INSTITUTE of HYDROLOGY

GREATER NAKURU WATER SUPPLY

Londiani Dam

HYDROLOGICAL ANALYSIS

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This report is prepared for

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1. INTRODUCTION

Londiani dam is one of the two dams constituting the first phase of the Greater Nakuru Water Development Plan which will provide the water needed for the southern area of the rift valley as delineated in the draft report (Ref. 1).

The chosen dam site (map reference 0870 0867) is on the river Kipchorian in the 1GC drainage basin and has a catchment area of 136 km^2. Figure 1 shows the location of the dam site in relation to the nearest river gauging station, which is 1GC5 on the Nyando river, and the long term raingauges.

The objectives of this study are to provide estimates of floods for the spillway design and construction works and to calculate the yield available from the dam site for 10, 20 and 50 years return period of failure and three retention levels.

The Londiani dam catchment stretches along the western side of the rift valley as far north as the equator. The altitude varies from 2200 metres at the dam site to over 2600 metres in the upper reaches where much of the land is forested. However, deforestation of these areas has been taking place for many years which may be causing a change in the catchment response to rainfall and also in the rate of soil erosion.

The climate of the Londiani catchment, as for most of the rift valley, is controlled by the Intertropical Convergence Zone (ITCZ). This zone is formed by a series of low pressure areas which are parallel to the equator and moves with the sun north and south of the equator. The "long" and "short" rains of Kenya are associated with the instability caused by the movement of the ITCZ in March to May and October to December respectively. Rainfall occurring between these periods, in July and August, are known as the "continental rains" and result from the development of local anticyclones.

This describes the general pattern of the underlying climate across

Location map

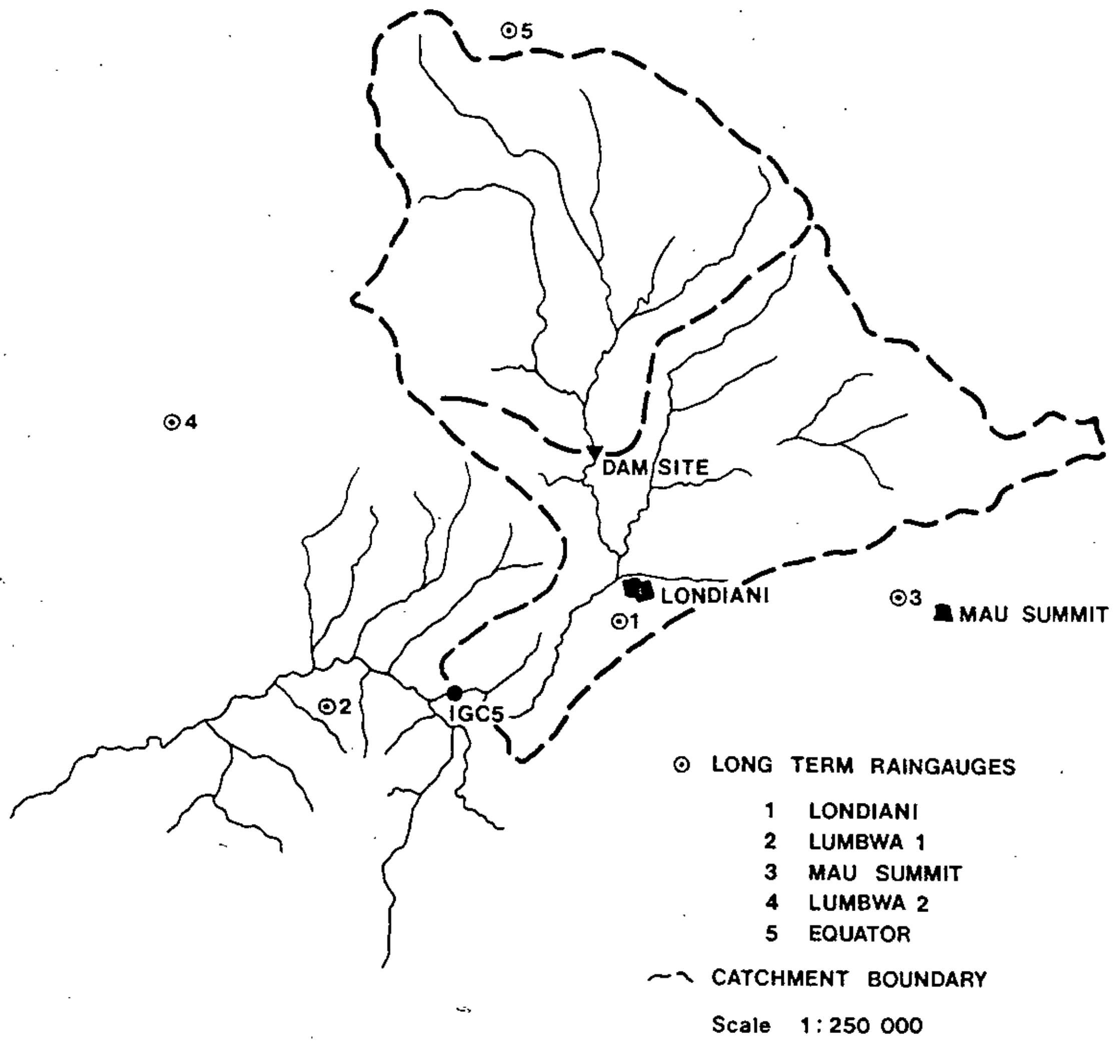


Figure 1

Kenya but the mountainous range of the rift valley tends to break up the effect of the ITCZ pattern and this, compounded with the effect of the large lakes in the valley, produces a very complex climatic region.

1.1 AVAILABLE DATA

Rainfall

Londiani has a mean annual rainfall of about 1200 mm, and most of this occurs between April and August forming a well-defined wet season. The variability of both annual rainfall and its time of occurrence is very high so that an analysis of water resources incorporating rainfall as a mean annual statistic could be very misleading. It is therefore necessary to include historic rainfall data wherever possible to incorporate the inherent variability in the analysis.

The raingauge network is fairly sparce in the northern rift valley but, of the records that do exist some extend back to 1902. There are only three automatic raingauges in the area and the nearest of these to Londiani is Kericho. The other raingauges are read daily and summarized as monthly totals.

The five long term raingauges are listed in Table 1 together with the period of record for which data are available and the weights used to determine catchment rainfalls for the reservoir yield analysis. The weights were simply estimated from Thiesson polygons drawn for the catchments to the damsite and to the gauging station 1GC5.

Evaporation

Potential evaporation estimates have been calculated by Woodhead (Ref. 2) using the Penman equation for all the available data at that time. Potential evaporation is a rather more conservative statistic than rainfall and thus the mean monthly estimate is more applicable for use in a resource analysis than the mean monthly rainfall.

TABLE 1

Catchment Rainfall Analysis - Londiani

Rainfall	Gauge	Period of Record	Weighting for dam site catchment	Weighting for 1GC5 catchment
Londiani	903502	1908-1980	29	130
Lumbwa	903568	1938-1976	10	11
Lumbwa	903520	1905-1980	0	4
Equator	903569	1938-1975	141	141
Mau Summit	903538	1932-1979	0	89
Mean annual rainfall (mm)			1202	1165

The average potential evaporation for the Londiani catchment was calculated from the mean of five stations which are the Equator, Kericho, Koru, Molo and Molo (Pyrethrum Res.) stations. The mean monthly potential evaporation estimates are shown in Table 2 together with the mean monthly rainfall estimates.

Runoff Data

The nearest gauging station to the Londiani dam site is 1GC5 on the Nyando river with a catchment area of 251 km^2 including the Kipchorian river. Records are available from this site for 1964 to 1980 in the form of mean monthly flows which are obtained from staff gauge readings and a rating curve.

The staff gauge is read twice daily and is in a natural section with a rock control downstream. The rating curve is based on discharge measurements carried out during low and medium flows extrapolated to encompass higher flows.

Of the other gauging stations in the area the majority have records of 20 years or less and some of the stations are not rated. Very few stations have had flows calculated since 1976 thus considerably reducing the available data set.

Sedimentation

There is very little available information about the sediment load of Kenyan rivers and how it is changing with land use; however, a large programme of sediment sampling has recently been set up by MOWD including measurements at station 1GC4 on an adjacent catchment to the Nyando. Dunne (Ref. 3) worked on suspended sediment sampling and produced sediment rating curves for 97 stations in Kenya. These are useful for estimation of sediment load when accompanied by a flow duration curve of the station, but the newest rated catchments are in the 1GD and 1GB drainage basins.

Two sediment samples have been collected at station 1GC4, one in May 1980 and one in June. The sediment concentrations measured were

TABLE 2

Mean Monthly Climatological Characteristics

	Evaporation (mm)	Rainfall to Dam Site (mm)	Seasonal Distribution of Runoff %
January	161	40	1.69
February	155	44	1.51
March	170	75	3.38
April	137	157	2.97
May	132	138	6.67
June	120	115	5.40
July	112	154	15.84
August	115	188	28.9
September	133	100	16.79
October	144	56	5.57
November	135	78	8.69
December	148	56	2.59
Total	1662	1201	100.00

approximately 112 and 84 ppm respectively. Neither of these measurements suggest a large sediment concentration but more data will be necessary to come to any conclusions.

A recent study in the Tana river catchment reported erosion rates of about 0.5 mm/year (Ref. 4). This may be used as an indication of possible rates of sedimentation in the Londiani area.

2 FLOOD ANALYSIS

2.1 INTRODUCTION

The objective of this flood analysis is to provide estimates of spillway design and construction floods for the proposed dam sites. The range of return periods for the spillway design floods is 100 to 500 years; for construction floods the range is 5 to 50 years.

A number of methods are available for the estimation of floods of these return periods namely:

1. statistical analysis of peak discharges,
2. statistical analysis of rainfall and then conversion to runoff using a suitable model,

and 3. empirical methods.

To use the first method without excessive extrapolation for estimating high return period floods requires many years of streamflow records. The analysis can be based either on records from a single gauging station or from a number of stations within a similar hydrological region. For the single station the annual maximum flows are abstracted from the records, ranked and then plotted using an assumed theoretical distribution; for the regional analysis, the sample size is increased by pooling the available data together in dimensionless form.

Raingauges are generally more plentiful than river gauging stations and their records longer. Consequently the statistics of extreme rainfall can often be estimated more accurately than flood statistics. The unit hydrograph - losses method uses a simple hydrograph model to convert a chosen design storm to runoff. If adequate data are available a unit hydrograph can be derived from observed data; otherwise a synthetic unit hydrograph is estimated using catchment characteristics such as channel length and slope. Rainfall intensity/duration/frequency relationships are used to construct design storms of the required return periods.

Some of the empirical methods for flood estimation can be applied to a wide range of climates and countries. Others, such as the design method for the United Kingdom described in the Flood Studies Report (FSR) (Ref 5) or for East Africa described in the Transport and Road Research Laboratory Report (TRRL) (Ref 6) relate to more specific regions.

The majority of river flow records in Kenya are for river gauging stations where river stage is observed one or two times a day. Flood statistics are therefore generally based on mean daily flow data rather than instantaneous peak discharges. Moreover since up to 80 per cent of daily rainfall occurs between 1300 hrs and 2200 hours (Ref 7) the flood peaks are rarely observed on medium and small catchments where no automatic recorders are installed. In these cases flood statistics based on mean daily discharges will tend to be underestimated.

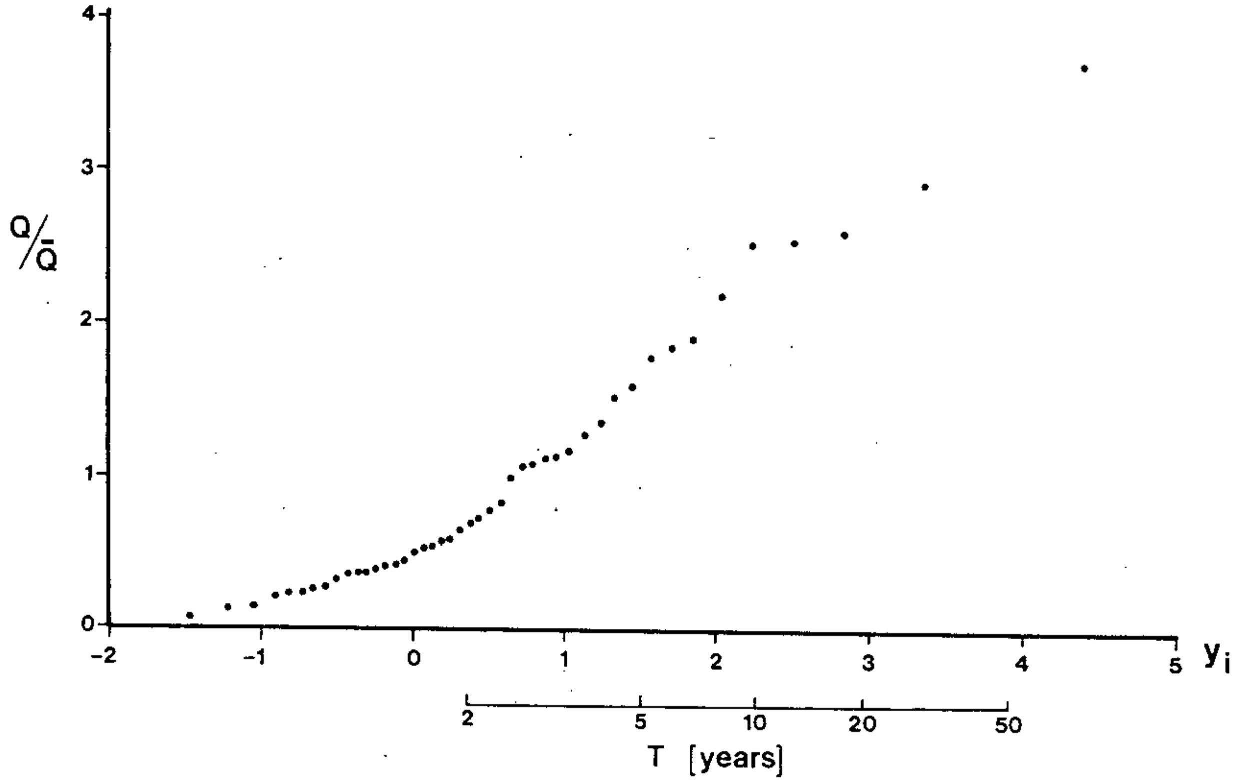
Rainfall data from autographic recorders in Kenya have been analysed and the results published in a convenient form for estimating design storms for given durations and frequencies (Refs8,9). Insufficient short-term rainfall and runoff data are currently available for the dam site catchment to allow derivation of real unit hydrographs in a conventional way.

Consequently it was decided that neither method (1) nor (2) could be used on its own to estimate design floods. On the other hand to rely solely on an empirical method would have meant ignoring the data that do exist. A combination of the three methods has been used here.

First a regional flood frequency curve was constructed using the local data available; these were annual maximum mean daily discharges. Experience from other parts of the world suggests that the dimensionless frequency distributions of instantaneous and mean daily peak discharges will be similar. This similarity is supported by data from river gauging station 2GB1 on the Malewa river located to the south east of Nakuru (Figure 2).

Thus provided an estimate of the mean annual flood ($\bar{Q}$) at the dam site can be made, flood peaks of return periods up to about 50 years can be deduced from the dimensionless frequency curve. The magnitude of $\bar{Q}$ was estimated using the TRRL method.

2GB1: Flood frequency curves



Mean daily discharge

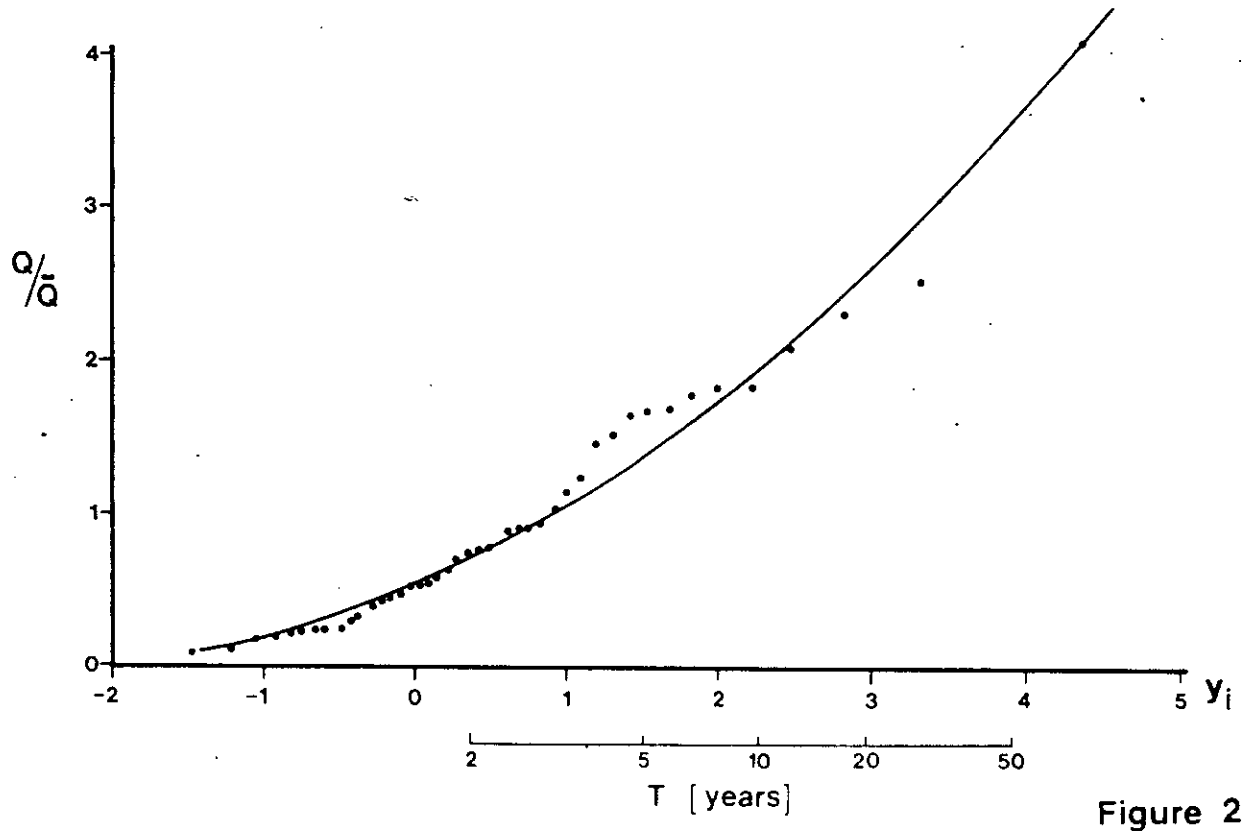


Figure 2

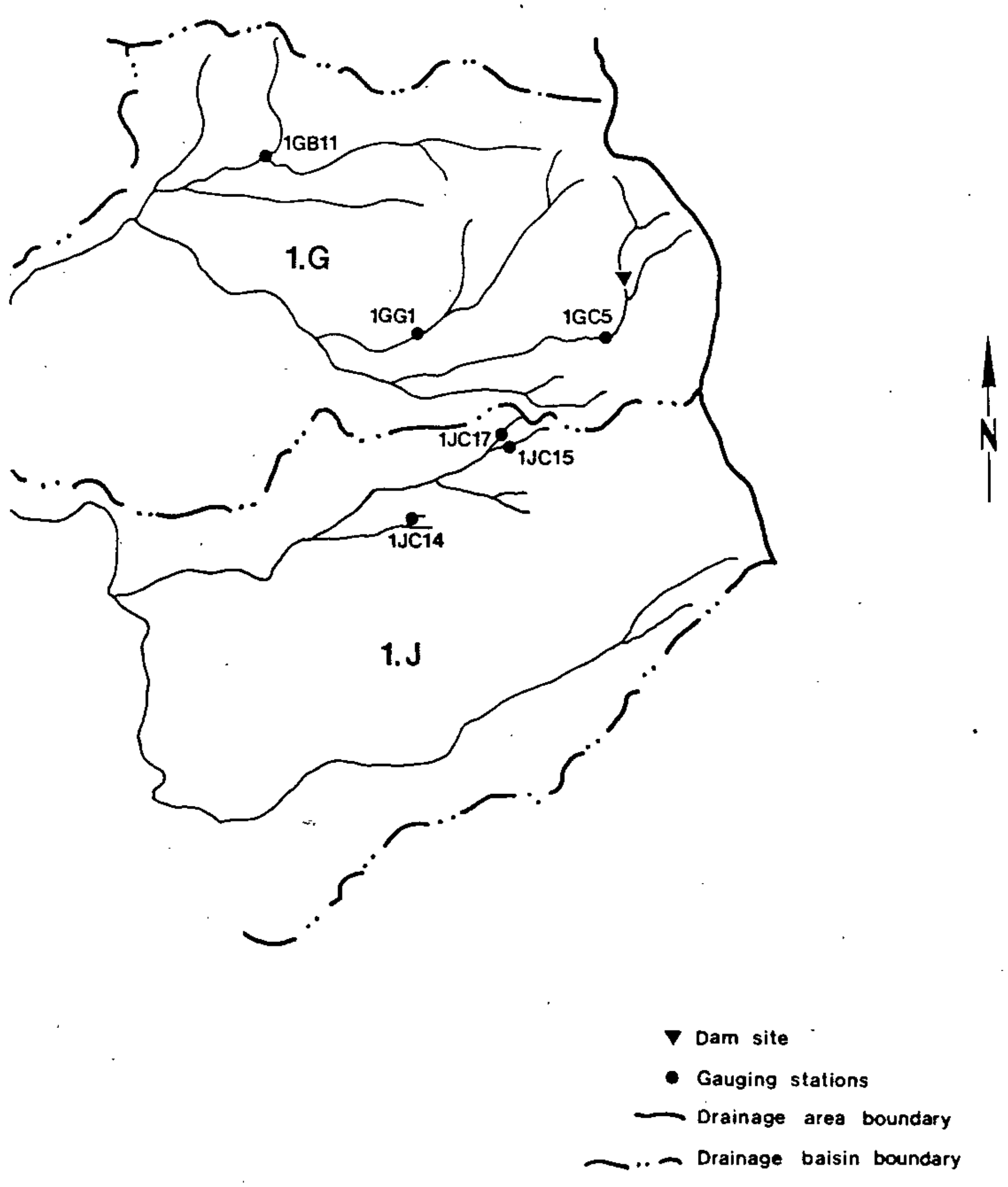
The model parameters of a unit hydrograph losses model were adjusted to ensure that the floods predicted for return periods up to 50 years were consistent with the regional flood frequency curve. The model was then used to calculate higher return period floods for spillway design.

2.2 DATA USED IN FLOOD ANALYSIS

The records of a number of gauging stations in drainage basin 1 were inspected. The data from several stations were excluded either because stage readings were taken only once every two or three days, or because the rating curves appeared to be particularly suspect. For the remaining stations (Figure 3) the annual maximum mean daily flows were extracted from the MOWD files; no chart records were available for the extraction of instantaneous peak discharges.

Rainfall data were taken from two published sources, namely the MOWD rainfall frequency atlas of Kenya (Ref 8) and the TRRL design manual (Ref 9).

Gauging station locations



APPROXIMATE SCALE:- 1:750.000

Figure 3

2.3 STATISTICAL ANALYSIS

A regional flood frequency curve is essentially a frequency distribution of $Q_T/\bar{Q}$, where Q_T is the flood of return period T years and $\bar{Q}$ is the mean annual flood. The relation is assumed to be valid for all catchments within a region, or alternatively to represent the mean of the different relationships for the different catchments in the region.

The curve is constructed from the series of annual maximum floods at the gauging stations shown in Figure 3. Each record was converted into a dimensionless series $Q/\bar{Q}$, and the individual events ranked in ascending order. The plotting position, y_i, that corresponds to the flood of rank i in the series was estimated from the Gringorten formula given by

$$F_i = \frac{i - 0.44}{N + 0.12}$$

and $y_i = -\ln -(\ln F_i)$

where F_i is the plotting position expressed as a probability,

i is the rank of the event,

and N is the number of events in the series.

These floods were then grouped into ranges of y (- 1.5 to 1.0 etc) and the mean values of y and the ratio $Q/\bar{Q}$ calculated for each range. By using these calculated means, it was possible to define the regional curve up to a value of $y = 2.6$. The curve may be tentatively extended further by plotting the three highest individual values of $Q/\bar{Q}$ as being the three highest events taken from a sample population of 75 events, 75 being the total number of events in the pooled record.

The resulting curve is shown in Figure 4.

Pooled flood frequency curve

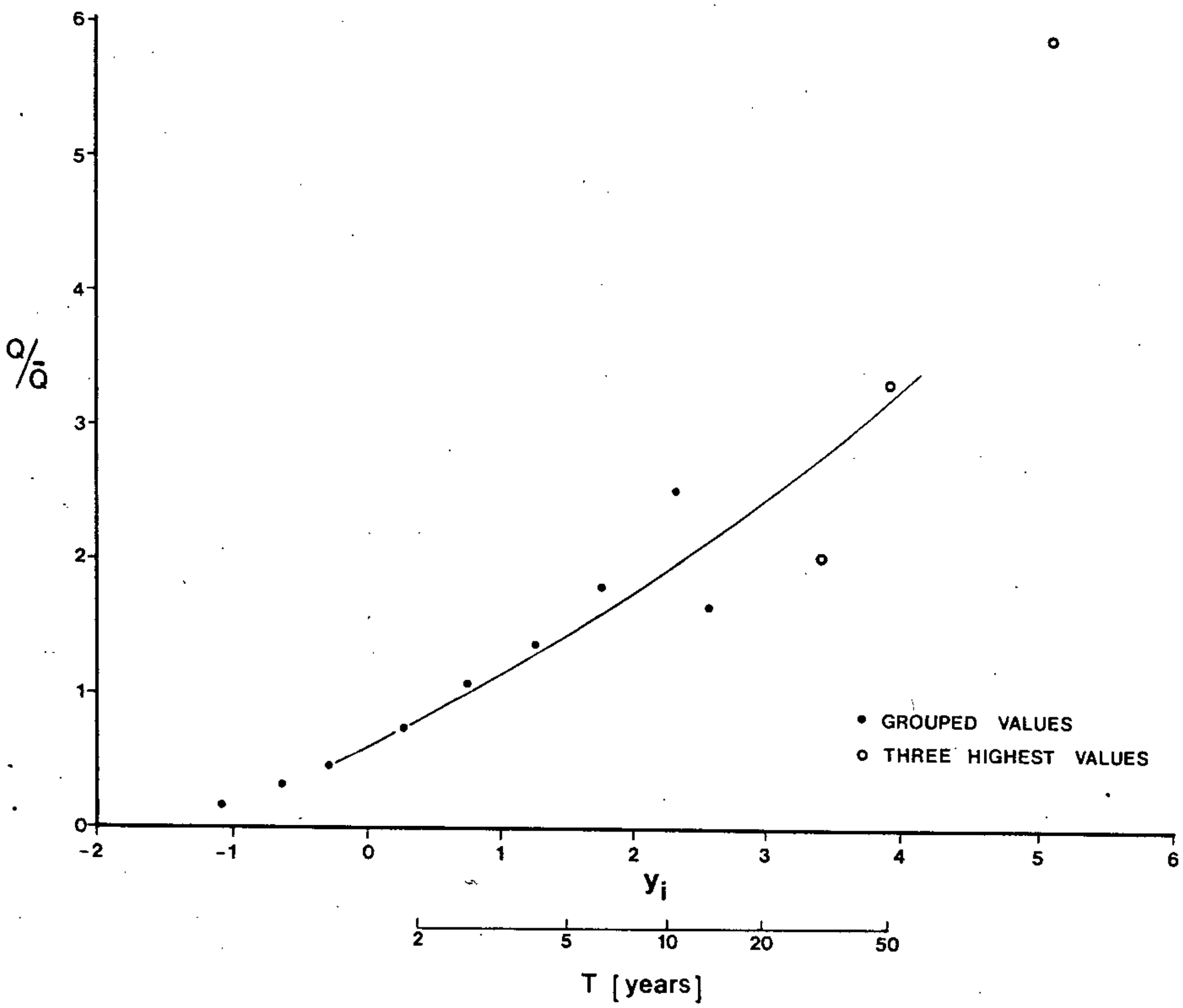


Figure 4

2.4 TRRL METHOD

The TRRL method of flood estimation is a simple technique for estimating design hydrographs for ungauged catchments. In common with the unit hydrograph-losses model, the method consists of converting a given design storm to runoff using an appropriate model. In both methods, it is assumed that a storm of a given return period will cause a flood of the same return period. The actual response of a catchment will depend on the local antecedent conditions and the assumption may not be strictly true, but in the absence of detailed local information it is considered to be reasonable.

The method is described fully in the relevant TRRL Reports (Refs 6, 9) so only a brief summary is given here. The selected design storm is converted to runoff using a simple three parameter model, whose parameter values depend on the catchment's physical and climatological characteristics. The parameters of catchment lag, initial retention and contributing area coefficient were estimated in the light of field visits and from the tables, maps and figures in the Reports.

For this study the TRRL method has been used to estimate the mean annual flood ($\bar{Q}$) at the dam site. Although strictly the return period of the mean annual flood is 2.33 years for the Gumbel distribution, we have assumed that this flood can be reasonably estimated from the 2 year return period rainfall. A summary of the parameters used in the calculation is given in Table 3, and gives a $\bar{Q}$ of 30 m^3/sec.

2.5 UNIT HYDROGRAPH - LOSSES MODEL

The unit hydrograph for a particular catchment defines the response to a unit volume of net or effective rainfall input over a specified time interval. The method relies on two main assumptions of catchment behaviour namely:-

(1) there is a linear relationship between net rainfall and flood discharge; ie twice the net rainfall doubles the flow

(2) the principle of superposition applies; the final flood hydrograph is made up from the direct addition of the ordinates of a series of unit hydrographs scaled and lagged according to the net rainfall hyetograph. This process is called convolution.

SUMMARY OF TRRL METHOD PARAMETERS

TABLE 3

		Source
Catchment area	136 km^2	
Land slope	4.47%	
Channel slope	2.44%	
S1085	14.72 m/km	
2 year daily point rainfall	55 mm	TRRL 623 Figure 1
Areal reduction factor	.83	" Figure 17
Rainfall time (T_p)	.75	TRRL 706 Table 8
Catchment lag (K)	8 hours	" Table 7
Antecedent rainfall zone	Nyanza	" Figure 14
	Dry zone	" Table 3
Catchment wetness factor (C_w)	.75	" Table 5
Standard contributing area coeff (C_s)	.45	" Table 4
Land use factor (C_L)	.50	" Table 6

The process of flood estimation using the unit hydrograph-losses model involves the following steps:

(1) Estimating the shape of the unit hydrograph. Ideally this should be based on recorded flood and rainfall data; in the absence of suitable data, an empirical formula has to be used

(2) Defining a design storm

(3) Estimating the percentage runoff from the design storm

(4) Combining the unit hydrograph with the (net) design storm. A slow response or 'baseflow' component of this hydrograph is added to the flood hydrograph, but this is usually small by comparison with the direct runoff from major floods.

Unit hydrograph estimation

In the absence of continuous flow records and recording rainfall data for catchments in the Londiani region, it was necessary to derive a synthetic unit hydrograph from catchment characteristics. Many empirical formulae have been used to estimate the time to peak, T_p, of a synthetic triangular unit hydrograph. These equations are based on physical catchment characteristics such as streamlength and slope. It is therefore not unreasonable to use this type of physically based equation in this work. An empirical relationship from the FSR based on stream length and slope (Vol I §6.5.4) gives the time to peak (T_p) of the hydrograph as:-

$$Tp = 2.8\left[\frac{L}{\sqrt{S}}\right]^{0.47} \quad \text{hours}$$

where L is the mainstream length,

and S is the slope of the mainstream measured between 10 per cent and 85 per cent of L from the mouth of the catchment in m/km.

The shape of the unit hydrograph is defined by a triangle whose time base (T_B) and peak discharge (Q_p) are defined by:-

$$T_B = 2.52\ Tp$$

$$Q_p = \frac{220}{T_p} \quad m^3/s/100\ km^2$$

Using the catchment characteristic data summarised in Table 3 the following values are obtained

T_p 6 hours

T_B 15 hours

Q_p 36 $m^3/s/100\ km^2$

Note that these figures have been rounded.

Design storm duration

The FSR (Vol I § 6.7.6) recommends the following equation for the duration of the design storm:

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

where SAAR is the catchment average annual rainfall. The choice of storm duration is not particularly critical for the calculation of flood peak, and we consider that the use of this equation is reasonable.

Design storm depth

Intensity - duration - frequency curves and maps have been prepared for a number of rainfall stations in Kenya (Ref 8). Using the curves and maps together, the 24 hour rainfalls for the Londiani catchment were estimated for return periods of 5 to 100 years. The 200 and 500 year return period rainfalls were estimated by extrapolation of the graph in Figure 5. Lumb's work (Ref 10) was used to estimate the probable maximum precipitation (PMP).

The 5 year 24-hour rainfall-intensity-duration frequency curves was used to construct the profile of the design storm. A nested profile was adopted such that for all durations the rainfall intensities of the same return period occurred within the same storm. The 5 year storm of 13 hours duration was therefore composed of the 1 in 5 year 1 hour fall in the centre of the 1 in 5 year 3 hour fall etc. Design storms of higher return periods were based on an identical profile because no other relevant data were available.

Rainfall growth curve

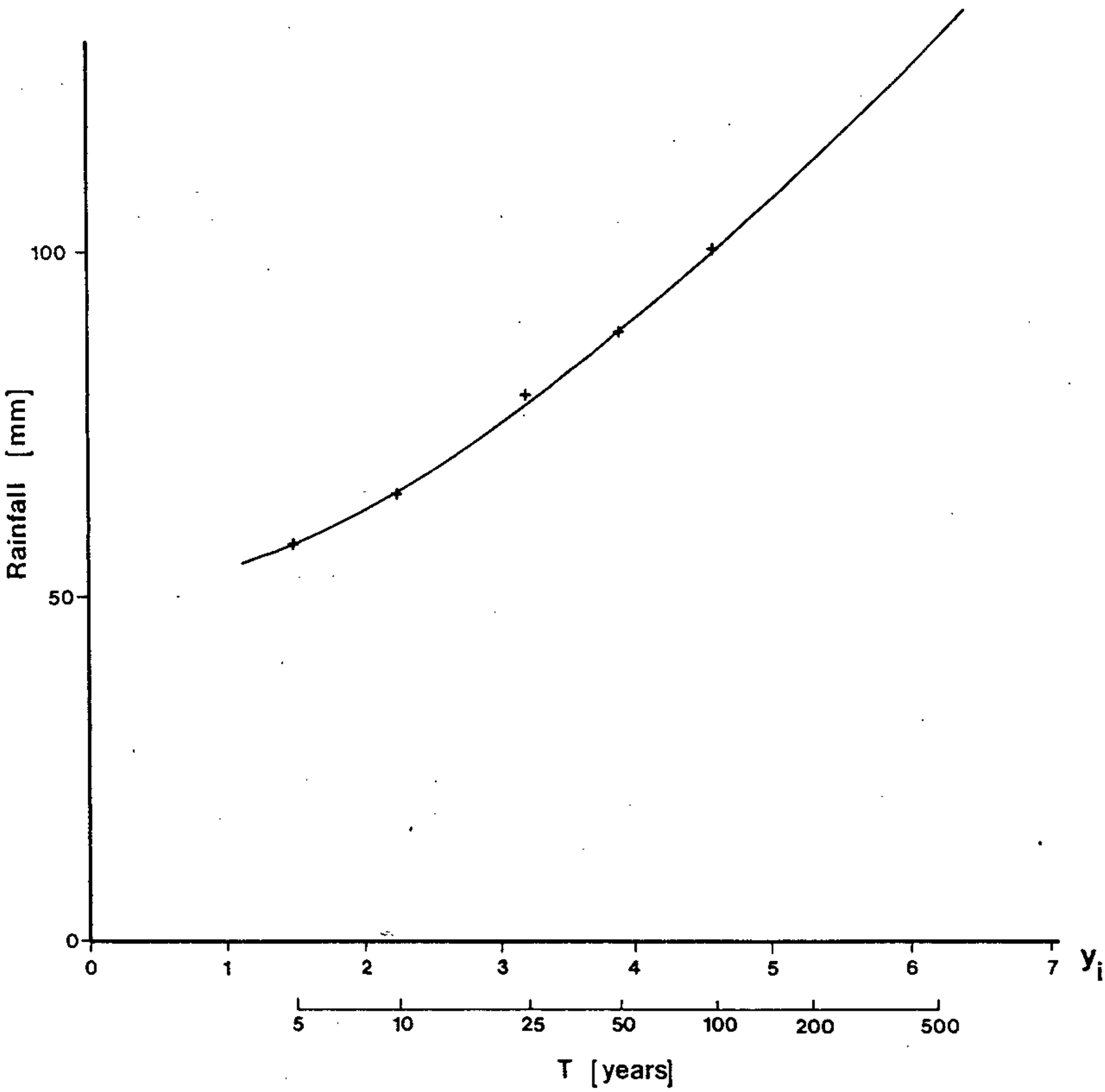


Figure 5

Although the average intensity over the total storm duration has the required return period, the nested profile will tend to create a larger flood because of its peaky nature. However it is preferable to use the local rainfall data in this conservative fashion rather than adopt other, less peaky profiles, such as those described in the FSR which are strictly valid only for the United Kingdom.

Areal reduction factor

The storm profiles derived so far apply to point rainfalls. An areal reduction factor (ARF) has to be used to take account of the fact that point rainfall intensities are higher than those occurring with the same exceedance probability over larger areas. ARF's have been calculated by the TRRL (Ref 9), and in the absence of other data, it has been assumed that for this basin an ARF of 0.83 is valid for design storms of all return periods. The 1 in 5 year areal profile for the dam catchment is shown in Figure 6.

Catchment wetness index

An indication of how wet the catchment is likely to be before a flood event is given by the catchment wetness index (CWI). This index is a combination of soil moisture deficit (SMD), and a 5 day antecedent precipitation index (API_5), defined by

$$CWI = + API_5 - SMD$$

For flood design it has been assumed that the SMD is zero, a reasonable assumption for the wet season.

If D is the duration of the design storm, then API_5 has been calculated from a storm of duration 5D; the design storm being nested at the centre of the longer storm. It is assumed that half the difference between the longer and design duration storms fell uniformly in the 2D hours prior to the design storm. For durations other than 24 hours a conversion equation from the FSR is used (Vol I § 6.8.3).

1 in 5 year rainfall profile

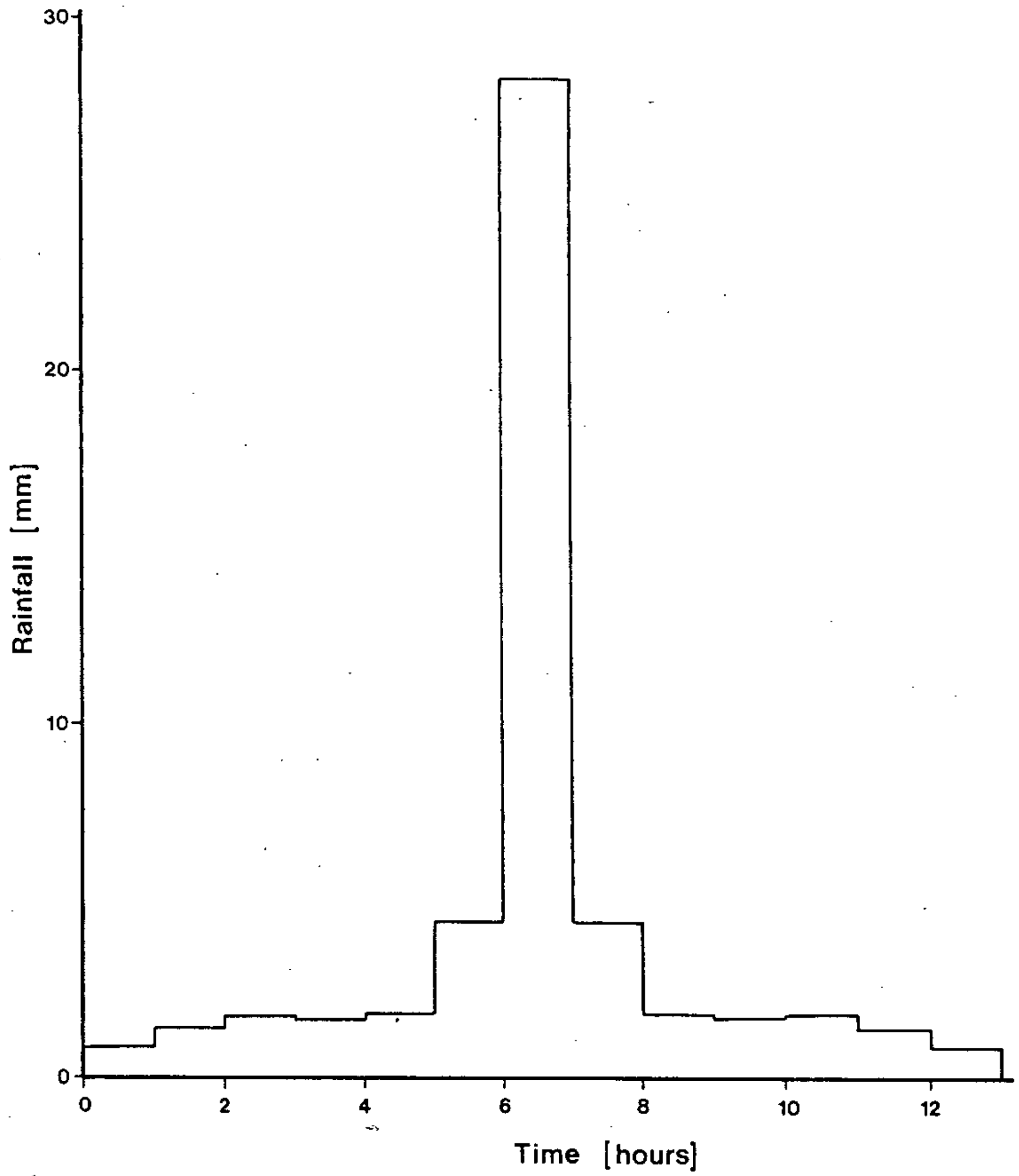


Figure 6

Baseflow

The convolution of the unit hydrograph with the net rainfall profile gives the rapid response component of the total hydrograph; the other component is the slow response or baseflow component. However baseflow is only a small proportion of the flood hydrograph and its value is therefore not critical to the estimate of the peak discharge.

The FSR gives an equation for the slow response component (Vol I § 6.5.11)

$$\text{Baseflow} = 0.000326(\text{CWI} + 0.00074\ \text{RSMD} + 0.003)$$

where RSMD is the net 1 day rainfall of 5 year return period.

Percentage runoff

There were no data available in this study from which an entirely objective assessment could be made of how much of the gross rainfall would be effective in producing flood runoff. For the United Kingdom the FSR proposed equations for percentage runoff composed of three components related to the physical characteristics of the catchment, its initial wetness and the size of the rainstorm. FSR type equations have also been successfully used in other parts of the world.

Initially these equations were used to estimate percentage runoff from local data. However the unit hydrograph model predictions based on these values, for floods with return periods up to 100 years, did not reproduce the steepness of the observed flood frequency curve shown in Figure 4. Consequently the estimates of percentage runoff were adjusted subjectively until the model predictions fitted the observed data more closely.

The model parameters finally used are summarised in Table 4; the model predictions and flood frequency curve are compared in Figure 7. The 100 year flood calculated using the TRRL method is 69 m^3/sec,

DESIGN FLOOD PARAMETERS

TABLE 4

Return Period (years)	Rainfall (mm)	Percentage Runoff (%)	Volume ($m^3 x 10^6$)	Q_{max} (m^3/s)
5	52.2	17.5	1.78	43.1
10	58.8	20.0	2.14	54.0
25	71.8	22.5	2.75	72.3
50	80.6	25.0	3.30	88.8
100	91.5	30.0	3.98	110.0

Unit hydrograph model: Flood predictions

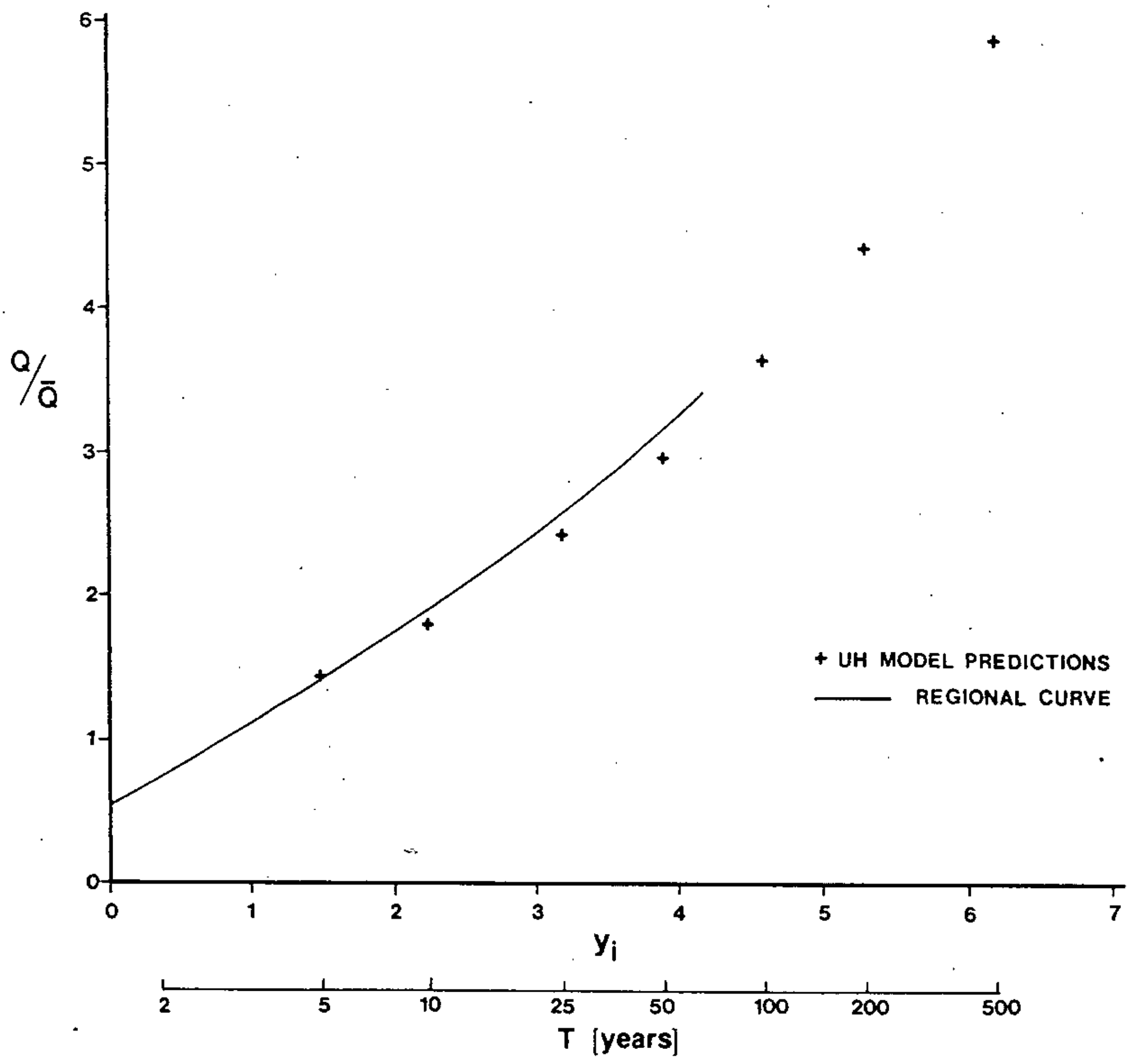


Figure 7

2.6 DESIGN FLOOD ESTIMATES

Construction floods

We recommend the peak discharge of construction floods, with return periods up to 50 years, should be estimated from the pooled flood frequency curve shown in Figure 4, and the estimate of $\bar{Q} = 30\ m^3/sec$ calculated using the TRRL method. The shape of the flood hydrograph can be a simple triangular unit hydrograph where the duration of the flood is 15 hours and the time to peak is 6 hours.

Spillway design floods

Estimates of spillway design floods are given in Tables 5 to 7 and in Figure 8. These estimates were made using the unit hydrograph losses model described above and assumed values of percentage runoff. These estimates are based on a number of assumptions, which we believe are consistent with our understanding of the hydrology of the Londiani region based on information presently available.

These assumptions, and in particular the estimate of time to peak and percentage runoff, could and should be verified by detailed examination of rainfall and flow records from instruments installed on the catchment specificially for this purpose.

Greater Nakuru Water Supply : Londiani Dam Design Floods TABLE 5

200 Year Flood

Area (Sq.Km.)	136.00
Data interval (Hr)	1.00
Design duration (Hr)	13.00
Total rain (mm)	102.46
Percentage runoff	30.00
Base flow (cumecs per sq.km)	.04083

Triangular unit hydrograph computed from Tp= 6.0

Convolution of unit hydrograph and net rain profile

Time	Total Rain mm	Net Rain mm	Unit Hydrograph ordinate	Total Hydrograph cumecs	
.00	1.71	.51	.00	5.55	
1.00	2.81	.84	6.11	5.98	
2.00	3.29	.99	12.22	7.10	
3.00	3.25	.97	18.33	9.05	
4.00	3.54	1.06	24.44	11.81	
5.00	8.95	2.69	30.56	15.45	
6.00	55.35	16.61	36.67	21.32	
7.00	8.95	2.69	32.67	40.29	
8.00	3.54	1.06	28.68	60.34	
9.00	3.25	.97	24.68	79.91	
10.00	3.29	.99	20.69	98.94	
11.00	2.81	.84	16.70	117.35	
12.00	1.71	.51	12.70	132.76	-Peak-
13.00			8.71	125.77	
14.00			4.71	115.09	
15.00			.72	102.96	
16.00				89.71	
17.00				75.52	
18.00				60.71	
19.00				45.72	
20.00				31.30	
21.00				18.17	
22.00				12.71	
23.00				10.07	
24.00				8.16	
25.00				6.80	
26.00				5.96	
27.00				5.60	

TOTAL FLOOD VOLUME (MILLION M3) 4.75

500 Year Flood

Area (Sq.Km.)	136.00
Data interval (hr)	1.00
Design duration (Hr)	13.00
Total rain (mm)	118.95
Percentage runoff	35.00
Base flow (cumecs per sq.km)	.04139

Triangular unit hydrograph computed from Tp= 6.0

Convolution of unit hydrograph and net rain profile

Time	Total Rain mm	Net Rain mm	Unit Hydrograph ordinate	Total Hydrograph cumecs	
.00	1.98	.69	.00	5.63	
1.00	3.26	1.14	6.11	6.21	
2.00	3.83	1.34	12.22	7.73	
3.00	3.77	1.32	18.33	10.37	
4.00	4.11	1.44	24.44	14.10	
5.00	10.39	3.64	30.56	19.03	
6.00	64.26	22.49	36.67	26.99	
7.00	10.39	3.64	32.67	52.68	
8.00	4.11	1.44	28.68	79.83	
9.00	3.77	1.32	24.68	106.33	
10.00	3.83	1.34	20.69	132.12	
11.00	3.26	1.14	16.70	157.04	
12.00	1.98	.69	12.70	177.92	-Peak-
13.00			8.71	168.45	
14.00			4.71	153.99	
15.00			.72	137.55	
16.00				119.61	
17.00				100.40	
18.00				80.33	
19.00				60.03	
20.00				40.50	
21.00				22.72	
22.00				15.33	
23.00				11.75	
24.00				9.17	
25.00				7.31	
26.00				6.19	
27.00				5.70	

TOTAL FLOOD VOLUME (MILLION M3) 6.25

GREATER NAKURU WATER SUPPLY : LONDIANI DAM DESIGN FLOODS TABLE 7

PROBABLE MAXIMUM FLOOD

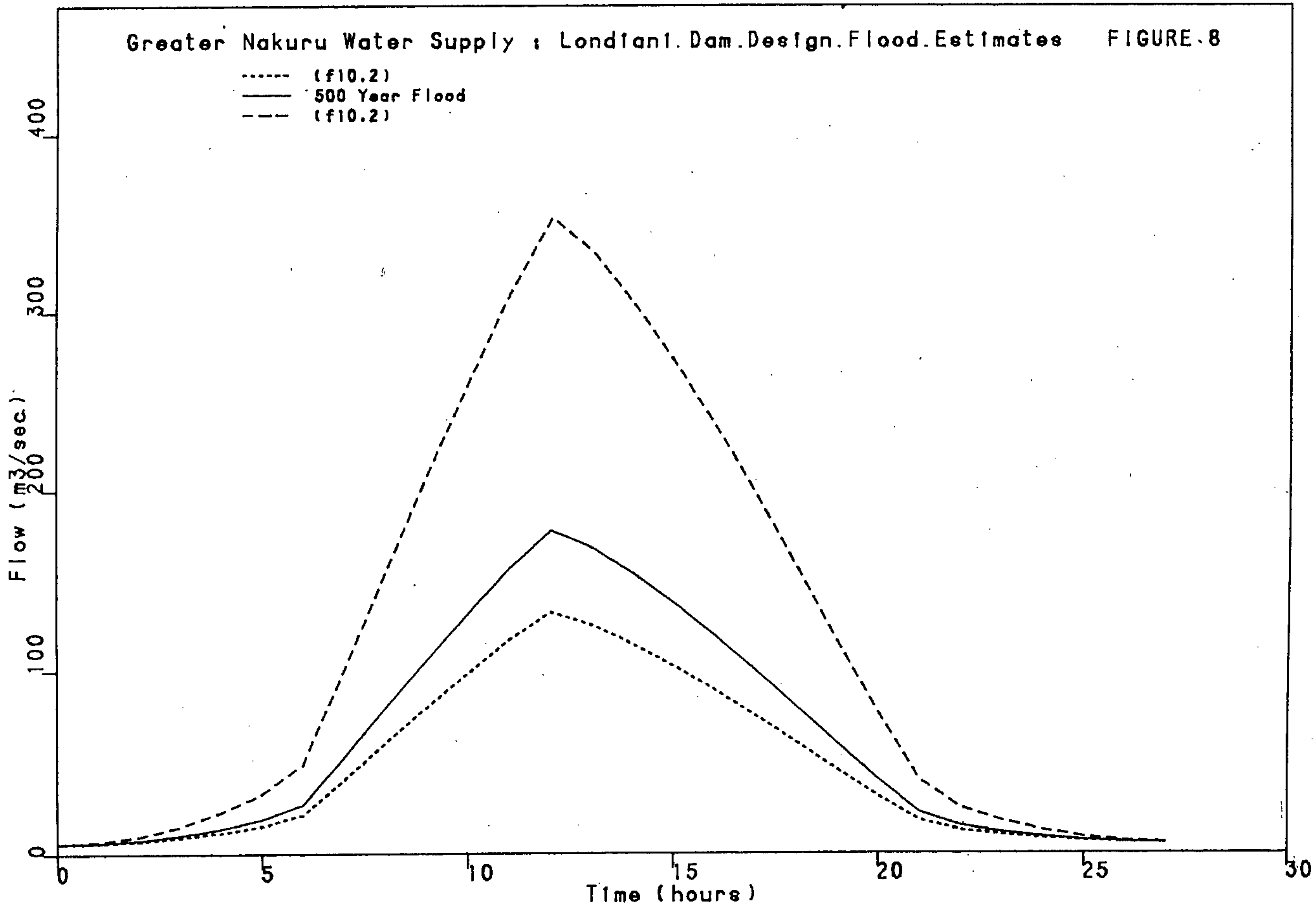
AREA (SQ.KM.)	136.00
DATA INTERVAL (HR)	1.00
DESIGN DURATION (HR)	13.00
TOTAL RAIN (MM)	186.92
PERCENTAGE RUNOFF	45.00
BASE FLOW (CUMECS PER SQ.KM)	.04139

TRIANGULAR UNIT HYDROGRAPH COMPUTED FROM TP= 6.0

CONVOLUTION OF UNIT HYDROGRAPH AND NET RAIN PROFILE

TIME	TOTAL RAIN MM	NET RAIN MM	UNIT HYDROGRAPH ORDINATE	TOTAL HYDROGRAPH CUMECS	
.00	3.11	1.40	.00	5.63	
1.00	5.13	2.31	6.11	6.79	
2.00	6.01	2.70	12.22	9.87	
3.00	5.93	2.67	18.33	15.20	
4.00	6.46	2.91	24.44	22.75	
5.00	16.34	7.35	30.56	32.71	
6.00	100.98	45.44	36.67	48.78	
7.00	16.34	7.35	32.67	100.70	
8.00	6.46	2.91	28.68	155.55	
9.00	5.93	2.67	24.68	209.10	
10.00	6.01	2.70	20.69	261.20	
11.00	5.13	2.31	16.70	311.55	
12.00	3.11	1.40	12.70	353.72	-PEAK-
13.00			8.71	334.60	
14.00			4.71	305.38	
15.00			.72	272.17	
16.00				235.92	
17.00				197.11	
18.00				156.56	
19.00				115.54	
20.00				76.08	
21.00				40.17	
22.00				25.22	
23.00				17.99	
24.00				12.78	
25.00				9.03	
26.00				6.75	
27.00				5.77	

TOTAL FLOOD VOLUME (MILLION M3) 12.04


Greater Nakuru Water Supply : Londiani.Dam.Design.Flood.Estimates
FIGURE.8
(f10.2)
500 Year Flood
(f10.2)
Flow (m3/sec)
0
100
200
300
400
Time (hours)
0
5
10
15
20
25
30

3. RESERVOIR YIELD ANALYSIS

For this report we are required to provide estimates of the firm yield of the reservoir at the Londiani dam site for risks of failure of 1 in 10, 20 and 50 years and retention levels of 2320, 2325 and 2330 m. We have assumed that a yield with a return period of failure of N years is defined as the yield which can be supplied from the reservoir with a failure, of unspecified duration, occurring, on average, once every N years.

Reservoir yield analysis relies on a series of river flows at the dam site. The nearest gauging station set up by MOWD is 1GC5 on the Nyando; however, Sir Alexander Gibb and Partners (Africa) have built a small structure at the dam site to enable flow readings of the river throughout this hydrological year. Readings were started in April 1981 and because the site has not yet been rated, these data have not been used for the present analysis.

The flow recorded at 1GC5 is approximately 14 km downstream from the dam site. The response to rainfall at this station should be similar to that at the dam site as the catchments have similar climate, topography and land use.

To transfer the flow data from 1GC5 to the site we considered corresponding catchment areas and mean annual rainfall.

The catchment rainfalls were determined using Theissen polygon weighted means of the five long term stations in the area as shown in Table 1. From these there is no evidence to suggest that the rainfall in the dam site catchment is significantly different from that in the catchment to 1GC5.

The runoff for the dam site can thus be best estimated using a simple catchment area ratio to transform the flow data from 1GC5.

3.1 EXTENSION OF RUNOFF RECORD

Thus we have a flow series of 16 years of monthly data (some of which are incomplete). This is not sufficient to define adequately the 50 year return period yield without extensive extrapolation of results and, as a consequence, allow confidence in these results. As the rainfall data span 76 years it should be possible to obtain a more reliable result by extending the runoff sequence using the rainfall data.

A rainfall-runoff relationship can be either conceptually or statistically based. A conceptual model would be difficult to fit and would lead to imprecise results in this case as adequate data, such as soil moisture content and infiltration rates, are not available to describe the process of the transition between rainfall and runoff.

A statistical model, in the form of a linear regression, was used instead to relate the runoff to rainfall on an annual basis. We believe that a monthly relationship would be much more tentative than an annual relationship with much larger inherent error.

The regression was carried out using logarithms of both the rainfall and runoff series as this removes the emphasis from flood flows which would, otherwise, tend to dominate the fitting procedure. This is particularly important for reservoir yield analysis as the accurate prediction of low flows is more pertinent to the analysis than the floods. The logarithmic transformation also ensures that there are no negative predicted flows. This would not necessarily be the case with a regression carried out on natural flows.

The data are plotted in Figure 9 together with the line of best fit for the 13 years included in the analysis. The equation describing the regression line is

$$\mathrm{LOG(RUNOFF)} = -11.041 + 4.291\ \mathrm{LOG(RAINFALL)}$$

with a correlation coefficient of 78 per cent.

Line of best fit between annual catchment rainfall and runoff for the dam site

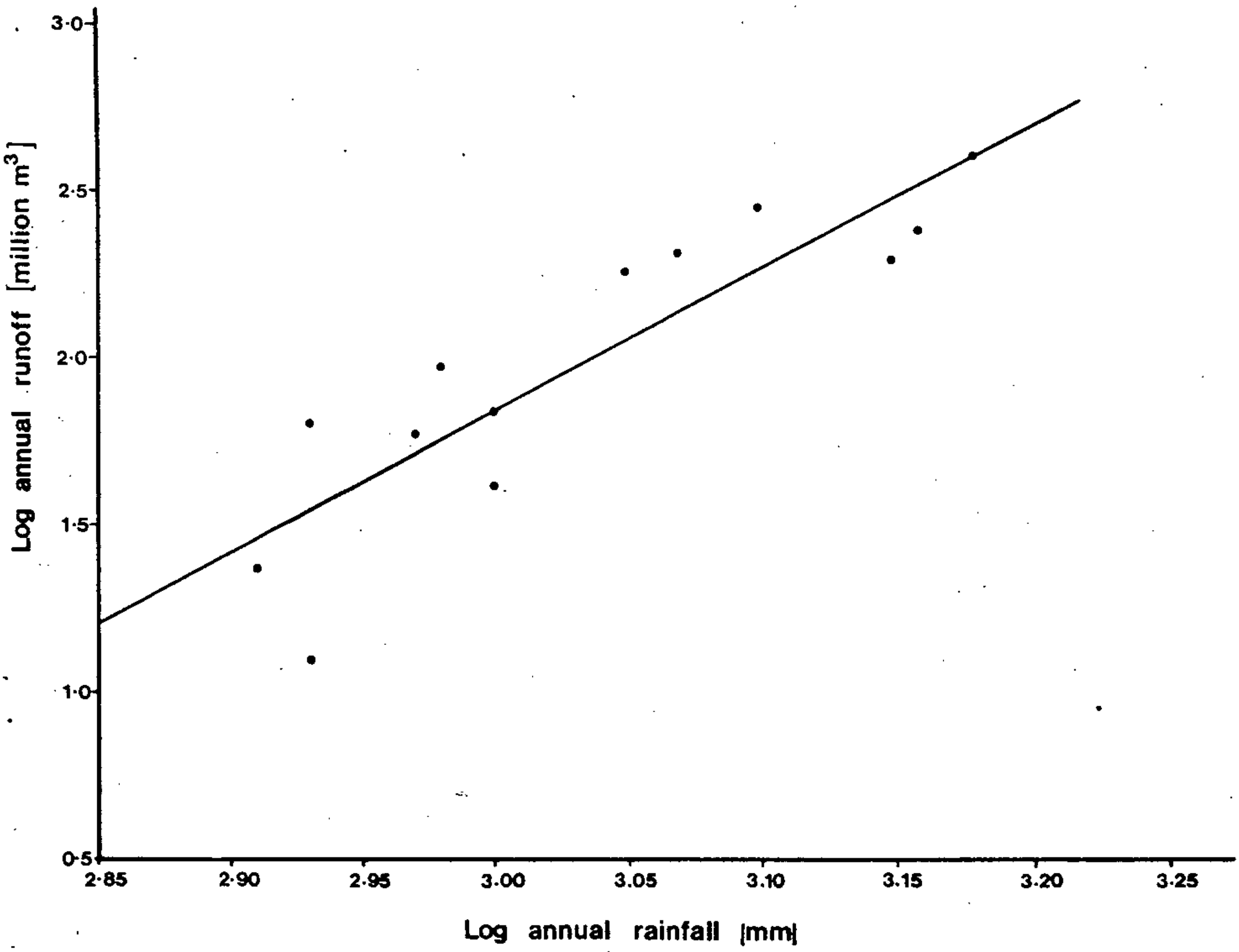


Figure 9

This equation is extremely sensitive to rainfall and could predict unreasonable results if it is used outside the range of fit of the regression; however, only 3 years of the 76 years of record fall outside the range of fit and these 3 fall only just outside, therefore the equation is adequate to describe a relationship between rainfall and runoff. Data from 1968 and 1979 were initially included in this analysis but were later discarded as obvious outliers.

The Predictive Mode

This simple regression equation could be used to predict runoff values for the years when we have only rainfall data but the variance of the synthetic series would not be representative of the actual runoff series. Using this equation assumes that the rainfall-runoff relationship is perfectly described by the line whereas there is, in fact, a scatter of points about the line in Figure 9. We must include a stochastic element into the prediction equation and to do this a normal random variable, of mean 0 and standard deviation 1, is scaled to represent the scatter of the regression by multiplying it by the standard deviation of the residuals.

The equation used for the prediction of annual runoff from annual rainfall values is

$$\mathrm{LOG(RUNOFF)} = -11.041 + 4.291\ \mathrm{LOG(RAINFALL)} + 0.297E$$

Thus a total series of 76 years of annual runoff was predicted for site 1GC5 and used to estimate the flows at the dam site.

The seasonal distribution of runoff was determined for the data from 1GC5 expressing the mean monthly flow as a percentage of the mean annual flow. This distribution was imposed on the 76 years of annual data to produce a 76 year series of monthly flows for the dam site.

The effect of the inclusion of a stochastic element in the prediction equation is that there is no unique solution for the synthetic series. An infinite number of series can be produced merely by altering the stochastic element, although these series will be highly correlated. If just one of these series is chosen at random and used for the reservoir yield analysis, the results might be biased. To guard against this we have predicted 9 separate series of inflows and the storage yield analysis is carried out using each series. The results are collated and the behaviour of the reservoir is described by the mean of the 9 series.

3.2 RESERVOIR STORAGE YIELD ANALYSIS

The storage yield analysis is required to provide results of the yield available, for 10, 20 and 50 year return periods of failure and for retention levels of 2320, 2325 and 2330 metres. There are many methods currently used for storage yield analysis but we believe that the most reliable are based on a reservoir behaviour analysis. The inflows are routed through the reservoir, on a monthly time base, and a water balance is carried out incorporating reservoir yield, spill, inflow, rainfall and evaporation. The evaporation used is shown in Table 2 and the catchment rainfall in Table 8. This will describe the behaviour of the reservoir under the conditions of the synthetic flow sequence. It only remains to quantify the return period of failure attributable to the yield.

We have chosen the Gould Probability Matrix method (Ref. 11) to assign a value to the return period of failure for a particular yield and a particular reservoir size.

The Gould method is described in the appendix but briefly it divides the reservoir storage into N states of equal storage and uses the reservoir routing procedure to determine the probability of ending a year in any state conditional on the starting state. The probability of failure from starting in any state is also determined and this is combined with the steady state probability of being in any state to give the total probability of failure.

The Gould method relies on the assumption that there is no serial

RAINFALL FOR LONDIANI CATCHMENT (MM) TABLE 8

YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
1905	114.3	11.7	225.0	189.0	166.9	31.7	114.8	98.6	94.5	106.9	40.1	145.8
1906	.0	140.0	213.4	174.0	294.6	166.9	304.0	150.6	106.4	181.4	25.4	64.5
1907	83.8	116.8	.0	584.2	429.3	373.4	273.0	109.5	73.2	49.3	58.9	20.8
1908	20.8	108.6	80.3	203.9	109.5	128.2	230.0	322.1	126.0	35.2	75.0	15.1
1909	24.2	.7	11.8	268.4	84.0	113.3	135.8	291.3	115.8	25.9	28.2	64.3
1910	9.3	.0	106.7	111.9	42.0	130.8	180.2	237.8	170.0	36.5	9.1	29.2
1911	.0	12.3	101.4	177.2	70.8	104.0	35.4	128.6	27.5	25.5	116.7	3.2
1912	4.2	160.4	66.2	156.3	64.3	172.8	205.7	125.5	152.3	25.6	69.7	19.8
1913	1.0	82.6	88.4	125.5	124.5	243.9	124.8	51.0	12.4	32.3	61.6	71.7
1914	57.3	66.6	95.2	98.0	181.9	54.1	138.5	208.6	92.8	27.5	90.9	2.1
1915	15.5	11.7	162.8	139.9	79.0	151.6	32.9	78.8	55.6	54.1	52.3	50.0
1916	72.6	22.9	35.3	144.5	177.5	185.9	54.5	145.2	203.5	150.2	63.0	48.4
1917	36.8	28.2	48.9	205.6	145.9	148.5	90.0	144.3	203.5	111.2	25.4	.0
1918	11.0	3.2	.1	82.0	89.1	38.1	74.5	158.2	6.8	15.1	9.2	10.9
1919	13.9	121.6	96.5	155.6	77.8	43.3	130.0	117.2	77.3	10.2	45.5	2.0
1920	31.1	2.2	138.9	196.7	64.1	62.5	109.4	99.7	38.2	90.3	83.5	30.4
1921	.3	62.7	12.1	7.6	113.4	195.1	212.7	174.8	44.6	42.8	62.1	3.9
1922	29.6	54.7	121.2	126.0	101.5	66.0	170.7	210.5	88.5	19.5	27.6	44.9
1923	.0	129.4	32.5	237.7	209.6	67.7	242.6	102.7	114.0	48.9	48.3	12.0
1924	.0	74.9	27.5	146.7	116.4	33.5	101.2	236.1	109.5	35.8	67.4	32.3
1925	134.0	2.7	96.3	6.0	137.3	100.0	126.5	187.3	12.0	20.1	115.5	44.2
1926	44.0	93.2	32.0	185.0	154.3	94.2	160.6	247.2	207.7	83.3	115.8	4.0
1927	10.5	52.2	36.8	101.7	89.8	46.7	145.4	123.8	76.9	14.6	15.8	13.7
1928	12.8	20.9	31.9	118.7	177.6	138.0	81.3	123.5	30.0	100.5	72.0	8.6
1929	.0	1.0	17.6	133.8	162.3	115.1	195.5	119.2	86.6	38.9	33.5	85.6
1930	150.1	20.1	282.2	261.8	182.7	154.0	92.2	104.0	129.2	54.6	56.6	27.6
1931	6.9	44.9	121.7	176.7	212.9	101.5	150.3	150.2	127.7	21.8	61.3	63.3
1932	6.4	45.3	170.4	136.3	118.5	129.7	155.0	154.8	172.7	33.4	34.7	34.0
1933	24.3	15.9	26.7	24.0	53.0	74.1	162.1	176.8	137.6	56.6	16.2	36.5
1934	3.2	16.2	22.1	116.9	105.4	114.1	150.6	180.4	22.9	45.9	37.3	2.8
1935	.0	76.3	11.5	74.6	221.7	69.5	117.8	94.8	83.7	92.9	21.1	110.1
1936	65.6	183.1	152.6	158.8	61.8	111.6	51.4	146.9	99.9	41.4	9.2	58.2
1937	39.9	27.0	134.4	251.6	138.6	166.6	151.2	197.8	13.8	43.0	138.8	20.3
1938	47.1	5.8	76.2	37.1	133.4	119.2	177.7	241.0	99.0	42.8	33.6	78.1
1939	25.0	19.5	32.0	112.3	49.5	98.9	197.8	139.1	11.4	24.3	82.8	11.7
1940	48.5	95.7	225.9	235.8	153.3	88.7	149.1	189.1	2.4	11.2	76.6	12.1
1941	61.3	46.5	122.7	221.7	184.0	118.1	137.1	116.2	82.6	72.4	188.1	100.5
1942	.8	5.0	162.8	189.0	202.0	140.4	96.2	187.9	103.2	8.2	11.7	33.3
1943	3.9	32.3	7.9	100.9	125.1	153.0	151.5	170.6	139.8	47.0	21.1	35.5
1944	5.2	16.1	48.6	99.9	109.6	72.8	132.2	216.1	124.6	57.4	112.3	26.9
1945	10.8	23.4	4.4	13.9	176.5	199.5	186.5	220.0	152.9	41.2	46.8	34.6
1946	3.6	2.5	38.5	212.0	119.6	196.0	136.2	221.7	103.0	59.9	19.2	7.5
1947	110.8	43.0	97.3	276.3	134.5	152.2	148.8	113.7	140.7	41.0	8.5	67.7
1948	6.1	2.1	53.6	132.3	122.7	167.0	183.6	187.6	107.7	85.5	28.3	32.8
1949	10.8	21.3	1.7	149.0	122.9	139.6	141.7	184.8	142.8	14.4	16.1	69.6
1950	24.0	1.6	56.6	139.8	84.4	81.4	200.8	151.9	120.4	55.7	12.8	2.4
1951	14.5	16.1	122.3	316.7	106.5	68.7	83.7	184.4	58.5	67.4	110.8	240.2
1952	.0	36.8	20.7	315.9	237.4	47.3	159.7	188.1	104.6	57.9	15.6	8.4
1953	9.9	1.9	9.2	147.1	121.6	192.7	82.5	103.2	48.0	76.7	27.6	82.9
1954	22.0	28.7	16.6	173.8	207.5	78.2	137.3	157.4	146.1	64.7	39.7	39.1
1955	8.8	73.1	34.3	137.4	84.4	81.6	159.7	363.4	262.5	53.8	41.0	161.8

TABLE 8
contd.

1956	146.3	87.3	77.9	145.5	114.0	142.5	169.2	221.3	93.9	81.2	30.5	27.3
1957	33.8	12.9	80.3	176.6	229.4	175.4	124.4	164.6	12.6	19.1	68.2	56.2
1958	70.6	126.7	86.7	99.7	102.2	161.4	152.2	223.8	127.3	75.7	15.1	103.3
1959	37.0	32.8	103.8	89.8	105.8	61.5	127.1	147.6	97.9	79.3	104.5	16.1
1960	41.2	13.4	141.4	158.4	95.6	58.8	134.5	237.5	63.3	27.3	63.7	39.7
1961	7.6	8.8	22.0	113.2	124.3	107.7	84.2	222.3	75.6	85.4	396.5	171.0
1962	60.8	4.7	76.3	115.8	203.2	98.0	191.1	156.9	130.3	79.5	120.5	55.6
1963	80.9	37.3	68.7	232.9	212.6	44.0	127.1	189.6	28.3	10.3	175.6	187.0
1964	14.9	52.2	121.5	232.2	122.5	91.8	234.2	131.0	177.0	62.4	23.2	53.1
1965	57.3	8.1	55.2	152.3	68.8	55.7	102.7	88.7	42.6	82.5	67.9	39.8
1966	5.3	103.0	84.3	258.6	47.9	69.0	210.9	143.2	98.6	34.8	68.7	6.2
1967	4.8	18.4	31.8	162.5	257.3	130.0	194.6	137.4	42.2	51.8	144.5	10.9
1968	.1	222.0	87.1	275.8	103.5	88.3	110.1	156.3	11.9	79.4	103.4	37.1
1969	74.6	69.3	63.8	14.4	129.0	28.1	137.5	110.1	100.5	54.6	68.9	5.1
1970	160.4	57.9	199.2	153.4	110.1	156.7	172.0	208.9	94.6	43.4	33.0	21.0
1971	42.3	5.3	18.1	124.7	135.5	123.7	180.7	253.4	78.7	49.1	24.3	172.2
1972	2.5	124.5	9.0	74.1	117.2	129.8	90.0	144.4	66.9	117.0	106.9	20.6
1973	84.3	96.8	1.6	16.3	153.1	72.3	98.5	171.2	205.7	12.7	81.7	5.9
1974	3.0	20.2	170.1	107.1	86.1	98.6	189.7	139.2	66.3	27.5	40.3	13.8
1975	.9	2.9	101.2	166.0	152.4	90.3	183.2	244.3	168.4	101.4	30.3	28.4
1976	20.0	6.9	9.2	114.0	112.3	99.9	173.5	170.5	41.4	28.6	62.7	13.3
1977	130.7	35.7	4.0	301.9	145.9	151.0	188.2	173.0	74.8	77.3	181.2	57.5
1978	122.5	102.8	201.9	100.5	136.4	179.4	122.0	109.1	169.9	101.4	26.3	83.9
1979	103.3	168.7	146.7	246.5	115.3	74.1	181.4	146.1	164.3	47.8	49.3	.9
1980	2.2	15.5	26.8	150.4	192.7	138.2	104.7	104.1	45.1	48.0	70.0	30.5

correlation in the annual runoff data and a statistical analysis resulted in no evidence to suggest that serial correlation did exist.

This process was carried out using all 9 sequences of inflows to produce curves describing the return period of failure for a particular retention level. The mean of these results is drawn in Figure 10 and the yields for return periods of 10, 20 and 50 years have been extracted and tabulated in Table 9. The results were also investigated for lower retention levels to extend the analysis to include much lower yields than were at first indicated.

3.3 CONCLUSIONS

The yields available for 10, 20 and 50 year return periods of failure and different retention levels are listed in Table 9. These have been plotted in Figure 11 to allow interpolation between the results, providing a continuous storage/yield relationship for the return periods concerned.

The error associated with the transfer of runoff data from station 1GC5 to the dam site is quite small but the confidence in the extension of the flows using the annual rainfall series is much less. However, with the data that is currently available and requiring results with return periods of up to 50 years, we believe that this method will provide the most reliable results possible.

The results of the yield available become more unreliable as the return period increases but they are well defined up to the 50 year return period. These results are based on the assumption that the reservoir is in a steady state, so that it has been constructed for long enough to negate the effect of initially being empty. If a reservoir is fairly small and is not expected to provide frequent overyear storage, the likelihood of filling reaches a steady value very soon after construction. However, in this case, the reservoirs considered include very large storages which could take many years to fill and, during the initial

Londiani reservoir storage yield analysis

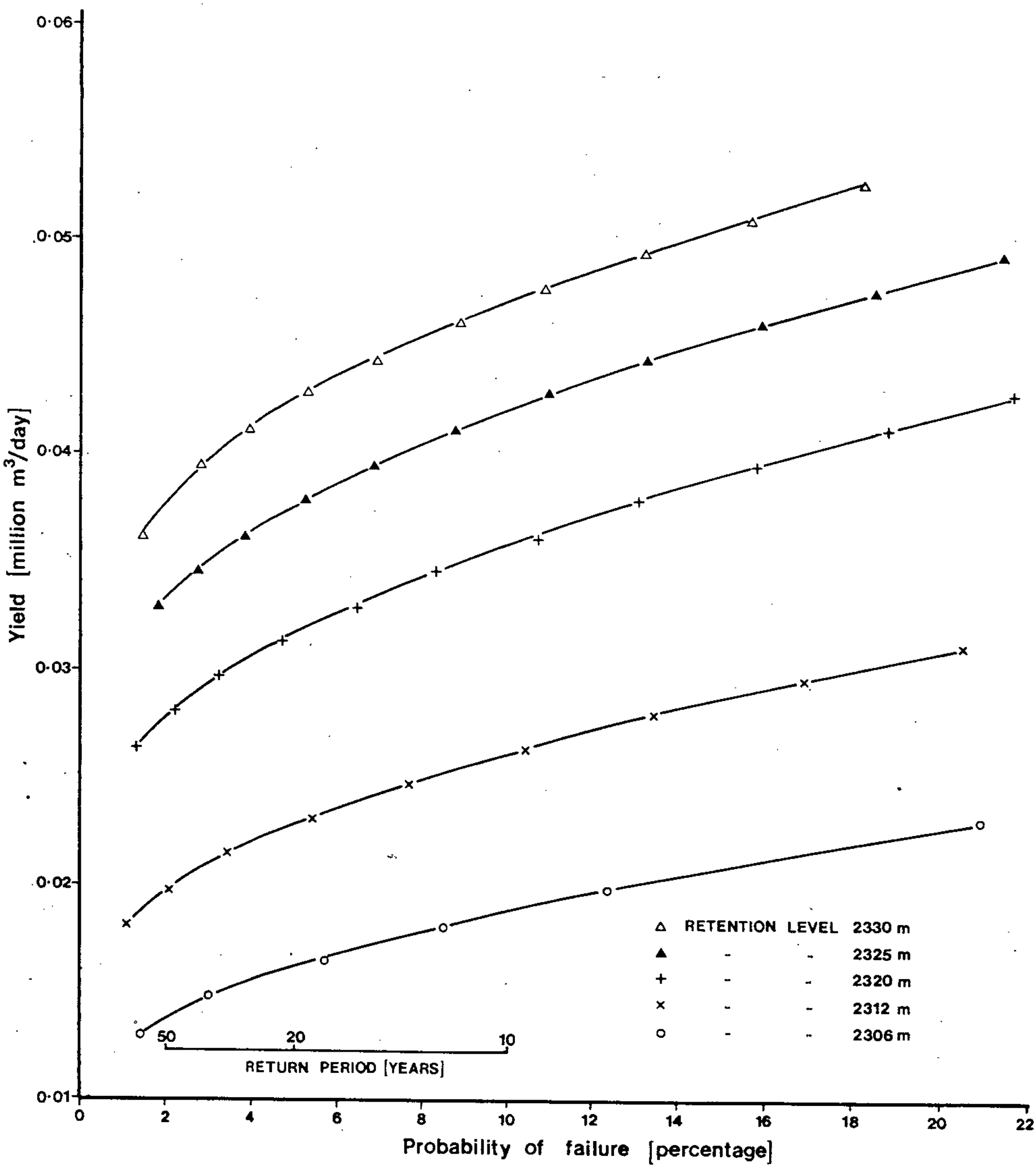


Figure 10

TABLE 9

Yield Results from Figure 11

Retention level (m)	Storage (million m^3)	Yield Available (thousand m^3/day) Return Period of Failure		
		10 yrs	20 yrs	50 yrs
2330	72.7	47.2	42.5	37.8
2325	45.5	42.2	37.5	33.2
2320	26.9	35.8	31.8	27.8
2313	12.0	26.0	22.8	19.5
2306	6.0	18.8	16.0	14.0

Storage - yield curves for Londiani reservoir

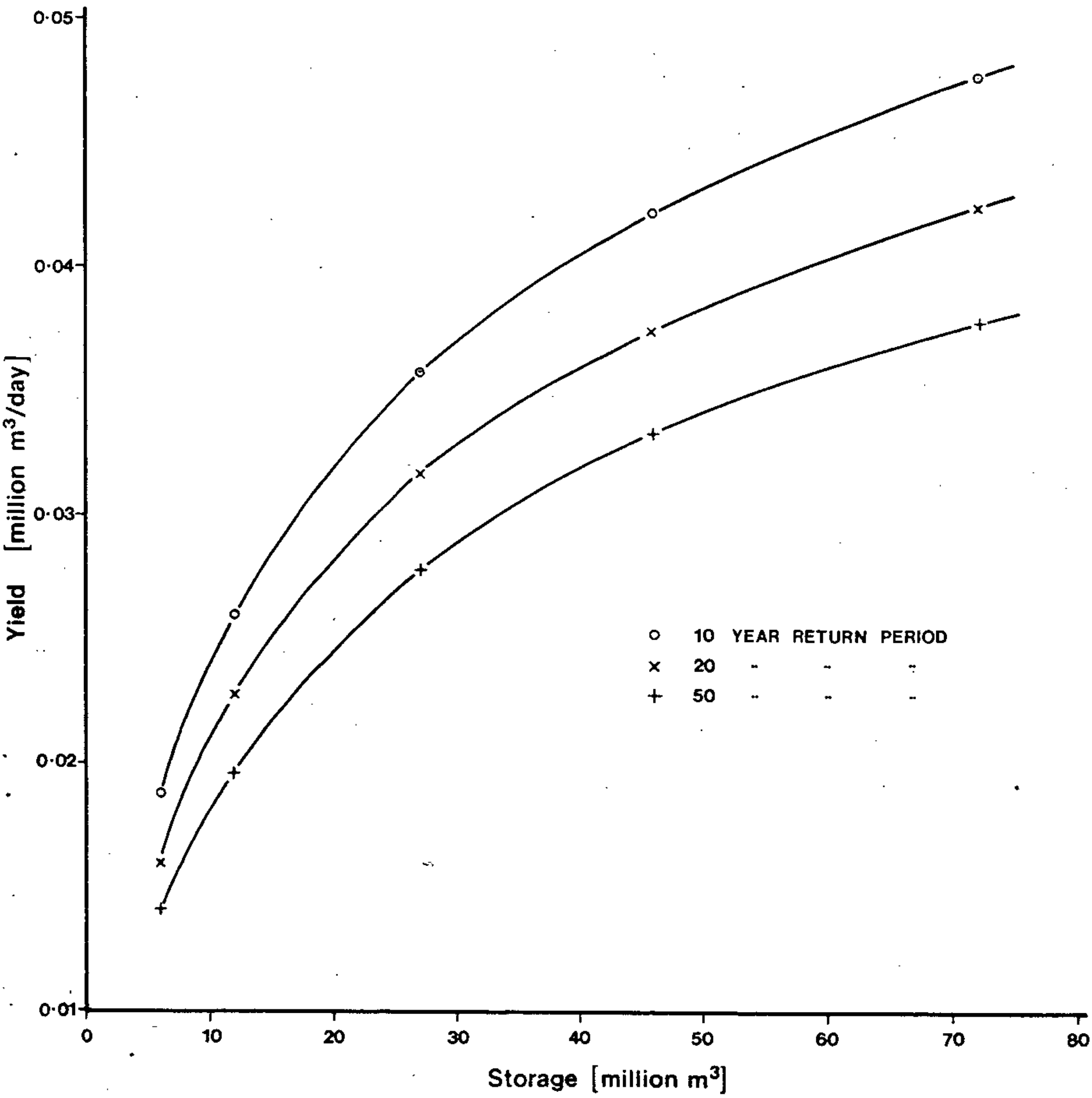


Figure 11

years the yield available, with a given risk, will be much reduced. If it is decided to construct a large reservoir in this area an analysis should be carried out to determine the length of time for which the reservoir will be likely to be unable to supply the yield and also the reduced yield which will be available.

The sedimentation information available is very scarce and has been discussed in the introduction. Using the figure of 0.5 mm per year we arrive at a sedimentation rate of about 3.5 million m^3 in 50 years. As the reservoir at the higher retention levels is designed to store several years of runoff the trap efficiency will be very nearly 100 per cent and thus all this sediment will be stored in the reservoir. This figure of 0.5 mm is suggested from research carried out on the Tana river (Ref. 4) and as such will only provide a possible estimate of sedimentation. It is imperative that sediment samples are recorded in the area particularly as this region is undergoing deforestation which increases the soil erosion dramatically.

Erosion of soil by rainfall is usually related to the intensity of rain but in regions where the mean annual precipitation is more than 1000 mm there is usually dense forest vegetation as in the upper reaches of the Londiani catchment. This forms a canopy which protects the soil. Severe soil erosion will take place in these areas if the vegetal cover is removed, exposing the soil to intense rainfall. Thus the extent of silt deposition in the reservoirs will largely depend on the deforestation taking place and the provision of soil conservation schemes.

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APPENDIX

THE GOULD PROBABILITY MATRIX METHOD

The Gould method requires that the reservoir is divided into several (N) states of equal storage value. Each year of the inflow data is treated separately and is routed through the reservoir, starting the reservoir in each of the N states and noting the state in which it finishes. When this procedure has been repeated for each year of data the results are collated in a transition matrix which expresses the probability of ending in any of the N states, conditional on the starting state. At the same time, the number of occasions in which the reservoir fails or spills is counted and noted with its corresponding starting state. Thus we can determine the probability of spilling, failing and ending in any particular state, conditional on the starting state. We need only determine the probability of being in each of the states at the start of a year and then the joint probability of this and of failing will determine the steady state likelihood of failure.

The steady state probability vector of storage contents can be determined from the transition matrix and starting conditions of the reservoir. If the transition matrix $|T|$ is multiplied by the initial vector of probabilities of starting contents $|P|$ we will arrive at the vector of probabilities of starting contents at the second year.

That is

$$|P|_2 = |T| x |P|_1$$

This process can be continued according to the scheme

$$|P|_{t+1} = |T| x |P|_t$$

However, with time, the vector $|P|_t$ reaches a steady state as the initial conditions at the beginning of the first year become negligible. Once the vector $|P|_t$ reaches a steady state this describes the likelihood of being in any of the N states and this occurs when

$$|P|_{t+1} = |P|_t$$

We are now in a position to determine the probability of failure which is the sum of the products of the probability of the reservoir being in each particular zone and the probability of failure from starting in that zone.

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