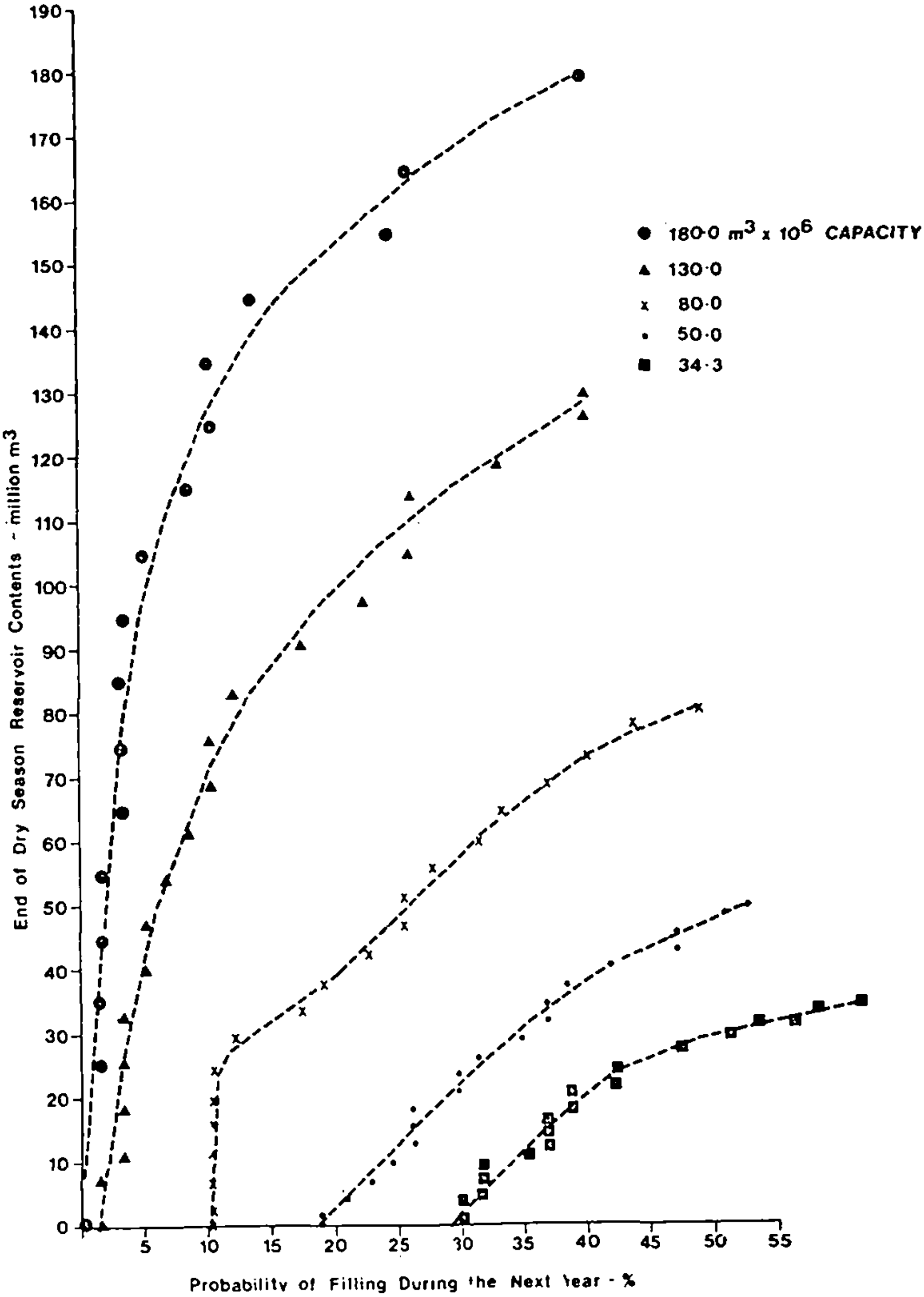


Fig. 25. Yield Available for Increased Height of Spillway (1985)

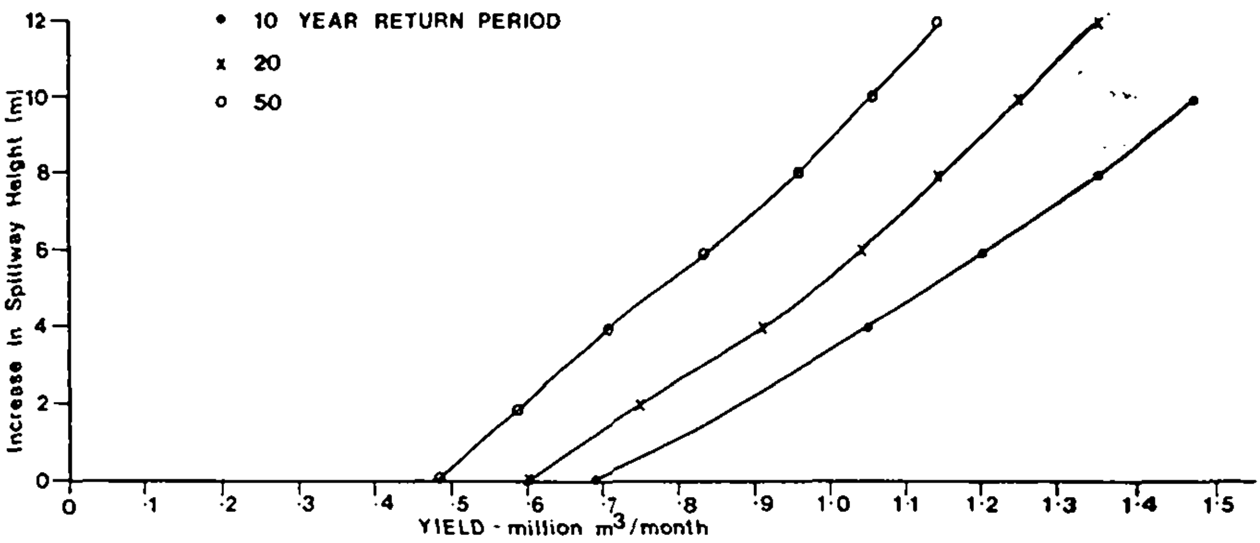
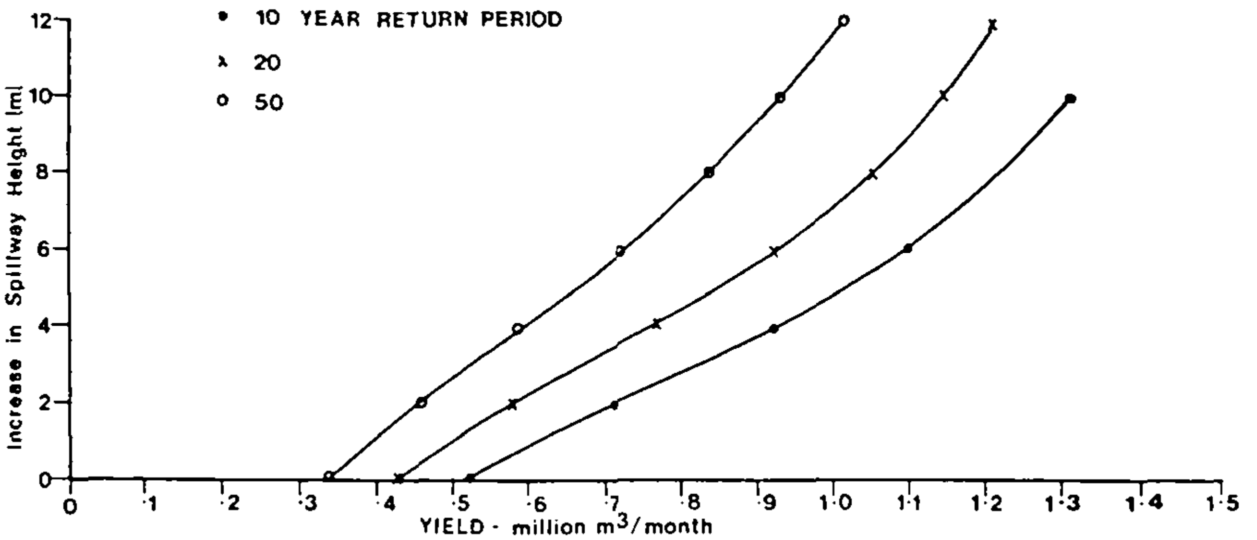


Fig. 26. Yield Available for Increased Height of Spillway (2035)



negligible effect from the starting conditions.

The effect of changing the sedimentation conditions persists throughout the range of capacities chosen as the additional deposits not only affect the size of the dead storage, but also change the shape of the area/capacity curves.

The reservoir yield analysis relies entirely on the inflow data for Gaborone reservoir. Any future study of the reservoir would benefit from gauging the Notwani river to accurately quantify the spill which occurs from the reservoir. Data of this kind would lead to much greater confidence in the results of flood and yield analysis and hence greater reliability of the design. It would also be very useful if up to date evaporation records were available for water resource analyses of this area.

The extension of the inflows to Gaborone Dam was complicated by discrepancies noticed in the daily rainfall values and the monthly summary sheets. Some considerable time had to be spent studying these data and making subjective decisions concerning particular values to be used in the analysis. It is important that the daily records and monthly summaries are corroborative as these records provide the main source of hydrological data in Botswana and their accuracy is heavily relied upon in studies of this type.

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