INSTITUTE of HYDROLOGY

LAIKIPIA WEST WATER SUPPLY

Nyarachi Dam

HYDROLOGICAL ANALYSIS

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This report is prepared for

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INTRODUCTION

The Nyarachi dam is located north of Nyahururu Falls on the river Nyarachi, a small tributary of the Ol Arabel river (Figure 1). From the dam site the river flows northwards for about 20 km before draining down the Laikipia Escarpment and into Lake Baringo. The catchment area to the dam site is 46 km^2, and is covered by a mixture of forest plantation and cultivated land. The forest is located mainly in the upper reaches of the catchment, and is being cut to support the local sawmills.

The objective of this analysis is to provide estimates of the expected yield from the reservoir with 1 in 10, 20 and 50 year probabilities of failure for three retention levels. Estimates of spillway design and construction floods have also been made.

There are no river gauging stations on the Nyarachi itself, so our analysis has had to be based on analogy with nearby catchments (Figure 1). However a temporary gauge had been set up by the Consultant just downstream of the dam site, and comprises a 'V' notch weir with readings taken from a simple staff gauge. Unfortunately the crest of the weir has been dammed with sandbags from time to time, so some of the readings are unreliable. It has not been possible to make use of the data from the gauge in this report.

The climate of the Nyarachi region has a seasonal pattern of wet and dry periods controlled by the movement of the intertropical convergence zone (ITCZ). In this equatorial region the ITCZ is a series of low pressure areas which form a belt parallel to the equator. This moves north and south following the movement of the sun, and this twice yearly movement together with the instability caused by the moisture bearing south-east and north-east monsoons produces rainfall. There are two periods of rainfall caused in this way, the 'long rains' (April and May) and the 'short rains' (October and November). In addition the development of local anticyclones cause the 'continental' rains of July and August.

It is these 'continental' rains that generally cause the maximum rainfall around Nyarachi, a pattern which is reflected in the runoff records. The mean annual rainfall is about 1000 mm, considerably less than the potential

Location map

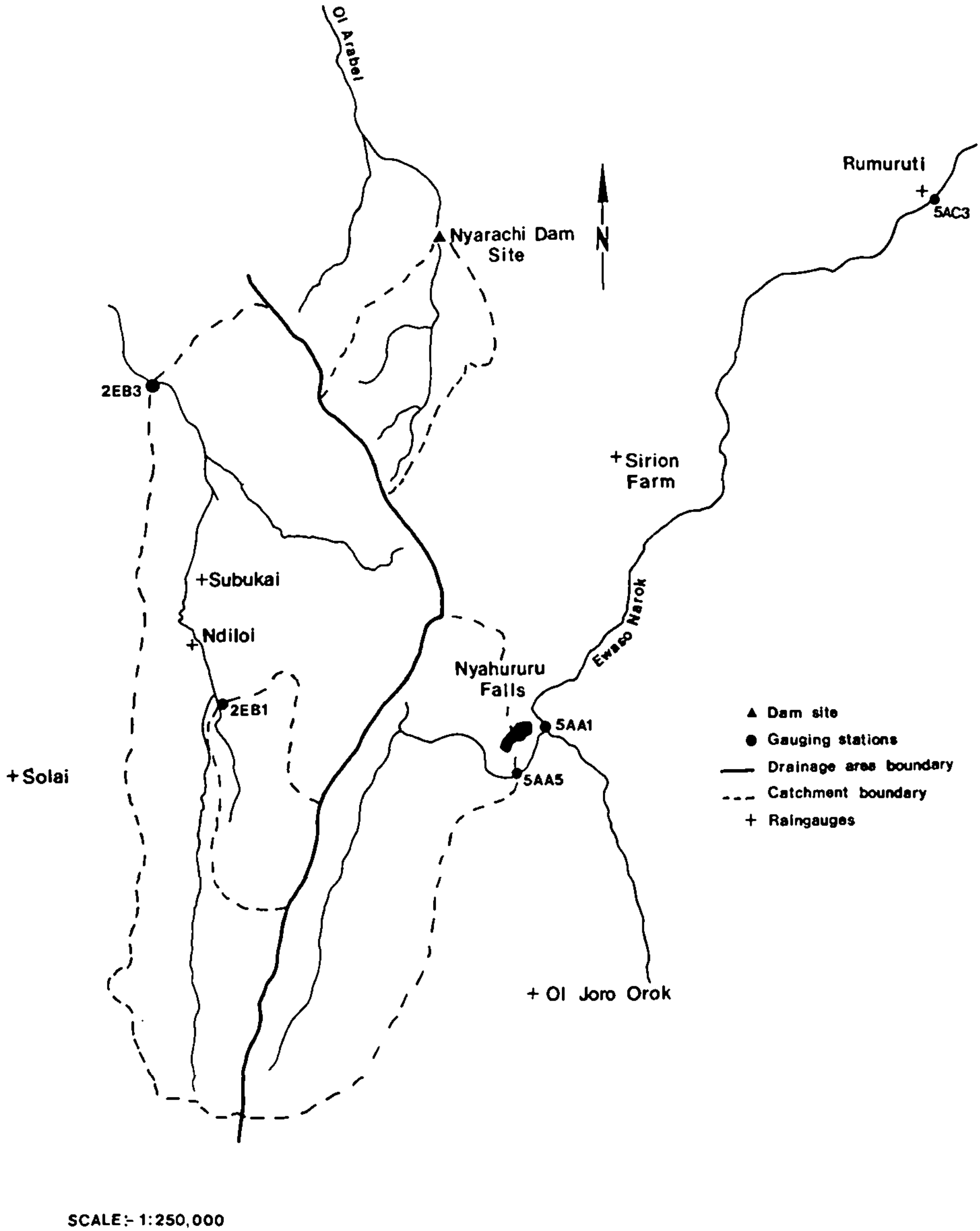


Figure 1

open water evaporation of about 1700 mm; flows in the dry season being maintained by local seepage from groundwater storages.

The Kenya Rift Valley was a region of intense volcanic and tectonic activity; consequently its geology, and the geology of the Nyarachi region, is extremely complex. The main feature of the area is the near horizontal laval flows occurring as phonolites. Weathering has produced well drained colluvial soils of varying depths.

1.1 AVAILABLE DATA

Rainfall

The locations of the main raingauges in the Nyarachi region are shown in Figure 1. Details of the records available are given in Table 1, which shows there are three stations with more than 40 years of record.

Copies of the monthly summaries for these stations were available at the MOWD, but these records were not completely up to date. Where appropriate these records were updated from the records at the Kenyan Meteorological Department.

The annual rainfall data for the long-term raingauge at Rumuruti were examined for persistence; the low lag-one serial correlation coefficient indicated that the annual data could be treated as independent events. However this record does include long periods of lower than average and higher than average rainfall. Figure 2, a plot of the cumulative departure of annual rainfall from the long-term mean, shows a dry period in the 1940's followed by a wetter period.

Two other studies (Refs 1,2) have analysed the recording raingauge data for Kenya : we have used these publications for rainfall intensity-duration-frequency information.

Runoff

There are no gauging stations on the Nyarachi river itself, so the water resources and flood analyses have been based on analogy with nearby catchments. The locations of these catchments and the main gauging stations

TABLE 1

RAINFALL RECORDS

Name	Station Number	Period of Record	Mean (mm)
Rumuruti D.O.	893601	1906-1980	680
Subukia Manyatta	893604	1923-1965	984
Thomson's Falls, Co-op	893615	1937-1980	958
Thomson's Falls, Sirion Farm	893630	1935-1980	913
Ndiloi	893634	1947-1973	1133
Solai	893663	1963-1979	1032
Ol Joro Orok	9036135	1946-1980	990

Laikipia West Water Supply * Nyarachi Dam
Figure 2
Rumuruti rainfall * Plot of cumulative departures from the mean
1000
500
0
-500
-1000
-1500
-2000
-2500
Sigma (R-BBAR) (mm)
1910
1920
1930
1940
1950
1960
1970
1980
Year

are shown in Figure 1; there are no gauges immediately to the north of the dam site.

The upper catchments of the Ewaso Narok in drainage area 5 are at roughly the same elevation as the Nyarachi catchment, and the vegetation is similar. In contrast the rivers of drainage area 2 are at a lower elevation below the Laikipia Escarpment which forms the boundary between the two drainage areas, and the vegetation is less abundant.

The gauging station 5AA5 is located at a rectangular section culvert under the main road some 2 km south of Nyahururu Falls. The siting of the gauge is not ideal, but the river channel appears to be reasonably stable and the approach sections are adequate. Stage readings are taken on average once or twice a day using a gauge attached to one of the abutments.

Although weirs have been built at the other gauging stations, 5AA1 is the only one equipped with a recorder. At 2EB3 daily readings are taken; at 2EB1 the frequency of readings is only once every 4 or 5 days. A recent field visit to the latter gauging station revealed that the top portion of the staff gauge is missing, and only low flows can be measured at present.

In common with many of the river gauging stations in Kenya, discharge measurements have been made only at low and medium stages. The rating curves are therefore based on a combination of weir equations and discharge measurements, and on discharge measurements alone where the gauging station is a natural river section. For high stages the ratings have been extended to cover the full range of stage by extrapolating plots of stage against discharge on logarithmic scales.

Flow records for these gauges were available from the MOWD files in the form of mean daily discharges and monthly summaries; the daily flows being calculated manually from the mean daily stage and the appropriate rating table. Recently the stage records have been punched on cards and are now stored on computer. It should now be possible to process these data automatically; at the time of this study this was not the case, and flow data for 5AA5 were reprocessed by the MOWD by hand following a revision to the rating curve. It was not possible to check the rating curves at the other stations and if necessary rework the data because of the time manual processing would have required.

Evaporation

A study of potential evaporation in Kenya was published by Woodhead in 1968 (Ref. 3). The results of his work have been used in this study. We consider that it is seldom worth updating a thorough evaporation study, purely because more recent data become available, because evaporation does not vary much from year to year. Moreover we were unable to obtain the most recent climatological data from which revised estimates of evaporation could be made using Penman's equation.

Estimates of the monthly potential open water evaporation from the reservoir surface were taken as the arithmetic mean of the values for Rumuruti, Ol Joro Orok and Subukia (Table 2).

Sedimentation

A summary of the sedimentation data available in Kenya was published in 1974 (Ref 4). The data compiled in that report were collected during the period 1948-65, and for the gauging stations where there are sufficient measurements, these are presented in the form of sediment-discharge rating curves. Several curves have been drawn for gauging stations in the Nyarachi region.

In order to use a sediment discharge curve to estimate the annual sediment load in a river it is also necessary to make use of a flow duration curve. Such curves were not available at the time of this study. Moreover the published data (Ref 4) strictly apply up to 1965. Since then there have been many changes in land use which are likely to have had a major influence on sedimentation rates.

Plans have been made recently by MOWD to undertake a co-ordinated programme of sediment measurements throughout Kenya (Ref 5), but the results of the programme are not yet available. We consider that this programme is essential and should be given the fullest support. In the meantime we

TABLE 2

ESTIMATED OPEN WATER EVAPORATION FROM THE RESERVOIR

(mm)

J	F	M	A	M	J	J	A	S	O	N	D	Total
153	155	172	141	144	129	124	133	149	153	136	141	1730

consider that the sediment data for the Nyarachi region are too limited to allow an objective assessment of the likely sedimentation rate in the reservoir to be made.

On the basis of evidence published so far, the overall annual sediment yield of the Upper Tana river is of the order of 0.5 mm (Ref 6). In the absence of other data, we consider that this is a reasonable figure to use for the Nyarachi catchment.

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 7) or for East Africa described in the Transport and Road Research Laboratory Report (TRRL) (Ref 8) 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 9) 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.

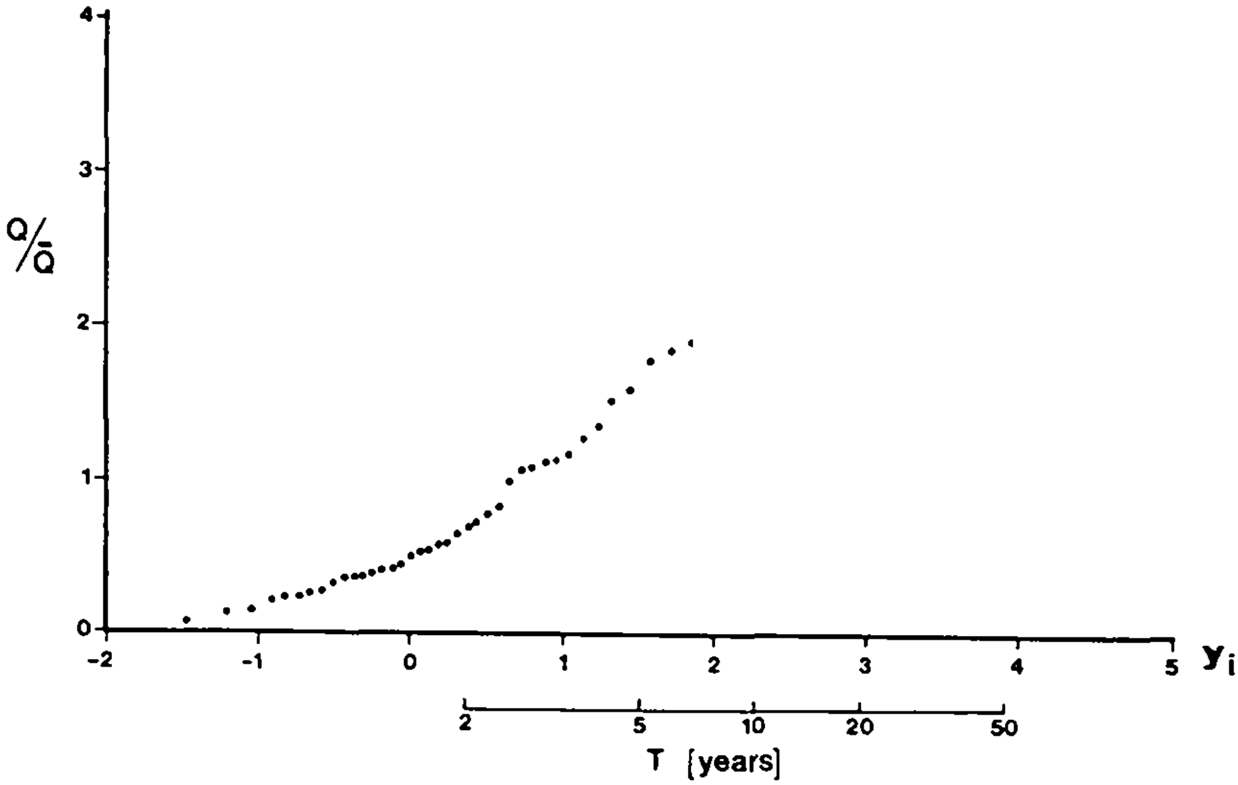
Rainfall data from autographic recorders in Kenya have been analysed and the results published in a convenient form for estimating design storms for given durations and frequencies (Ref 1,2). Insufficient short-term rainfall and runoff data are currently available for the dam site catchment to allow 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.

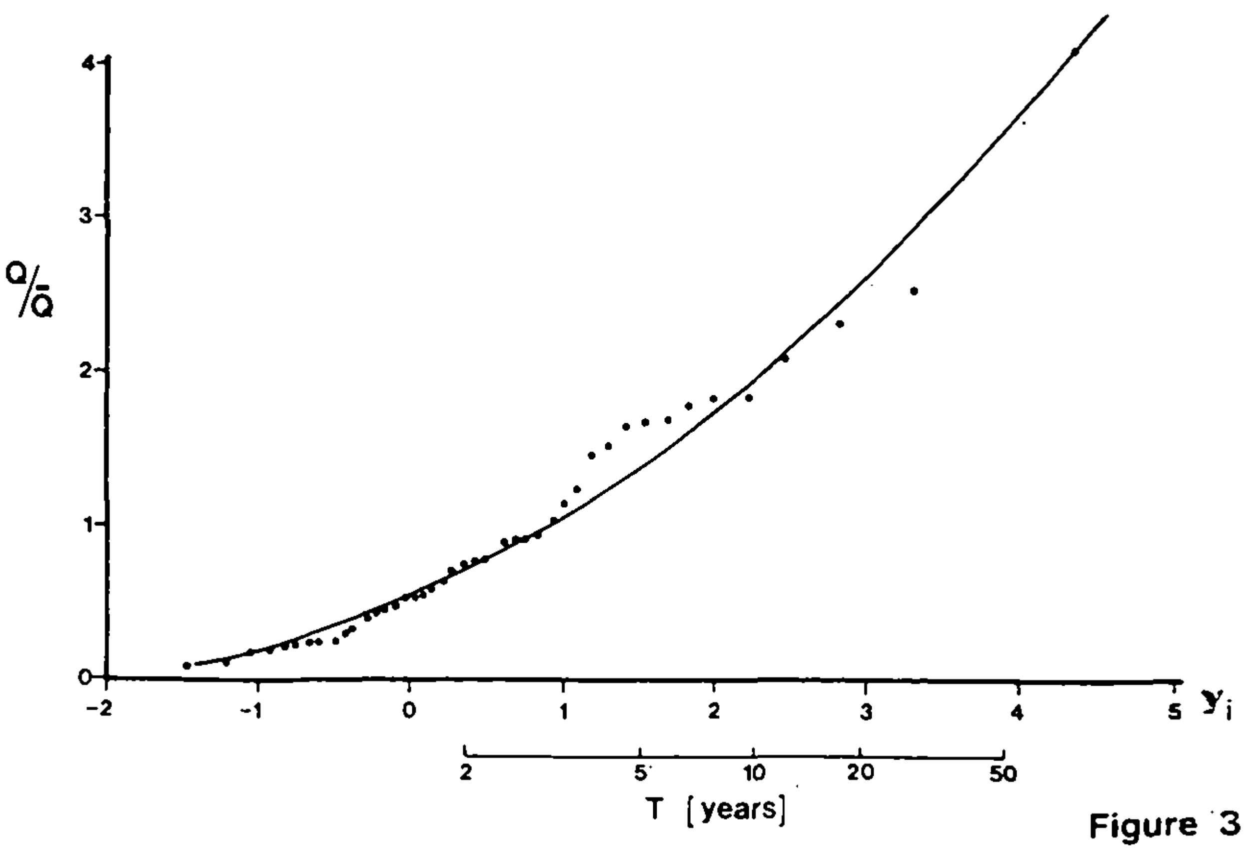
First a regional flood frequency curve was constructed using the local data available; these were annual maximum mean daily discharges. Experience from other parts of the world suggests that the dimensionless frequency distributions of instantaneous and mean daily peak discharges will be similar. This similarity is supported by data from river gauging station 2GB1 on the Malewa river located to the south east of Nakuru (Figure 3).

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



Instantaneous discharge

Figure 3

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 periods floods for spillway design.

2.2 DATA USED IN FLOOD ANALYSIS

The records of a number of gauging stations in the Nyarachi region were considered for possible inclusion in the floods analysis. At a number of stations, including 2EB1, stage readings are taken regularly but only once every three or four days. Such stations, and others where the rating curves appeared to be suspect, were excluded from the analysis. For the remaining stations shown in Figure 1, the annual maximum mean daily flows were extracted from the MOWD files; no charts 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 1) and the TRRL design manual (Ref 2).

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 all the gauging stations shown in Figure 1 except 2EB1. 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 $y = 3.1$. 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 112 events, 112 being the total number of events in the pooled record.

The resulting curve is shown in Figure 4.

Pooled flood frequency curve

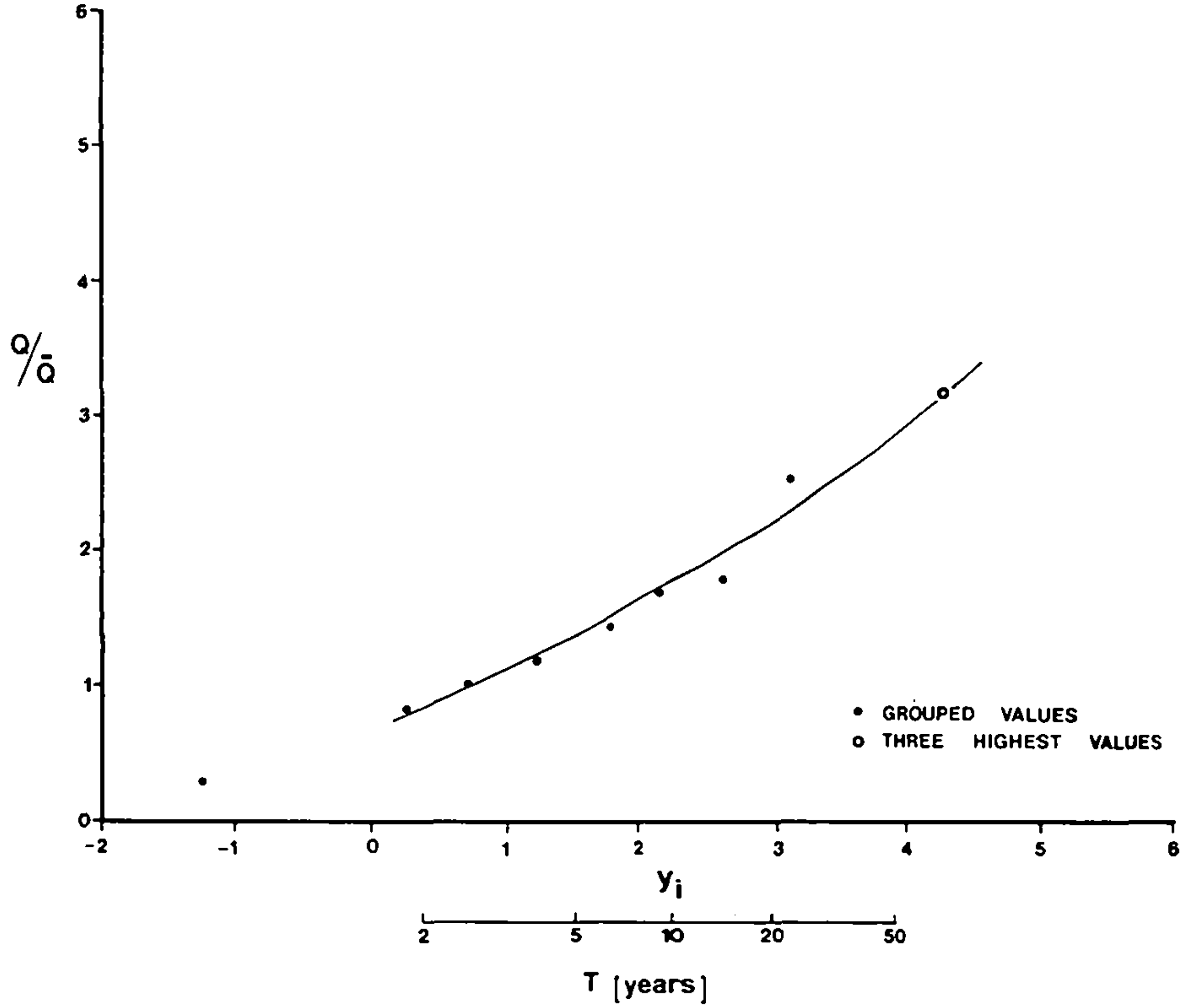


Figure 4

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 2, 8) 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 11 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.

TABLE 3--

SUMMARY OF TRRL METHOD PARAMETERS

Streamlength	12.4 km	
Catchment area	46 km^2	
Land slope	9.5%	
Channel slope	2.02%	
S1085	17.5 m/km	Source
2 year daily point rainfall	55 mm	TRRL 623 Figure 1
Areal reduction factor	.88	" 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 Nyarachi 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 5 hours

T_B 13 hours

Q_p 44 $m^3/s/100\ km^2$

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 1). Using the curves and maps together, the 24 hour rainfalls for the Nyarachi 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.

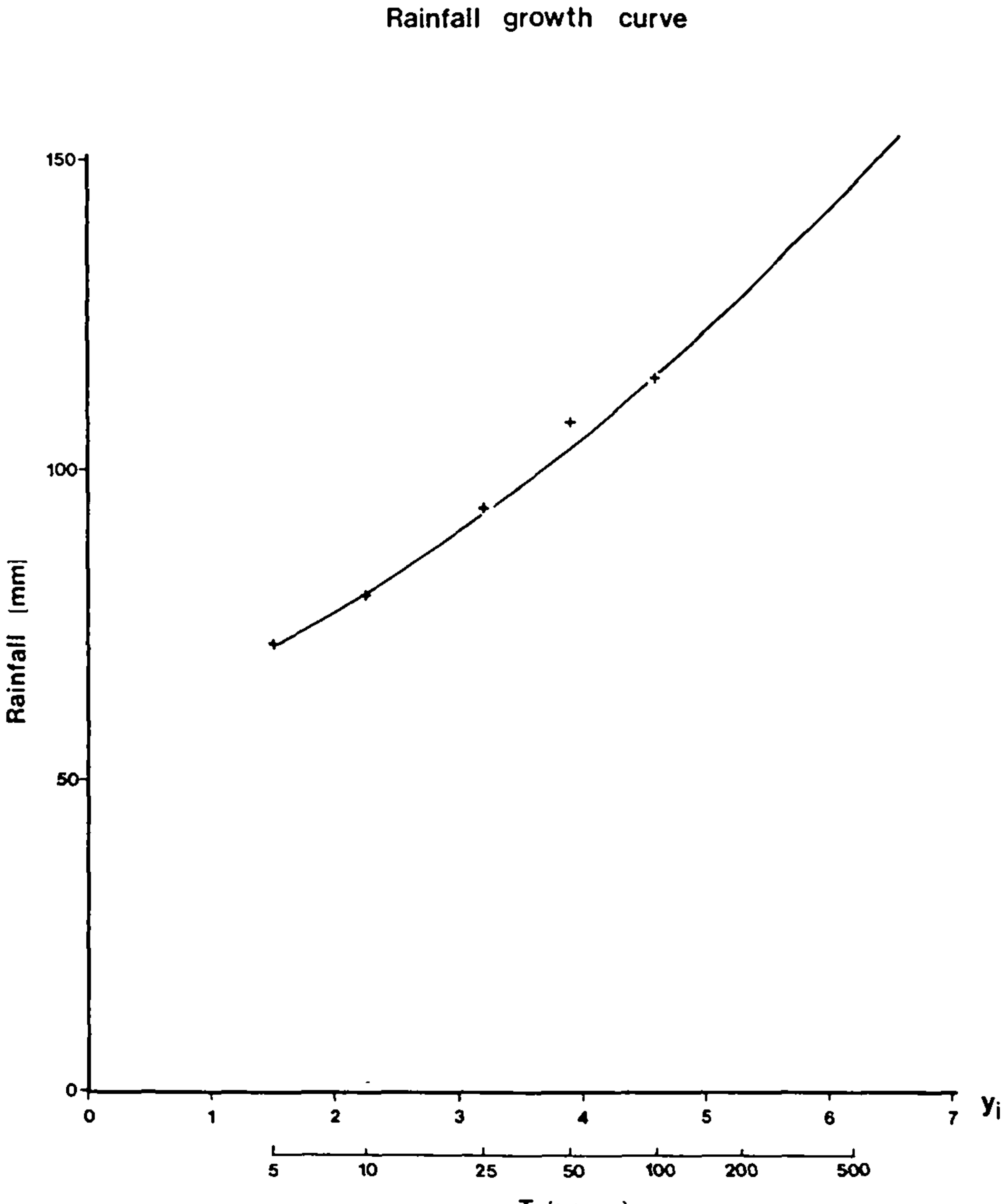

Rainfall growth curve
150
100
50
0
Rainfall [mm]
0
1
2
3
4
5
6
7
y_i
5
10
25
50
100
200
500
T [years]

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 2), and in the absence of other data, it has been assumed that for this basin an ARF of 0.88 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

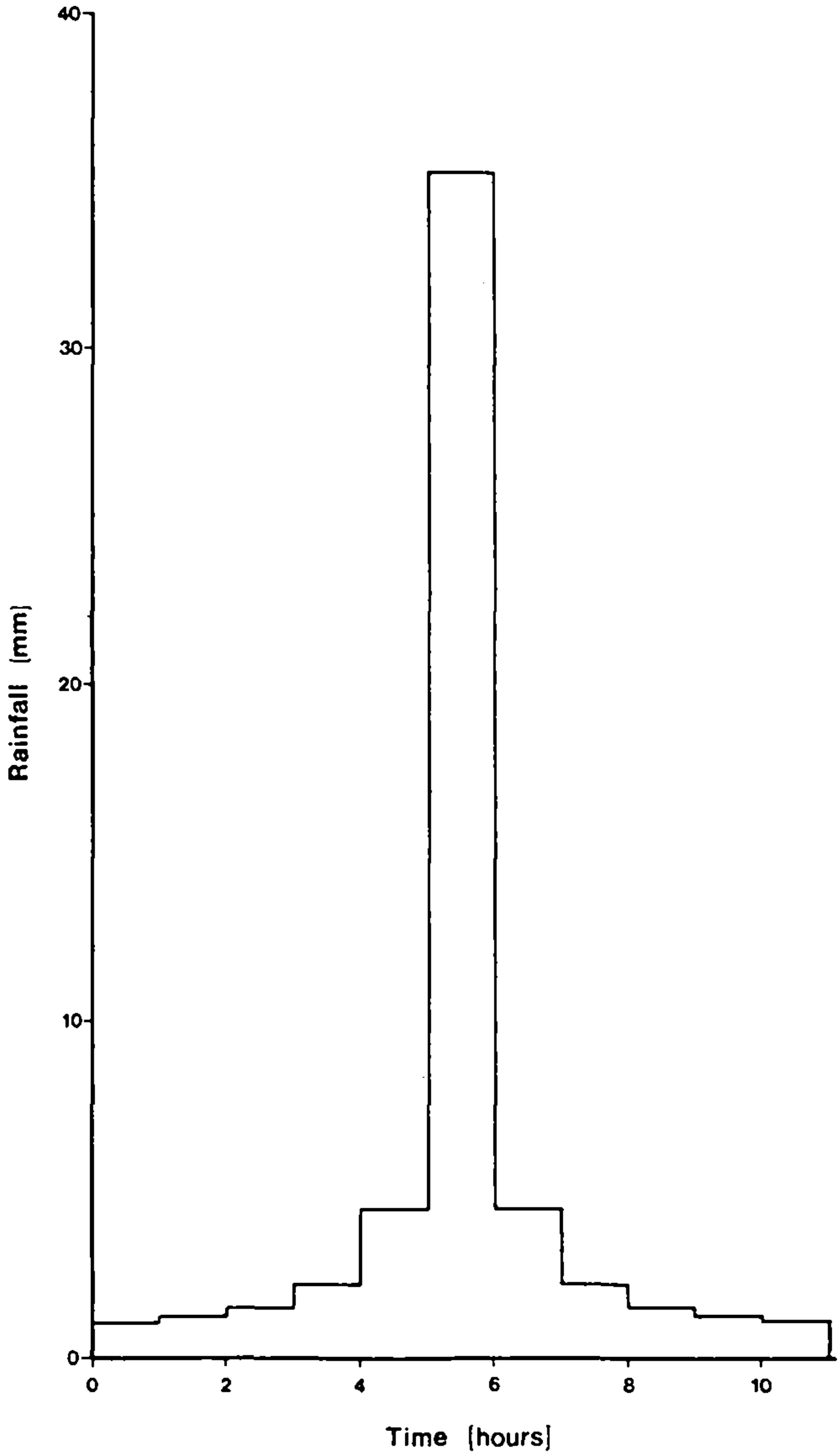


Figure 6

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 25 m^3/sec,

DESIGN FLOOD PARAMETERS

Return Period (years)	Rainfall (mm)	Percentage Runoff (%)	Volume ($m^3 x 10^6$)	Q_{max} (m^3/s)
5	56.1	13	.55	15.2
10	65.7	15	.67	19.8
25	73.3	17	.79	24.3
50	84.2	19	.96	30.5
100	89.9	21	1.09	35.6

Unit hydrograph model: Flood predictions

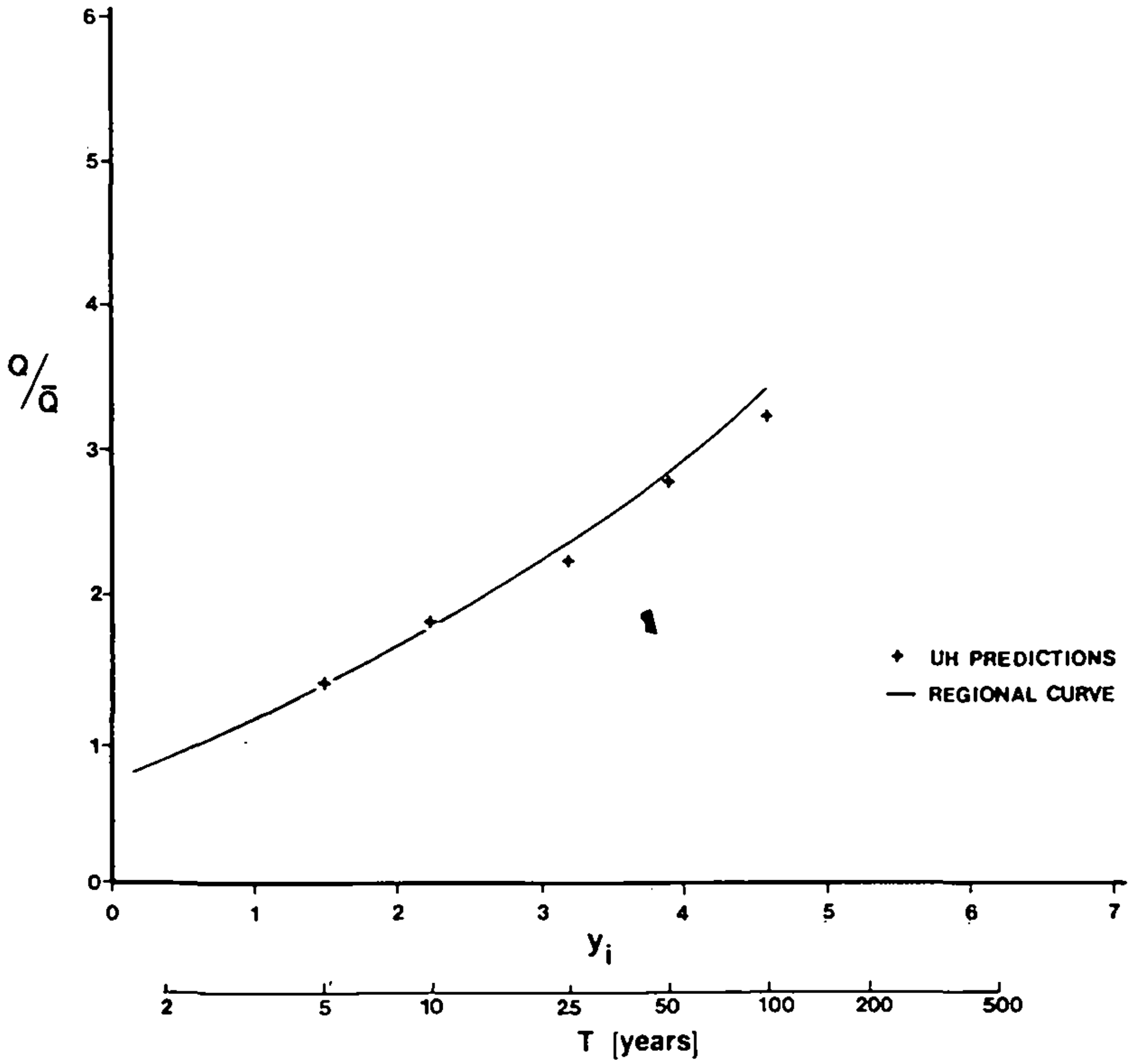


Figure 7

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 $\bar{Q} = 11\ m^3/sec$ 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 13 hours and the time to peak is 5 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 Nyarachi 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.

Laikipia West Water Supply : Nyarachi Dam Design Floods TABLE 5

200 Year Flood

Area (Sq.Km.)	46.00
Data interval (Hr)	1.00
Design duration (Hr)	11.00
Total rain (mm)	99.47
Percentage runoff	24.00
Base flow (cumecs per sq.km)	.05950

Triangular unit hydrograph computed from Tp= 5.0

Convolution of unit hydrograph and net rain profile

Time	Total Rain mm	Net Rain mm	Unit Hydrograph ordinate	Total Hydrograph cumecs	
.00	1.81	.44	.00	2.74	
1.00	2.18	.52	8.80	2.91	
2.00	2.77	.66	17.60	3.30	
3.00	3.90	.94	26.40	3.96	
4.00	7.88	1.89	35.20	4.99	
5.00	62.40	14.98	44.00	6.79	
6.00	7.88	1.89	38.25	14.37	
7.00	3.90	.94	32.50	22.35	
8.00	2.77	.66	26.75	30.27	
9.00	2.18	.52	20.99	37.84	
10.00	1.81	.44	15.24	44.35	-Peak-
11.00			9.49	41.01	
12.00			3.74	36.41	
13.00				31.22	
14.00				25.70	
15.00				19.99	
16.00				14.19	
17.00				8.73	
18.00				4.98	
19.00				3.97	
20.00				3.38	
21.00				3.02	
22.00				2.81	

TOTAL FLOOD VOLUME (MILLION M3) 1.33

Laikipia West Water Supply : Nyaracni Dam Design Floods TABLE 6

500 Year Flood

Area (Sq.Km.)	46.00
Data interval (Hr)	1.00
Design duration (Hr)	11.00
Total rain (mm)	114.14
Percentage runoff	27.00
Base flow (cumecs per sq.km)	.06006

Triangular unit hydrograph computed from Tp= 5.0

Convolution of unit hydrograph and net rain profile

Time	Total Rain mm	Net Rain mm	Unit Hydrograph ordinate	Total Hydrograph cumecs	
.00	2.08	.56	.00	2.76	
1.00	2.50	.67	8.80	2.99	
2.00	3.18	.86	17.60	3.49	
3.00	4.47	1.21	26.40	4.34	
4.00	9.04	2.44	35.20	5.68	
5.00	71.60	19.33	44.00	8.00	
6.00	9.04	2.44	38.25	17.77	
7.00	4.47	1.21	32.50	28.08	
8.00	3.18	.86	26.75	38.31	
9.00	2.50	.67	20.99	48.07	
10.00	2.08	.56	15.24	56.48	-Peak-
11.00			9.49	52.17	
12.00			3.74	46.23	
13.00				39.53	
14.00				32.41	
15.00				25.04	
16.00				17.55	
17.00				10.50	
18.00				5.65	
19.00				4.36	
20.00				3.60	
21.00				3.12	
22.00				2.86	

TOTAL FLOOD VOLUME (MILLION M3) 1.65

LAIKIPIA WEST WATER SUPPLY : NYARACHI DAM DESIGN FLOODS   TABLE 7

PROBABLE MAXIMUM FLOOD

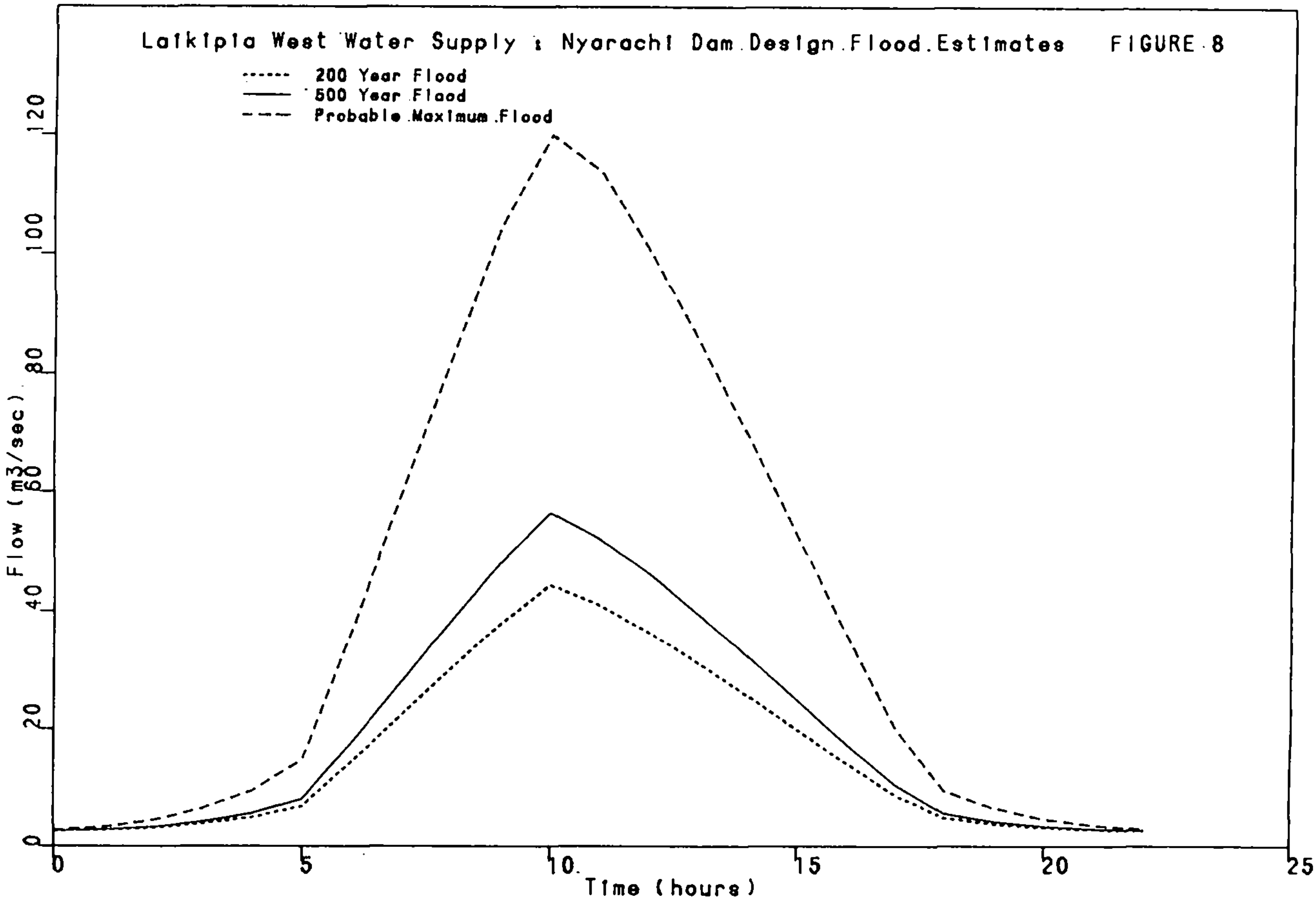
AREA (SQ.KM.)	46.00
DATA INTERVAL (HR)	1.00
DESIGN DURATION (HR)	11.00
TOTAL RAIN (MM)	197.67
PERCENTAGE RUNOFF	35.00
BASE FLOW (CUMECS PER SQ.KM)	.06316

TRIANGULAR UNIT HYDROGRAPH COMPUTED FROM TP= 5.0

CONVOLUTION OF UNIT HYDROGRAPH AND NET RAIN PROFILE

TIME	TOTAL RAIN MM	NET RAIN MM	UNIT HYDROGRAPH ORDINATE	TOTAL HYDROGRAPH CUMECS	
.00	3.60	1.26	.00	2.91	
1.00	4.33	1.51	8.80	3.42	
2.00	5.50	1.93	17.60	4.54	
3.00	7.75	2.71	26.40	6.44	
4.00	15.65	5.48	35.20	9.44	
5.00	124.00	43.40	44.00	14.66	
6.00	15.65	5.48	38.25	36.61	
7.00	7.75	2.71	32.50	59.75	
8.00	5.50	1.93	26.75	82.71	
9.00	4.33	1.51	20.99	104.63	
10.00	3.60	1.26	15.24	123.49	-PEAK-
11.00			9.49	113.82	
12.00			3.74	100.48	
13.00				85.44	
14.00				69.46	
15.00				52.92	
16.00				36.11	
17.00				20.27	
18.00				9.40	
19.00				6.49	
20.00				4.78	
21.00				3.72	
22.00				3.12	

TOTAL FLOOD VOLUME (MILLION M3)   3.44


Laikipia West Water Supply : Nyarachi Dam Design Flood Estimates
FIGURE 8
200 Year Flood
500 Year Flood
Probable Maximum Flood
Flow (m3/sec)
0
20
40
60
80
100
120
0
5
10
15
20
25
Time (hours)

3. WATER RESOURCES

The surface water analysis for the Nyarachi catchment has been based on data collected in other catchments under similar climatological and topographic conditions as there are no available runoff records for the Nyarachi river.

At first sight there appeared to be little to choose between the records from 5AA5 and 2EB1 as the basis for this work. However it soon became apparent that there were advantages in using 5AA5, the most important being that stage readings are taken once a day rather than once every 4 or 5 days as at 2EB1. This frequency of observations at 2EB1 is inadequate because for such a small catchment the response to rainfall will be rapid and the peak of the runoff will often not be recorded.

Catchment 2EB1 is situated on the western slope of the Laikipia escarpment whilst 5AA5 and Nyarachi are on the eastern side. The shape of the slopes are rather different on each side of the range and the rainfall generally moves from the east to the west so that the Nyarachi catchment rainfall and response should be better represented by 5AA5.

There has also, in the past, been some confusion about the actual location and catchment area of 2EB1. The National Water Master Plan (Ref. 11) quotes 46 km^2, the figure that was used in the pre-feasibility investigations (Ref 12). Recently it has been confirmed that the area is about 35 km^2, a figure that appeared in earlier MOWD files. Yield estimates calculated using the larger catchment area, and therefore lower specific runoff, will have been underestimated by about 30 per cent.

3.1 EXTENSION OF RUNOFF RECORD

The 22 year flow record from catchment 5AA5 was used to estimate a synthetic flow record for Nyarachi from 1959 to 1980 by a translation of the data using the ratio of the catchment areas. This length of record is insufficient to be able to define a yield with return period of failure of 50 years and, more importantly, the long term rainfall records show that since 1950 there has been a period of higher than average rainfall (Figure 2). Therefore to base the yield analysis on flow records of this period alone would tend to overestimate the water resources available.

As there are no suitable long term flow records in the area, the extension of the 22 year flow records had to be based on a rainfall-runoff relationship. The nearest long term raingauge in the area is at Rumuruti where monthly records have been collected since 1906 (with 5 missing years from 1913 to 1917).

Conceptual models are sometimes used to predict the runoff expected from rainfall inputs, but these require a considerable amount of good quality data including percentage runoff, infiltration rates etc. As these data are not available a simple regression model was developed.

The regression analysis was carried out between Rumuruti and the flow at 5AA5 on an annual basis. It is not feasible to use the catchment rainfall for this regression as the extension of the runoff will depend on the early Rumuruti rainfall alone, thus the relationship for the extension must be based on Rumuruti.

We have chosen to carry out the regression using a logarithmic transformation for both the rainfall and runoff series. This has several advantages over the use of the natural series in that the emphasis is taken away from the floods in the series which would otherwise tend to dominate the regression. This is particularly important for reservoir yield analysis as the accurate prediction of low flows is more pertinent to the analysis than the floods which would tend to spill. The use of logs also ensures that there are no negative flows predicted by the regression equation which is not the case with the natural series. Finally, from trials of the regressions, the synthesized flows from the logarithmic regression equation more closely fitted the distribution of actual flows than those from the regression of the natural series.

The equation of best fit, estimated from the available 22 years of overlapping rainfall and runoff data, is

$$\text{Log(Runoff)} = -4.02 + 1.87\ \text{Log (Rainfall)}$$

with a correlation coefficient r^2 of 74.4 per cent. When this equation is used in the predictive mode the variability of the runoff estimated will be less than the expected true variability of the series as it assumes that all the runoff values will be perfectly related to rainfall.

The variability of the synthesized data has to be increased and this is done by adding a stochastic element to the regression equation. The stochastic element is scaled so that it reflects the series of residuals formed by the regression on the available series. Thus a random variable, distributed with a mean of zero and standard deviation of one, is added to the prediction equation and is scaled according to the standard deviation of the residuals of the regression.

The prediction equation is, therefore,

$$\text{Log (Runoff)} = -4.02 + 1.87 \text{ Log (Rainfall)} + 0.132\,\zeta$$

$$(\text{where } \zeta \sim (0,1))$$

This equation was used, together with the 69 years of Rumuruti rainfall, to produce a series of 69 years of annual runoff for station 5AA5. The annual runoff values were distributed in a monthly pattern according to the monthly distribution of flows recorded at 5AA5. A regression, carried out on a monthly basis, would have had a small correlation coefficient and the prediction equation would have been dominated by the stochastic element.

The series of flows, predicted from the Rumuruti rainfall, is just a sample of the whole suite of series that can be predicted from this equation. This is because each time the prediction is carried out the stochastic element will be different and thus the predicted series will be different. In cases where the stochastic element is very small and the two variables are very closely related the effect of this difference can be negligible but in this case we believe that it is necessary to look at the effect of the stochastic element on the results of the analysis. For this purpose 9 different series of possible flows at Nyarachi were calculated, each using the same Rumuruti rainfall data but starting with a different seed for the random variable. It was proposed to carry out the storage yield analysis for the reservoir with each of the nine series of flows, and to take the average of the results to achieve an unbiased estimate of the yield.

3.2 RESERVOIR STORAGE YIELD ANALYSIS

The requirements for the storage yield analysis are to provide estimates of the yield available, with 10, 20 and 50 year return periods of failure, from a reservoir with retention levels of 2160, 2165, 2170 and 2175 m. For the purpose of this analysis it has been assumed that a yield with a T-year return period of failure is a yield which a reservoir can provide with a failure (of unspecified length of time) occurring, on average, once every T years.

There are many methods available for the estimation of reservoir yield, but the most reliable are based on a reservoir routing procedure. This allows the incorporation of time varying rainfall, evaporation and demand in the monthly water balance therefore providing realistic conditions under which the yield can be determined. The return period of failure of the yield has then to be estimated from the behaviour of the reservoir supplying that yield. We have chosen the Gould probability matrix method (Ref. 13) to assign a value to the return period of failure for a particular yield and a particular reservoir size. This method relies on the assumption that there is no serial correlation in the annual flows. There was no evidence to suggest that there is serial correlation in either the annual rainfall or the corresponding annual runoff series.

The Gould method is described in detail in the appendix to this report. The reservoir routing was carried out using the mean monthly evaporation from Table 2 and an estimate of the catchment rainfall for the period of synthesized flow record. The catchment rainfall was determined from Rumuruti rainfall multiplied by the ratio of mean annual rainfall for the Nyarachi area and the corresponding Rumuruti records.

The Gould method was employed with each of the nine series of data to produce nine curves describing the return period of failure attributable to a yield for a particular retention level. The mean of these results have been drawn in Figure 9 and the yields available with return periods of failure of 10, 20 and 50 years are shown in Table 8 together with their associated standard deviations. The standard deviation is a measure of the spread of the nine sets of results and we can expect the yields to be correct to within plus or minus twice the standard deviation to account for the error caused by the stochastic element. The possible variation

Nyarachi reservoir storage yield analysis

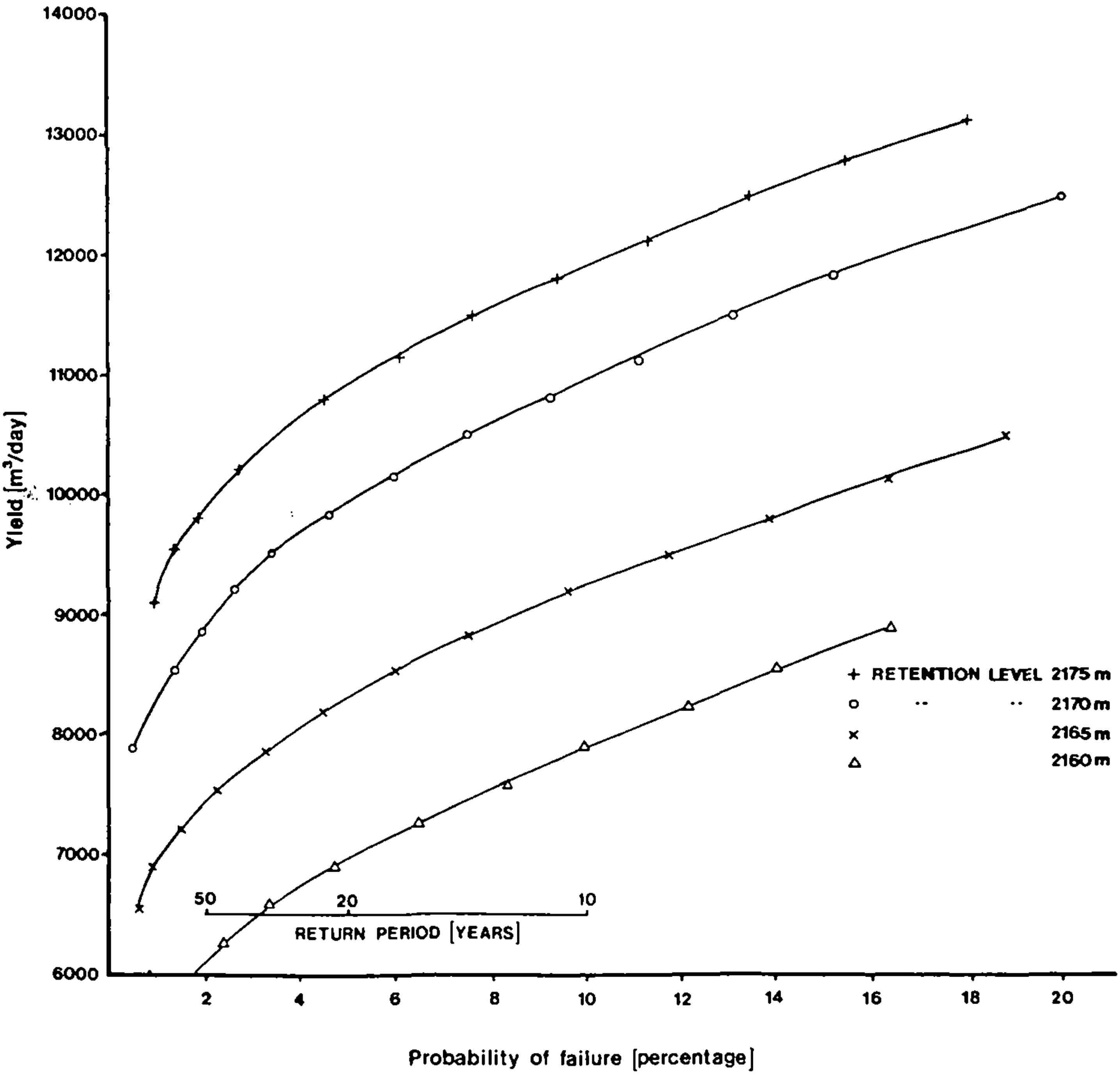


Figure 9

RESERVOIR STORAGE YIELD ANALYSIS RESULTS

RETENTION LEVEL (m)	10 YEAR RETURN PERIOD YIELD (m^3/d)	20 YEAR RETURN PERIOD YIELD (m^3/d)	50 YEAR RETURN PERIOD YIELD (m^3/d)
2160	7880 (78)	6950 (50)	6080 (40)
2165	9250 (230)	8325 (260)	7425 (260)
2170	10950 (360)	9925 (260)	8875 (230)
2175	11900 (330)	10925 (300)	9900 (260)

Values in brackets are the standard deviation of the yields immediately above.

in the annual flows will, of course, be much higher than this as there is a much larger error associated with fitting the correct model in the first place.

3.3 CONCLUSIONS

The yields available for different return periods of failure and different retention levels are listed in Table 8 and plotted in Figure 10. Although the error involved in estimating the return period of failure from the flows is quite small the error in estimation of the flows because of choice of model will be much greater. However with no available data for the Nyarachi catchment and only limited data for nearby catchments we believe that this is the most reliable method of yield estimation with the available information.

The curves in Figure 10 have been extrapolated to indicate the likely yield available for lower retention levels than 2160 m; however the uncertainty involved with the effect of sedimentation rates on reservoir designs of this size casts doubts on the utility of the results.

The analyses so far have assumed a sediment free flow but sediment deposition is an important factor in the design of reservoirs in Kenya. There is virtually no data on which to base an estimation of the sedimentation rate likely in this reservoir; however, research in the Upper Tana catchments has suggested a suitable estimate of 0.5 mm per m^2 per year for this area (Ref.4). Using this figure for Nyarachi a sedimentation of about 1 million m^3 would be expected after 50 years of reservoir operation. This estimate is very inaccurate and is put forward here as a guide to the possible rate of sedimentation. The rate will of course be dependent on the land use management of the catchment, particularly the extent of forest removal and also 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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