INSTITUTE of
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[redacted]

GREATER NAKURU WATER SUPPLY

MALEWA DAM

HYDROLOGICAL ANALYSIS

This report is prepared for

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INTRODUCTION

The Malewa dam is one of the two dams proposed in the first phase of the Greater Nakuru Water Development Plan. The dam site is situated on the Malewa river some 14 kilometres upstream of its confluence with the Turasha river, and has a catchment area of 616 km^2. Figure 1 shows the catchment boundaries of the Malewa and Turasha rivers and the locations of the dam sites, the major river gauging stations and long-term raingauges.

The hydrology of the Malewa river basin has been discussed in some detail in previous reports (Refs 1, 2, 3). However the objectives of this study are somewhat different and the hydrology has been reappraised. This report gives estimates of spillway design and construction floods for the dam site, and the yield available from the reservoir for 10, 20 and 50 year return periods of failure and several retention levels.

The tributaries of the Upper Malewa rise in the Aberdares at altitudes of over 3000 metres. From there they join together on the plain to the south of Ol Bolossat to form the main stem of the Malewa river, which then flows southwards through gorges to the Rift Valley. The upper regions of the catchment are largely mixed mountain forest and moorland. In contrast the remainder is mostly settled and cultivated.

The climate of the Malewa catchment is controlled by the movement of the Intertropical Convergence Zone (ITCZ). This zone is a belt, parallel to the equator formed by a series of low pressure areas, which moves with the sun north and south of the equator. Two periods of rainfall are associated with the movement of the zone through Kenya and the instability caused by the south-east and north-east monsoons; the "long rains" occur from March to May and the "short rains" from October to December. An additional period of rainfall can occur in July and August. These are known as the "continental rains" which result from the development of local anticyclones.

Although this description of the climate holds for the eastern side of the Rift valley, the distribution of rainfall over the catchment is not uniform. The mean annual rainfall ranges from about 950 mm near the dam site to over 1500 mm on the Aberdares. The potential open water evaporation ranges from 1600 to 1400 mm.

Location map

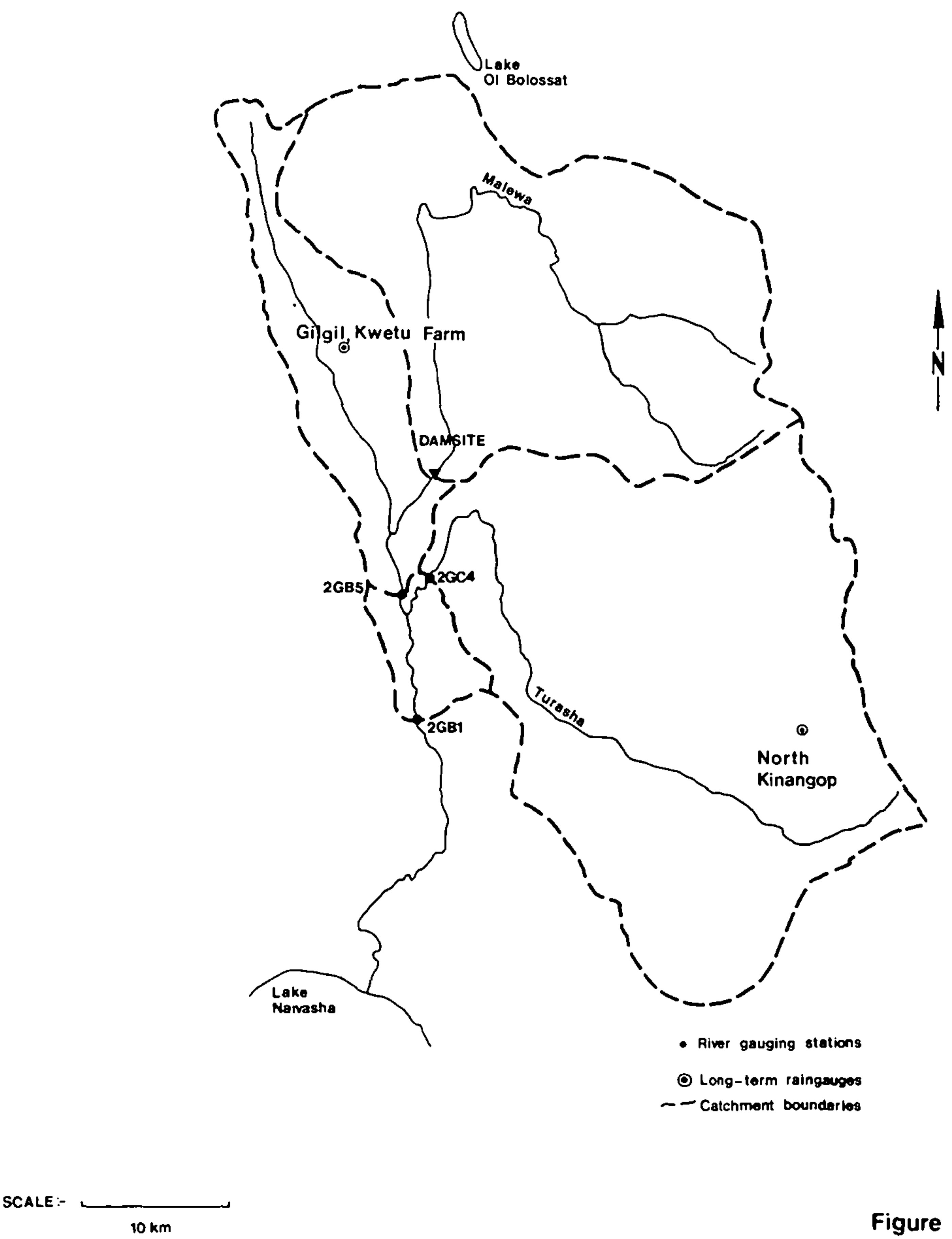


Figure 1

The Rift Valley is a result of major volcanic and tectonic activity; the geology of the Malewa reflects this origin. The region is heavily faulted and is characterised by gently dipping hard agglomerates and sandy tuffs with scattered, thin soil cover.

1.1 AVAILABLE DATA

Rainfall

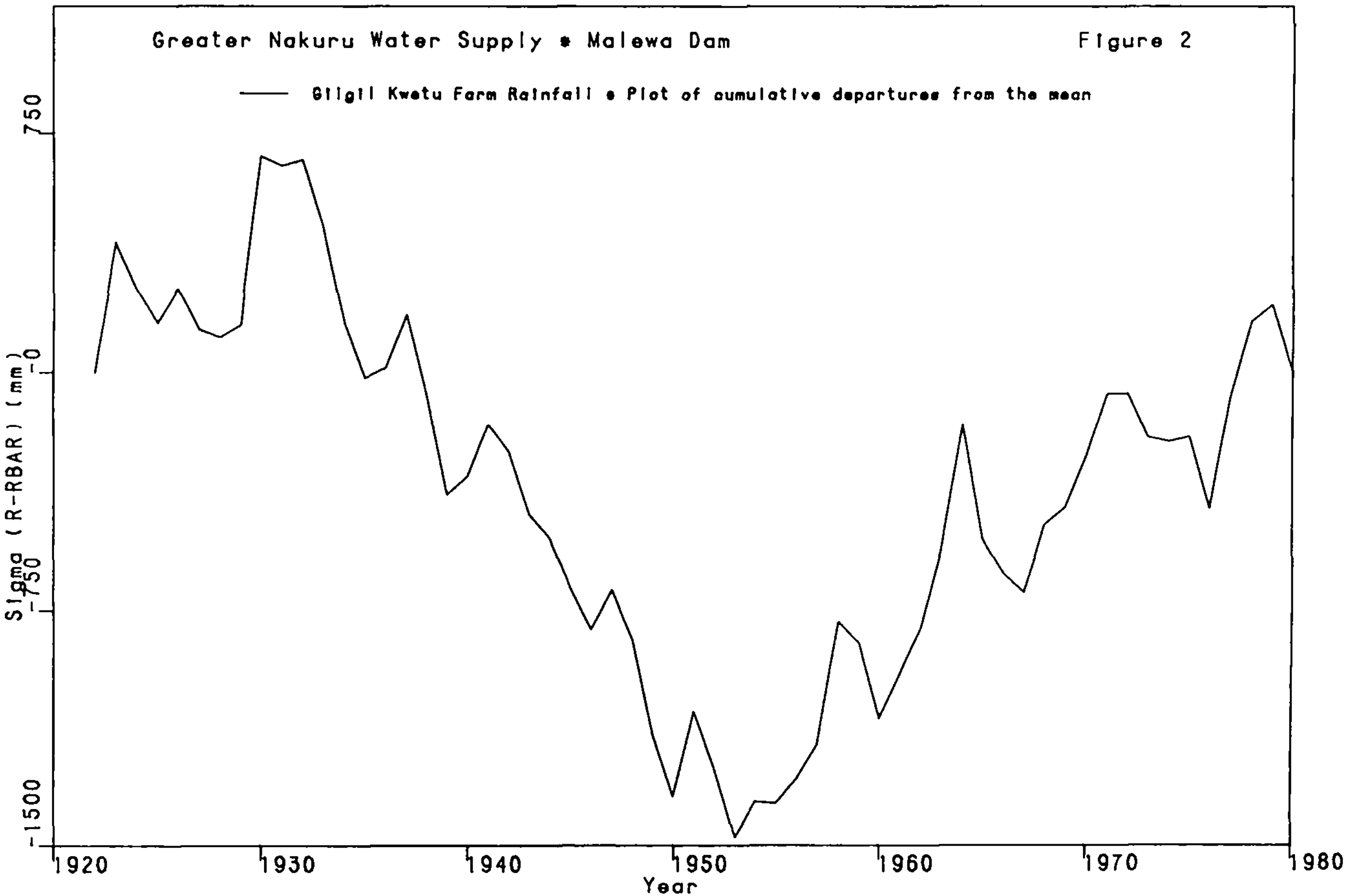
There are several raingauges located in the Malewa catchment. Of these only two can be considered long-term stations, North Kinangop (1915-1980) and Gilgil, Kwetu Farm (1923-1980) (Figure 1). The other stations have much shorter records, and more importantly do not cover the whole period for which streamflow records exist.

The rainfall records are published by the Kenya Meteorological Department (formerly the East African Meteorological Department), and in 1966 a map of mean annual rainfall was produced. In the National Water Master Plan (1979) (Ref 4) it was concluded that only minor changes were needed to update the annual rainfall estimates. Using the two sources the estimates of mean annual rainfalls shown in Table 1 were calculated.

The two long-term rainfall records were examined to detect any evidence of persistence in the annual data. The low values of the lag - 1 serial correlation coefficient suggests that there is no year to year persistence in the rainfall data. However there is evidence that over the period 1930 to 1950 rainfall was in general below average. Figure 2, which is a plot of cumulative departures from the mean, clearly shows this effect which is reflected in the falling levels of Lake Naivasha over the same period (Ref 3).

Runoff

The locations of the major gauging stations in the Malewa catchment are shown in Figure 1; information on these stations is summarised in Table 2. At the three stations only a limited number of current meter gaugings are available, and these invariably cover the low and medium range of discharges. High flows are generally outside the rated range of the stations. The upper portions of the rating curves used by the MOWD for calculating discharge from stage are drawn by extrapolation. It has not been feasible to reassess the rating curves during the time available,



Greater Nakuru Water Supply * Malewa Dam

Figure 2

MEAN ANNUAL RAINFALL TABLE 1

Location	River	Area (km)	Mean annual rainfall (mm)
2GB1	Malewa	1430	1030
2GC4	Turasha	695	1090
2GB5	Malewa	667	950
DamSite	Malewa	616	980

RIVER GAUGING STATIONS TABLE 2

River	Gauging Station	Control	Gauge	Period of record
Malewa	2GB1	compound weir	staff plus recorder	1932 - present
Malewa	2GB5	road bridge	staff	1959 - present
Turasha	2GC4	compound weir	staff plus recorder	1952 - present

ESTIMATED OPEN WATER EVAPORATION FROM THE RESERVOIR (mm) TABLE 3

J	F	M	A	M	J	J	A	S	O	N	D	Total
142	139	141	112	128	112	109	115	123	135	118	121	1495

(EVAP STATIONS USED ?)

&

for this study.

Chart recorders are installed at 2GB1 and 2GC4 and the readings from these are supported by readings from staff gauges. Mean daily stage is calculated from the continuous records, and mean daily discharge computed using rating tables based on current meter gaugings and their extrapolation to higher discharges. However, inspection of the charts for 2GB1 suggested that for some flood events mean daily discharge was calculated from the hourly discharge hydrograph.

The averaging of stage readings rather than discharges will introduce a systematic underestimate of discharge owing to the non-linearity of the stage discharge relationship. However for days when the range of river stage is small, the errors will tend to be insignificant.

At 2GB5 stage readings are taken only once every 4 or 5 days, the missing readings being deduced by interpolation between the observed values. This fact, combined with the uncertainty of the shifting control at the station makes the accuracy of the flow data at this site particularly suspect.

Raw data tabulated *

The relevant flow data were abstracted from the MOWD river gauging station files, these had been calculated manually from mean daily stage and the appropriate rating table. Recently the daily stage data have been available on computer, and the conversion to discharge processed automatically. At the time of this study computer printouts of the flow data were unavailable.

Evaporation

The estimates of evaporation from open water surfaces published by Woodhead (Ref 5) were calculated from meteorological data using Penman's equation. The stations closest to the dam site for which evaporation estimates are available are Ol Joro Orok and Naivasha. Estimates of the open water evaporation from the reservoir surface (Table 3) are based on the mean of these two stations.

Sedimentation

A summary of the sedimentation data available for Kenya was published in 1974 (Ref 6). The data compiled in that report were collected during the period 1948-65 and for the gauging stations for which adequate data exist, they are presented in the form of sediment-discharge rating curves. The only gauging station in the Malewa catchment included is 2GB1.

The sediment rating curve for 2GB1 covers a wide range of flows, and is based on over 250 observations. Since 1965 insufficient data have been collected to establish whether the curve is still valid. In some parts of Kenya land use has changed to the extent that the earlier records are no longer relevant to predictions of present or future conditions.

A flow duration curve is needed to calculate the annual sediment yield of a river. For 2GB1 only a monthly flow duration curve was available (Ref 7); sedimentation rates calculated using this curve will tend to be underestimated. Assuming a bulk density of 0.7 m^3/tonne for sand (Ref 6), an annual sediment load of about 0.025 x 10^6 m^3 was estimated, giving an erosion rate of about 0.02 mm/year over the catchment.

Recent work on the Tana River reports that erosion rates of about 0.5 mm/year are indicated by the evidence available (Ref 8). We consider that this is likely to be an upper bound for erosion on the Malewa.

2 FLOOD ANALYSIS

2.1 INTRODUCTION

The objective of this flood analysis is to provide estimates of spillway design and construction floods at the dam site. The range of return periods for the spillway design floods is 100 to 500 years, whilst for construction floods it is 5 to 50 years.

The length of records available at 2GB1 (1936 to 1980) means that construction floods can be estimated by statistical analysis of the available annual maximum data. The statistical method used for this report is one of those described in detail in the Flood Studies Report (FSR) (Ref 9). For spillway design floods it was necessary to extrapolate. Rainfall data are often more plentiful than runoff data, and also cover a longer period. Consequently the statistics of extreme rainfall can be extrapolated more easily than flood statistics. Design storms of the required return period can be estimated from rainfall/duration/frequency relationships. The storm is then converted to runoff using a simple model such as the unit hydrograph losses model.

It was not possible to deduce the parameters of the unit hydrograph model from observed data. Based on a number of assumptions concerning the physical and climatic characteristics of the catchment, empirical equations were used to estimate initial values for the parameters. These were then adjusted so that the unit hydrograph predictions broadly fitted the observed frequency curve of annual maxima.

2.2 DATA USED IN FLOOD ANALYSIS

Data from gauging stations 2GB1 and 2GC4 were used in the flood analysis; data from 2GB5 were not considered because of the infrequent observations and the unstable control section.

Annual maximum mean daily flows were extracted from the MOWD files. For 2GB1 the instantaneous peak stages were abstracted from the recorder charts and converted to discharges using the MOWD rating tables; no charts for 2GC4 were available. The magnitude of the peak flood in November 1961 was taken from a previous report (Ref 10).

Rainfall data were taken from two published sources, namely the MOWD rainfall frequency atlas of Kenya (Ref 11) and the TRRL design manual (Ref 12). Daily data from some of the gauges in the Malewa catchment were also used.

2.3 STATISTICAL ANALYSIS

A flood frequency curve can be drawn in terms of $Q_T/\bar{Q}$, where Q_T is the flood of return period T years and $\bar{Q}$ is the mean annual flood. In areas where few flow records exist, data from different gauging stations can be pooled together to improve the accuracy of the curve. Over 40 years of both instantaneous and mean daily annual maximum discharge data were available for 2GB1. With the exception of 2GC4, it was decided that the data from other gauging stations in the catchment were too poor, either because of infrequent observations or poor rating curves, to contribute usefully to the analysis.

Flood frequency curves were constructed from the two series of annual maximum mean daily flows and the single series of corresponding peak flows. 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 $y_i \quad - \ln(- \ln F_i)$

where F_i is the plotting position expressed as a probability,

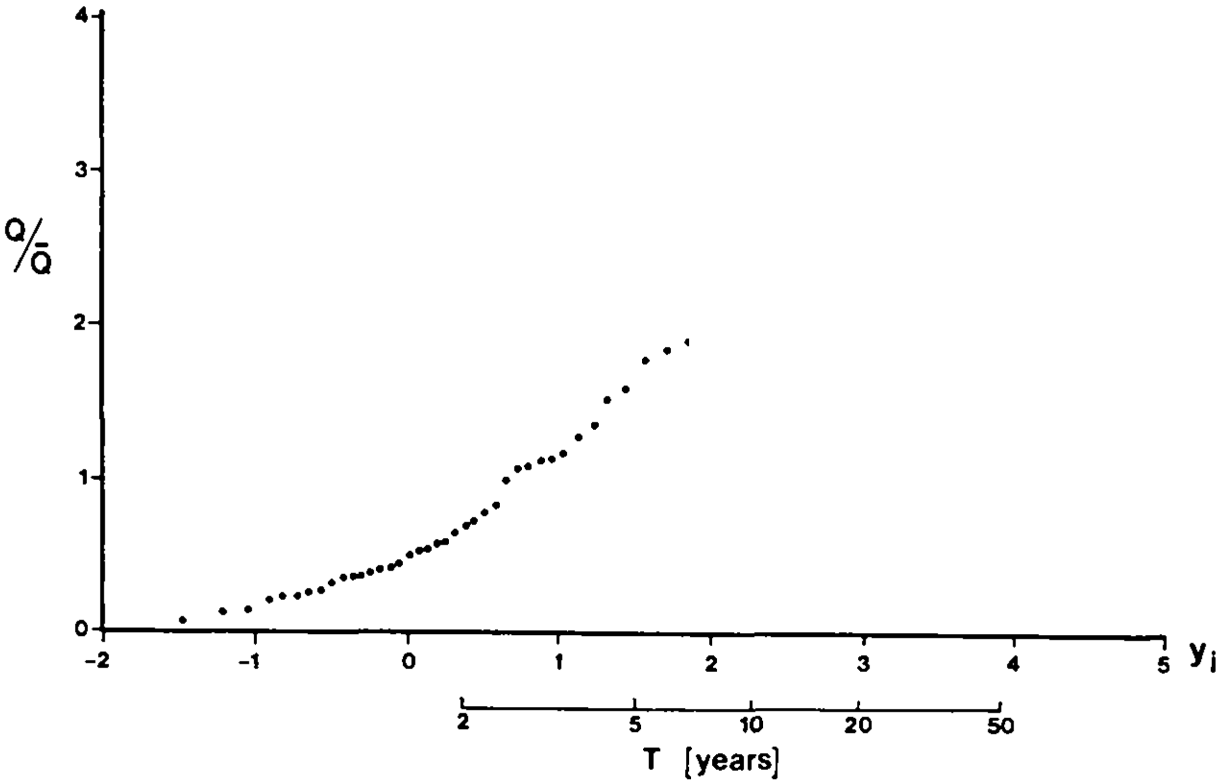
i is the rank of the event,

and is the number of events in the series.

Comparison of the curves for instantaneous and mean daily discharges for 2GB1 shown in Figure 3, indicates that frequency distributions of the two series are very similar. The curve for 2GC4 also had the same shape.

Thus floods of return period up to about 50 years can be estimated from the bottom curve in Figure 3, provided $\bar{Q}$ is known.

2GB1: Flood frequency curves



Mean daily discharge

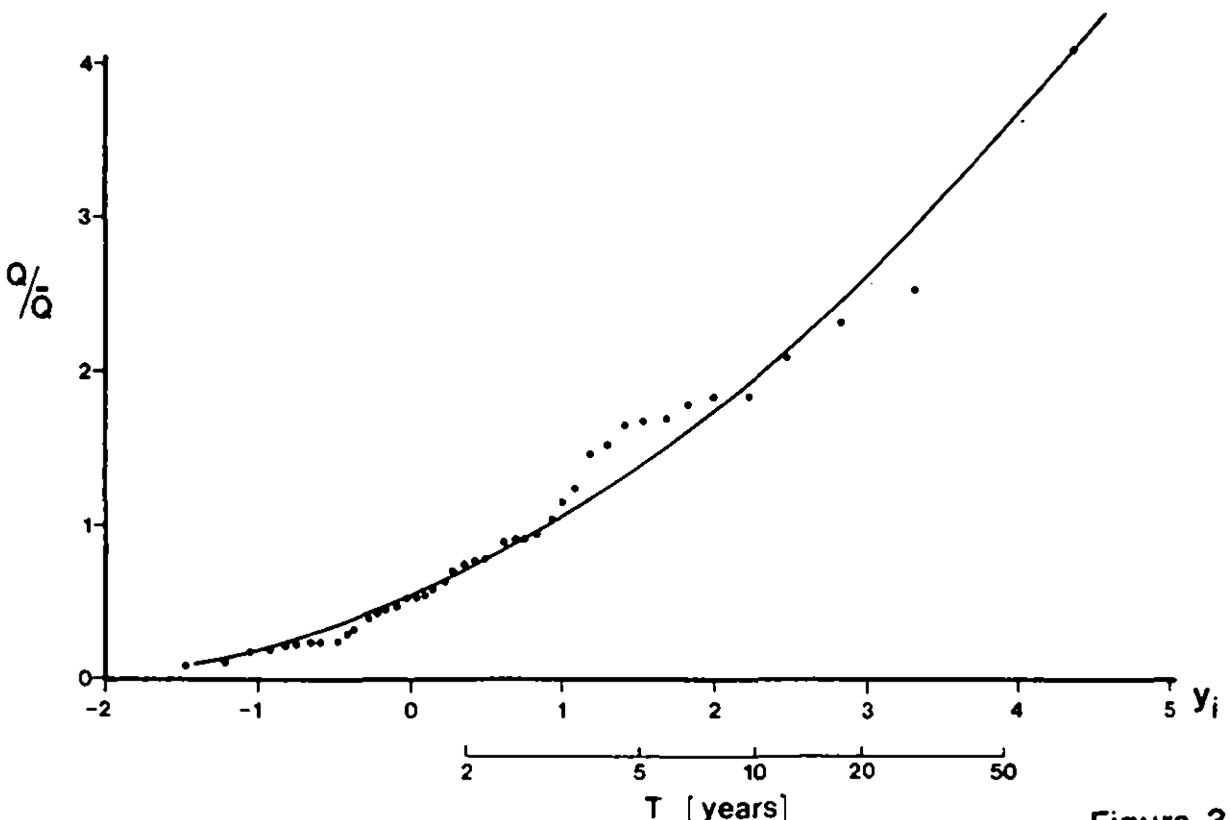


Figure 3

Instantaneous discharge

A number of empirical formulae have been proposed to relate the flood magnitudes to catchment area, a general form being

$$Q = k A^n$$

where Q is the discharge
and k and n are constants.

Comparison of the mean annual daily floods for 2GB1 and 2GC4 suggests that a value of $n = 0.61$ is appropriate.

The ratio of annual maximum instantaneous to mean daily discharge at 2GB1 and 2GC4 is 1.4; from this and the relationship above the annual maximum instantaneous flood, $\bar{Q}$, at the dam site is estimated as 47 m^3/s.

Another method of estimating $\bar{Q}$ would be to use the TRRL design method. This method was derived for small catchments, and its use was considered to be inappropriate in this case.

2.4 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.

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

An attempt was made to derive the parameters of the unit hydrograph at 2GB1 from observed data. Although there were several well defined flood events, the major problem was to estimate the timing and size of the rainstorms that caused these floods. Although there are no autographic raingauges located within the Malewa catchment it was hoped that the regular diurnal pattern of rainfall (Ref 13) would enable a rough estimate of the timing of storms to be made. The areal coverage of raingauges for which daily data was available was poor, and it was impossible to use the observed data to estimate the parameters of the unit hydrograph.

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.

It was therefore necessary to derive a synthetic unit hydrograph from catchment characteristics. 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:-

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

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:-

$$T_B = 2.52\ Tp$$

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

Using the catchment characteristic data summarised in Table 4 the following values are obtained

T_p	12 hours
T_B	30 hours
Q_p	18 $m^3/s/100\ km^2$

Note that these figures have been rounded.

Design storm duration

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

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

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. A design duration of 25 hours was chosen.

Design storm depth

Intensity-duration-frequency curves and maps have been prepared for a number of rainfall stations in Kenya (Ref 11). Using the curves and maps together, the 24 hour rainfalls for the Malewa dam site catchment were estimated for return periods of 5 to 100 years. The 200 and 500 year return period rainfall were estimated by extrapolation of the graph in Figure 4. Lumb's work (Ref 14) was used to given an estimate of 250 mm for the probable maximum precipitation (PMP).

The 5 year 24-hour rainfall-intensity-duration frequency curves were 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

DAM SITE CATCHMENT CHARACTERISTICS TABLE 4

Area	616	km^2
S_{1085}	9.2	m/km
Main stream length	68	km

Rainfall growth curve

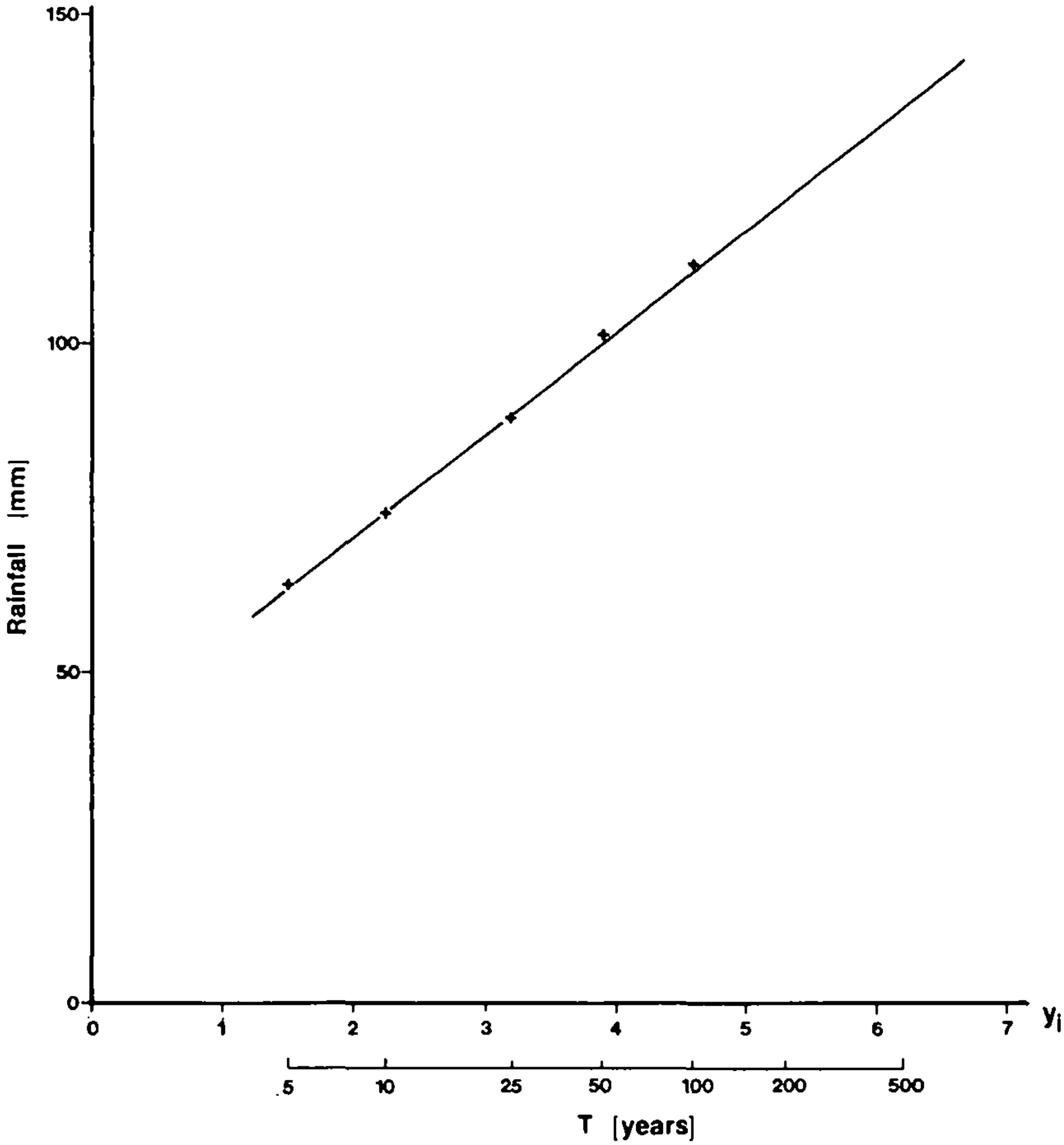


Figure 4

occurred within the same storm. The 5 year storm of 25 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.

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 exceedence probability over larger areas. ARF's have been calculated by the TRRL (Ref 12), and in the absence of other data, it has been assumed that an ARF of 0.74 is valid for design storms of all return periods. The 1 in 5 year areal profile for the dam catchment is shown in Figure 5.

Catchment wetness index

An indication of how wet a 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 is 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

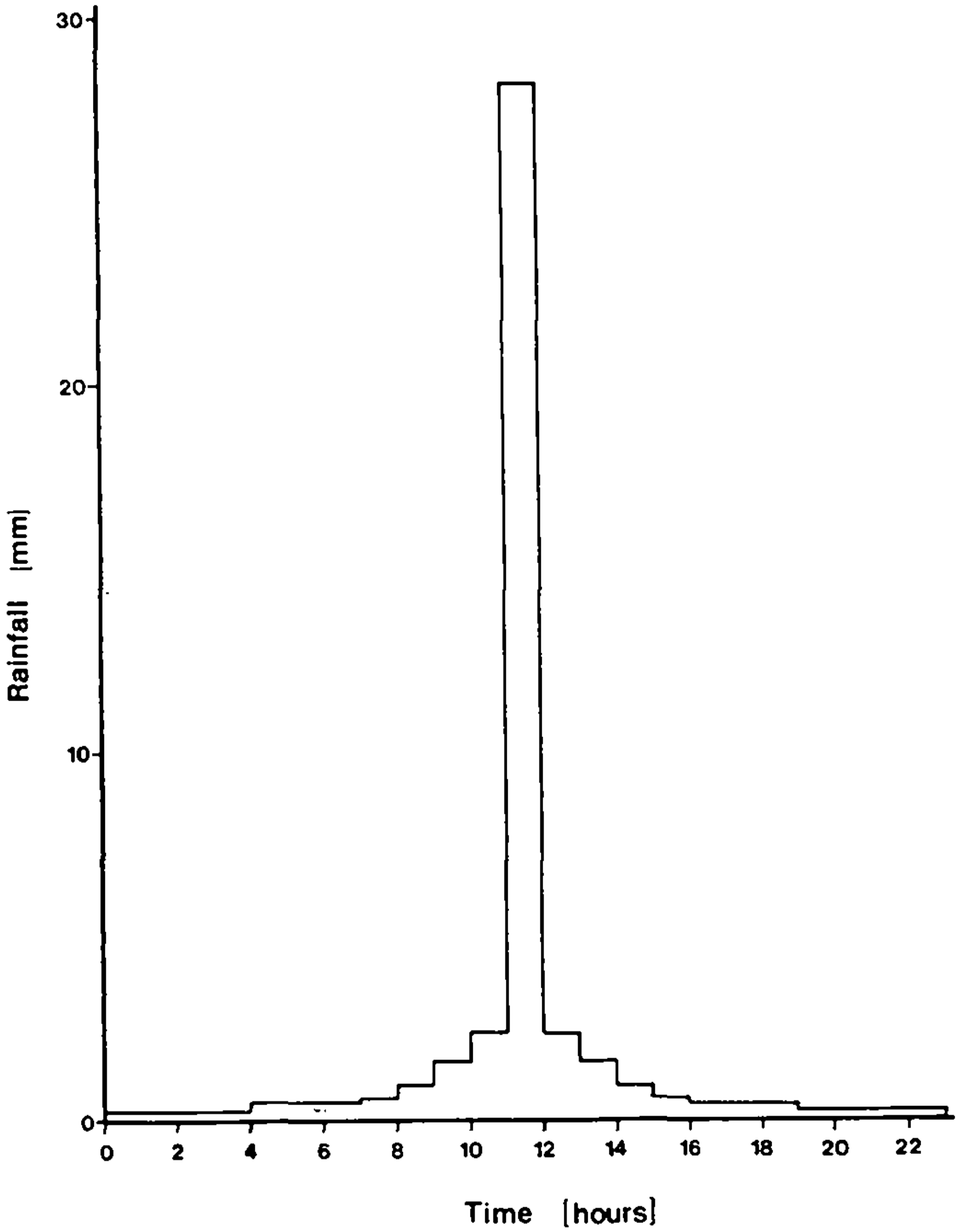


Figure 5

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.

Figure 3.3 of the 1979 Turasha Report (Ref 2) shows that a discharge of 54 ft^3/sec is exceeded 50% of the time; this is equivalent to a specific discharge of 0.002 $m^3/s/km^2$. We consider that it is reasonable to use this value for the flood calculations at the Malewa dam site.

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 3. 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 5; the model predictions and flood frequency curve are compared in Figure 6.

2.5 DESIGN FLOODS

Using the methods discussed above design storms for the 200 and 500 year return period floods, as well as the probable maximum precipitation, were calculated. These storms were multiplied by values of percentage runoff deduced from Table 5, and convoluted with the unit hydrograph. To this the baseflow was added to give estimates of the 200 and 500 year return period and the probable maximum floods (Tables 6 to 8). The flood hydrographs are shown in Figure 7.

TABLE 5

DESIGN FLOOD PARAMETERS

Return Period (years)	Rainfall (mm)	Percentage Runoff (%)	Volume ($m^3 x 10^6$)	Q_{max} (m^3/s)
5	48.1	12.5	4.01	63.1
10	56.5	15.0	5.55	88.5
25	66.9	17.5	7.56	121.6
50	77.9	20.0	9.96	161.5
100	85.1	22.5	12.17	198.0

Unit hydrograph model: Flood predictions

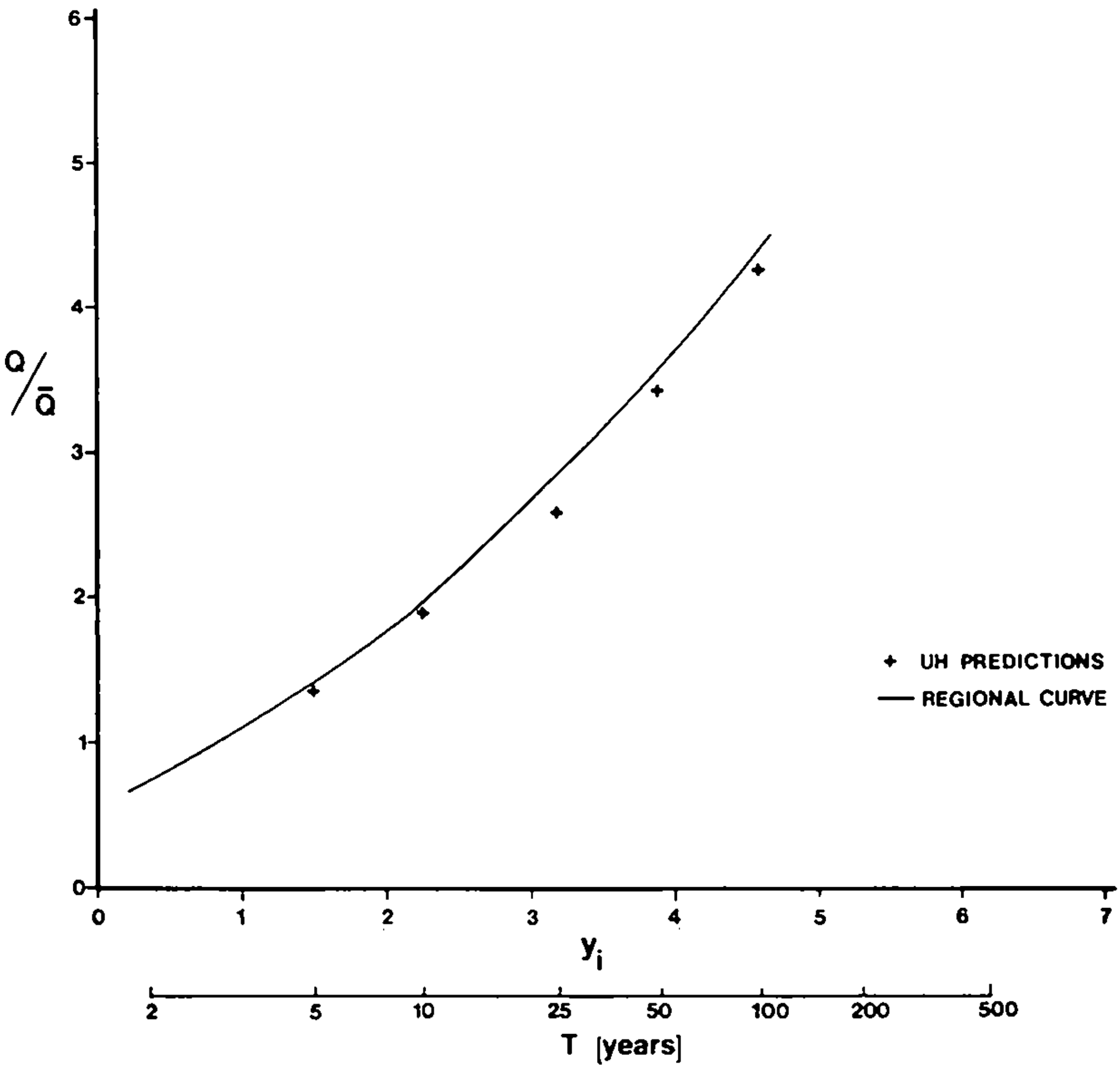


Figure 6

200 Year Flood

Area (Sq.Km.)	616.00
Data interval (Hr)	1.00
Design duration (Hr)	25.00
Total rain (mm)	92.21
Percentage runoff	27.50
Base flow (cumecs per sq.km)	.00296

Triangular unit hydrograph computed from Tp= 12.0

Convolution of unit hydrograph and net rain profile

Time	Total Rain mm.	Net Rain mm	Unit Hydrograph ordinate	Total Hydrograph cumecs	
.00	.32	.09	.00	1.82	
1.00	.46	.13	1.53	1.90	
2.00	.50	.14	3.06	2.10	
3.00	.55	.15	4.59	2.43	
4.00	.61	.17	6.11	2.91	
5.00	.69	.19	7.64	3.54	
6.00	.77	.21	9.17	4.35	
7.00	.88	.24	10.69	5.35	
8.00	1.07	.30	12.22	6.59	
9.00	1.81	.50	13.75	8.10	
10.00	3.09	.85	15.28	10.08	
11.00	8.38	2.30	16.81	12.87	
12.00	53.96	14.64	18.33	17.82	
13.00	8.38	2.30	17.33	36.60	
14.00	3.09	.85	16.34	57.35	
15.00	1.81	.50	15.34	78.65	
16.00	1.07	.30	14.34	100.26	
17.00	.88	.24	13.34	121.84	
18.00	.77	.21	12.34	143.36	
19.00	.69	.19	11.34	164.75	
20.00	.61	.17	10.34	185.95	
21.00	.55	.15	9.35	206.84	
22.00	.50	.14	8.35	227.09	
23.00	.46	.13	7.35	246.16	
24.00	.32	.09	6.35	261.76	-Peak-
25.00			5.35	254.34	
26.00			4.35	243.35	
27.00			3.36	231.02	
28.00			2.36	217.93	
29.00			1.36	204.37	
30.00			.36	190.44	
31.00				176.21	
32.00				161.76	
33.00				147.13	

Greater Nakuru Water Supply   Malewa Dam Design Floods   TABLE 6 (contd)

Convolution of unit hydrograph and net rain profile

Time	Total Rain mm	Net Rain mm	Unit Hydrograph ordinate	Total Hydrograph cumecs
34.00				132.35
35.00				117.46
36.00				102.49
37.00				87.50
38.00				72.66
39.00				57.98
40.00				43.57
41.00				29.60
42.00				16.73
43.00				10.20
44.00				7.28
45.00				6.39
46.00				5.29
47.00				4.42
48.00				3.71
49.00				3.13
50.00				2.66
51.00				2.33
52.00				2.08
53.00				1.92
54.00				1.84

TOTAL FLOOD VOLUME (MILLION M3)   16.02

Greater Nakuru Water Supply   Malewa Dam Design Floods   TABLE 7

500 Year Flood

Area (Sq.Km.)	616.00
Data interval (Hr)	1.00
Design duration (Hr)	25.00
Total rain (mm)	102.60
Percentage runoff	32.50
Base flow (cumecs per sq.km)	.00306

Triangular unit hydrograph computed from Tp= 12.0

Convolution of unit hydrograph and net rain profile

Time	Total Rain mm	Net Rain mm	Unit Hydrograph ordinate	Total Hydrograph cumecs	
.00	.35	.11	.00	1.98	
1.00	.51	.17	1.53	1.99	
2.00	.56	.18	3.06	2.25	
3.00	.61	.20	4.58	2.69	
4.00	.68	.22	6.11	3.31	
5.00	.77	.25	7.64	4.14	
6.00	.85	.28	9.17	5.20	
7.00	.98	.32	10.69	6.53	
8.00	1.19	.39	12.22	8.15	
9.00	2.02	.66	13.75	10.14	
10.00	3.44	1.12	15.28	12.75	
11.00	9.32	3.03	16.81	16.40	
12.00	60.04	19.51	18.33	22.91	
13.00	9.32	3.03	17.33	47.61	
14.00	3.44	1.12	16.34	74.90	
15.00	2.02	.66	15.34	102.96	
16.00	1.19	.39	14.34	131.32	
17.00	.98	.32	13.34	159.71	
18.00	.85	.28	12.34	188.01	
19.00	.77	.25	11.34	216.13	
20.00	.68	.22	10.34	244.00	
21.00	.61	.20	9.35	271.47	
22.00	.56	.18	8.35	298.11	
23.00	.51	.17	7.35	323.18	
24.00	.35	.11	6.35	343.69	-Peak-
25.00			5.35	333.95	
26.00			4.35	319.45	
27.00			3.36	303.28	
28.00			2.36	286.06	
29.00			1.36	268.73	
30.00			.36	249.91	
31.00				231.30	
32.00				212.20	
33.00				192.96	

Convolution of unit hydrograph and net rain profile

Time	Total Rain mm	Net Rain mm	Unit Hydrograph ordinate	Total Hydrograph cumecs
34.00				173.53
35.00				153.95
36.00				134.26
37.00				114.55
38.00				95.03
39.00				75.73
40.00				56.78
41.00				38.41
42.00				21.48
43.00				12.91
44.00				9.54
45.00				7.89
46.00				6.45
47.00				5.30
48.00				4.36
49.00				3.61
50.00				3.01
51.00				2.56
52.00				2.23
53.00				2.01
54.00				1.91

TOTAL FLOOD VOLUME (MILLION M3) 20.96

GREATER NAKURU WATER SUPPLY : MALEWA DAM DESIGN FLOODS TABLE 8

PROBABLE MAXIMUM FLOOD

AREA (SQ.KM.)	616.00
DATA INTERVAL (HR)	1.00
DESIGN DURATION (HR)	25.00
TOTAL RAIN (MM)	185.08
PERCENTAGE RUNOFF	40.00
BASE FLOW (CUMECS PER SQ.KM)	.00395

TRIANGULAR UNIT HYDROGRAPH COMPUTED FROM TP= 12.0

CONVOLUTION OF UNIT HYDROGRAPH AND NET RAIN PROFILE

TIME	TOTAL RAIN MM	NET RAIN MM	UNIT HYDROGRAPH ORDINATE	TOTAL HYDROGRAPH CUMECS	
.00	.63	.25	.00	2.43	
1.00	.92	.37	1.53	2.67	
2.00	1.01	.40	3.06	3.26	
3.00	1.10	.44	4.58	4.22	
4.00	1.23	.49	6.11	5.60	
5.00	1.38	.55	7.64	7.44	
6.00	1.54	.62	9.17	9.80	
7.00	1.76	.70	10.69	12.75	
8.00	2.15	.86	12.22	16.35	
9.00	3.64	1.46	13.75	20.77	
10.00	6.21	2.48	15.28	26.55	
11.00	16.82	6.73	16.81	34.67	
12.00	108.30	43.32	18.33	49.12	
13.00	16.82	6.73	17.33	103.95	
14.00	6.21	2.48	16.34	164.53	
15.00	3.64	1.46	15.34	226.82	
16.00	2.15	.86	14.34	289.79	
17.00	1.76	.70	13.34	352.81	
18.00	1.54	.62	12.34	415.64	
19.00	1.38	.55	11.34	478.08	
20.00	1.23	.49	10.34	539.95	
21.00	1.10	.44	9.35	600.94	
22.00	1.01	.40	8.35	660.08	
23.00	.92	.37	7.35	715.74	
24.00	.63	.25	6.35	761.27	-PEAK-
25.00			5.35	739.63	
26.00			4.35	707.53	
27.00			3.36	671.55	
28.00			2.36	633.32	
29.00			1.36	593.74	
30.00			.36	553.07	
31.00				511.54	
32.00				469.35	
33.00				426.64	

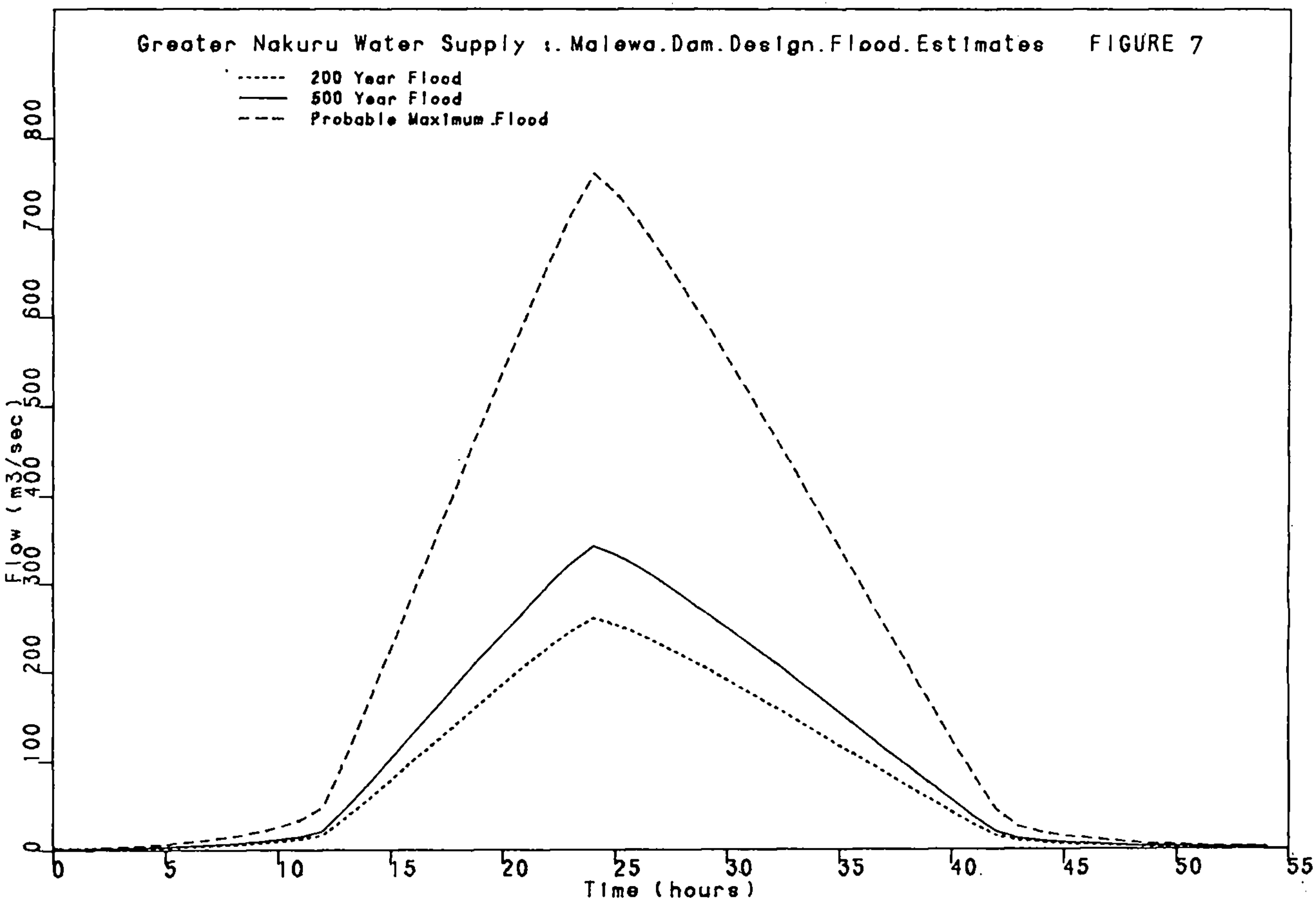
GREATER NAKURU WATER SUPPLY : MALEWA DAM DESIGN FLOODS

TABLE 8
Contd/

CONVOLUTION OF UNIT HYDROGRAPH AND NET RAIN PROFILE

TIME	TOTAL RAIN MM	NET RAIN MM	UNIT HYDROGRAPH ORDINATE	TOTAL HYDROGRAPH CUMECS
34.00				383.50
35.00				340.03
36.00				296.31
37.00				252.56
38.00				209.22
39.00				166.38
40.00				124.30
41.00				83.52
42.00				45.94
43.00				26.90
44.00				20.11
45.00				15.78
46.00				12.57
47.00				10.02
48.00				7.94
49.00				6.26
50.00				4.94
51.00				3.93
52.00				3.20
53.00				2.73
54.00				2.49

TOTAL FLOOD VOLUME (MILLION M3) 46.19


Greater Nakuru Water Supply :. Malewa.Dam.Design.Flood.Estimates
FIGURE 7
200 Year Flood
500 Year Flood
Probable Maximum.Flood
Flow (m3/sec)
0
100
200
300
400
500
600
700
800
0
5
10
15
20
25
30
35
40
45
50
55
Time (hours)

We recommend that the peak discharge of construction floods, with return periods up to 50 should be estimated from the bottom curve in Figure 3, and the estimate of $\bar{Q} = 47\ m^3/s$. The shape of the flood hydrograph can be estimated from a simple triangular unit hydrograph where the duration of the flood is 30 hours and the time to peak is 12 hours.

2.6 CONCLUSIONS AND RECOMMENDATIONS

A large number of assumptions have been made in this analysis to allow design floods to be estimated. The assumptions relate specifically to the time to peak of the hydrograph, the percentage runoff and the design rainfall profile. Where possible the assumptions have been based on local information and data; in the remaining cases they have been based on experience and judgement.

Given the limitations of the present data, we believe that the analysis described above, which combines elements of statistical analysis and unit hydrograph models, makes the best use of the available information. Nevertheless reliability of the flood estimates could be greatly improved by the collection of additional data. In particular the installation of a recording raingauge in the dam site catchment, and a river level recorder at or near the dam site would be valuable. Data from even one or two storms should enable the estimates of time to peak, rainfall profile and percentage runoff to be verified. We consider that installation and running costs of such instruments would be very small in comparison with the overall costs of the scheme; the benefits would be considerable.

WATER RESOURCES

An estimate of the available yield from the Malewa dam site is required for retention levels of 2135, 2140 and 2145 m and for return periods of failure of 10, 20 and 50 years. 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.

Gauging station 2GB5 is fairly close to the dam site on the same river so it has been assumed that an inflow sequence to the proposed reservoir is best estimated from the flow at 2GB5 scaled by the ratio of the catchment areas. Thus the inflow to the dam site will be estimated from 92 per cent of the flow at 2GB5. However there are only 21 years of data available for this station and this is not sufficient to determine a yield with a return period of failure of 50 years. This record must therefore be modelled and extended using a longer record.

3.1 EXTENSION OF FLOWS AT 2GB5

The model for the flow at 2GB5 can be either conceptually or statistically based. A conceptual model is probably the most difficult and least effective method of record extension in this case as it would require data to describe the process of runoff produced by rainfall including infiltration rates and the behaviour of the soil storage; these were not available.

The other approach for record extension is to develop a statistical relationship between the flows at 2GB5 and some other time varying parameter such as flow measurements from similar nearby catchments or rainfall records.

River gauging stations 2GC4 and 2GB1 have recorded data for 28 and 44 years respectively although records are missing for 2GB1 during 1977. There are also the two long term raingauges in the area at Gilgil and North Kinangop. A multiple regression was carried out to determine the equation which best described flow at 2GB5 using the series described above and the flow at 2GB5 lagged by one month.

The Regression Equation

The regression was carried out using a logarithmic transformation of all the data to reduce the effect of flood flows which would otherwise dominate the regression. It is more important to accurately model the low and medium flows when the series is to be used for reservoir design as the high flows will usually cause spillage but the low flows will define the critical periods. The use of logarithms also ensures that negative flows are not predicted for 2GB5 when the equation is used to produce a synthetic record.

The best fit for the regression equation was optimised on a computer by adding the independent variables one at a time to find the series which produced the highest correlation with 2GB5. The possible selection of variables was

Log of monthly flows at 2GB1 (L2GB1)

Log of monthly flows at 2GB1lagged by one month (L2GB1L1)

3. Log of monthly flows at 2GC4 (L2GC4)

4. Log of monthly flows at 2GB5 lagged by one month (L2GB5L1)

5. Rainfall from Kwetu Farm.

6. Rainfall from North Kinangop

Two regression analyses were carried out simultaneously, one including 2GC4 and one excluding it. As 2GC4 has only 7 years more data than 2GB5 the regression equation involving it would only be useful for the prediction of these 7 years. Therefore, as this equation was not much better than the regression excluding 2GC4 it was decided to predict the whole 23 year series with just one equation.

The regression equation thus chosen to best model flow at 2GB5 is

$$\mathrm{L2GB5} = -0.5427 + 0.898\,\mathrm{L2GB1} + 0.458\,\mathrm{L2GB5L1} - 0.342\,\mathrm{L2GB1L1}$$

This model seems reasonable, as the correlation of flows at 2GB5 should be much closer with other flow stations than with rainfall. This is because the conversion of rainfall to runoff is governed by a complex combination of several physical processes, the effect of which would have to be explained by the regression parameters. Conversely one would expect the monthly flow on a river to correspond well with the flow measured downstream and with the previous flow at that point.

The Predictive Mode

The explained variance of this equation was 86.7 per cent. This is a measure of the amount of variability in 2GB5 which is modelled by the equation. To truly represent the flow it is necessary to reproduce 100 per cent of the variability of the original series. Therefore when the equation is used in the predictive mode a stochastic element is added. This stochastic element is usually a random variable (ε), normally distributed, with a mean of zero and standard deviation of one. ε is scaled by the variability of the residuals of the regression, in this case we have a standard deviation of 0.374, to account for the unexplained variance of the regression.

The equation used for the synthesis of flows at 2GB5 is

$$L\,2GB5 = -\,0.5427 + 0.893L2GB1 + 0.458L2GB5L1 - 0.342L2GB1L1 + 0.374\,\varepsilon$$

This equation was used with the 23 years of data for 2GB1 from 1936 to 1958 to extend the flow series at 2GB5 to a 44 year record from 1936 to 1979.

When this equation is used the series of flows produced will depend on the stochastic input so that many different series could be produced merely by altering the random series. Each series is a possible flow series and is a sample of the whole suite of flow series. However it may constitute a biased representation of the flows. To guard against this it is necessary to produce several series of inflows and to carry out the storage yield analysis using each series (and then to pool the results to determine a more reliable estimate of the expected yield.) rewrite as below

The regression equation was used to synthesise nine series of possible inflow sequences to the reservoir which could then be incorporated as the basis for reservoir storage yield analyses. Tables — —

3.2 RESERVOIR STORAGE YIELD ANALYSIS

There are many methods for estimating the yield available for a particular storage but we believe that the most reliable methods are based on a reservoir operation procedure. This is usually carried out using a monthly water balance by routing the inflows through the reservoir, imposing the evaporation, rainfall and yield on the contents. Any further manipulation of these results is then based on the true behaviour that we can expect from the reservoir.

The problem is then to affix a return period of failure to the yield and storage for a particular reservoir. We have chosen the Gould Probability Matrix method (Ref 15) for this purpose and it is explained in detail in the appendix. Briefly, the reservoir is divided into N equal parts and a transformation matrix is calculated describing the probability of ending the year in any particular state conditional on its starting state. This is then combined with the probability of failing (from starting in any state) to produce the steady state probability of failure. WHM

This method relies on the assumption that there is no annual serial correlation in the flow data. A statistical analysis of the data for the area concluded that there was no evidence to suggest that annual serial correlation was present in the rainfall or corresponding runoff series.

3.3 RESULTS

The Gould method was carried out for each of the nine synthetic sequences using the monthly rainfall from the Gilgil station at Kwetu Farm and the monthly evaporation estimates from Table 3. Nine curves were produced for each retention level describing the return period of failure attributable to a particular yield. The mean of these curves was calculated and plotted in Figure 8. The yields available with 10, 20 and 50 year return periods are shown in Table 9 together with the standard deviation of the estimate which is a measure of the spread of the nine results.

A much greater error is introduced in the estimation of yields by the original choice of model; however, in this case, a regression on a monthly basis including nearby flow records should provide a reasonable model for the extension of flows.

3.4 CONCLUSIONS

Our estimates of the yields available for different retention levels are listed in Table 9 and plotted in Figure 9. Although the error involved in estimating the return period of failure of the yield from the inflows is

Malewa reservoir storage yield analysis

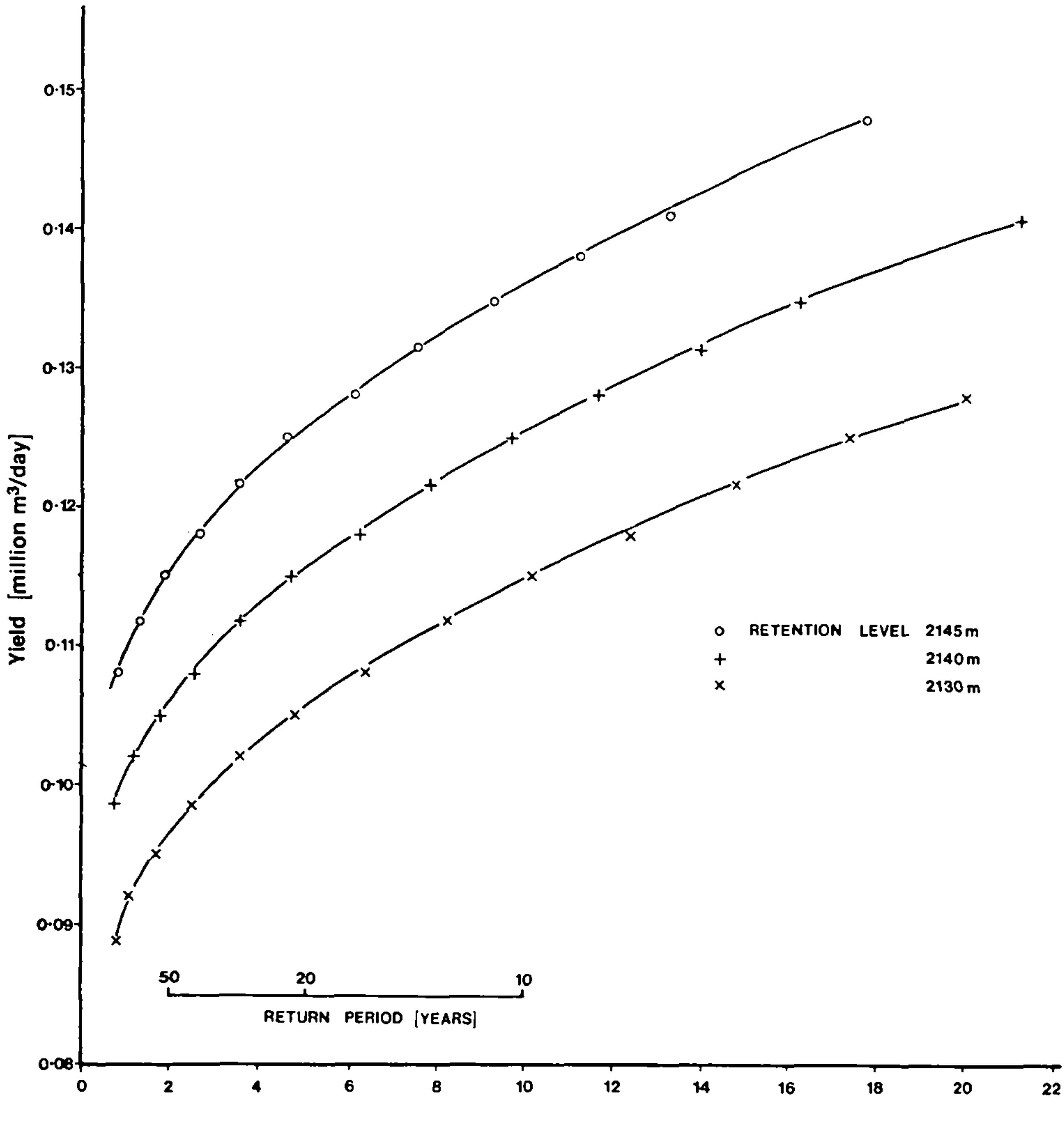


Figure 8

TABLE 9

RESULTS OF YIELDS FOR MALEWA RESERVOIR

RETENTION LEVEL (m)	STORAGE (million m^3)	YIELD AVAILABLE (Thousand m^3/day) RETURN PERIOD 10 yr	20 yr	50 yr
2145	59.0	136.2	125.6	115.6
2140	44.7	125.4	115.6	106.4
2135	33.6	114.8	105.4	96.4

Storage - yield curves for Malewa Reservoir

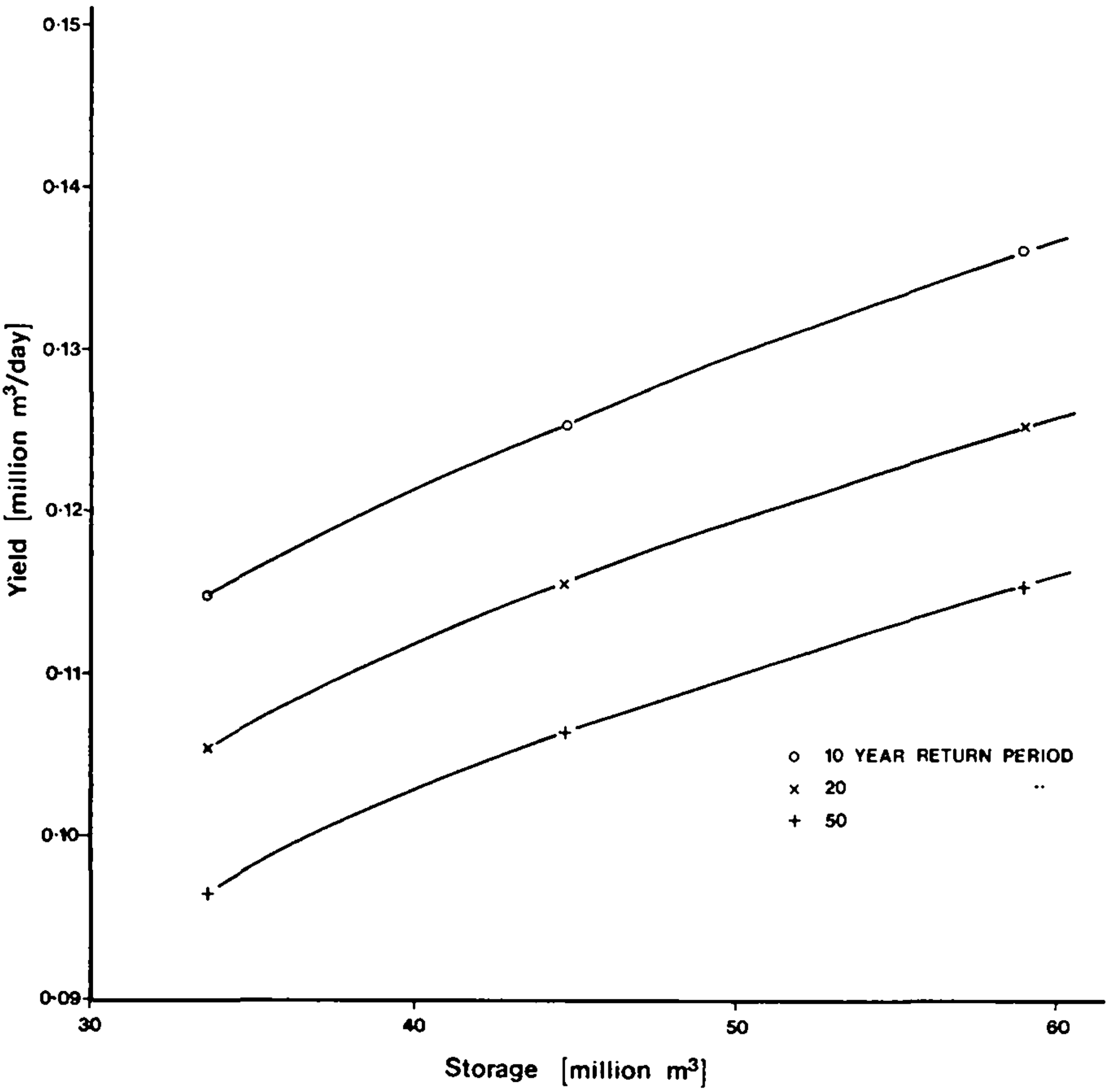


Figure 9

quite small the error expected in initially determining the inflow series is much greater. We believe that with the data that is currently available the approach we have adopted here will provide the best results possible.

The MOWD have just started collecting sediment samples from many locations in Kenya but there is not enough data available, at present, to calculate an estimate of the sedimentation rate likely in a dam situated at the Malewa site. Research in the upper Tana catchment has suggested a suitable estimate of 0.5 mm per year for this area (Ref 8). If this figure is used for the Malewa catchment a rate of sedimentation of 0.3 million m^3 per year or about 3 million m^3 every ten years would occur. This is only a very rough estimate of the sedimentation rate which is intended to give an order of magnitude to the possible sedimentation rates. If one considers the difference in soil type and topography in the upper Tana and the Melawa catchments this value may be rather conservative; however without data from the river Malewa it is very dangerous to attempt to estimate the sedimentation rate.

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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.