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The TRRL East African flood model

by

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ABSTRACT

Four years of data from 13 small representative rural catchments in Kenya and Uganda were analysed to develop improved methods of flood estimation for highway bridges and culverts. Due to the short period of record and the very quick response time of the catchments, Unit Hydrograph techniques were found inappropriate. A technique which made better use of limited data, therefore, had to be developed. Rainfall and runoff were fitted to a simple three parameter conceptual catchment model. The model was then used to predict the 10 year flood using a 10 year design storm. A simple technique is then developed for predicting the peak flow and base time of design hydrographs for ungauged catchments.

1. INTRODUCTION

A large proportion of the total cost of building a road in East Africa is for the construction of bridges and culverts to cross streams from small catchments[1]. Whereas most of the larger rivers in East Africa have flow measuring stations, very few smaller streams are so equipped. Design methods must therefore be based on rainfall-runoff models.

Very few data are available for the development of suitable flood models. In 1966 the Kenya and Uganda Governments and the UK Transport and Road Research Laboratory co-operated in a project to instrument 12 representative catchments, results from which could be used to develop improved flood design methods. The choice of location and instrumentation of these catchments is described elsewhere[2]. This report describes the programme of analysis of results and the design method that was developed. It was presented as a paper to a Flood Hydrology Symposium in Nairobi, Kenya in October 1975 which was jointly sponsored by the Economic Commission for Africa, the East African Community and the Transport and Road Research Laboratory.

2. CATCHMENT MODELS

A number of possible rainfall-runoff models were considered.

2.1 The unit hydrograph

A unit hydrograph is a hydrograph of unit volume resulting from a rainstorm of unit duration and specified areal pattern. Hydrographs for other storms of similar areal pattern can be constructed by superimposing hydrographs, suitably offset in time with ordinates proportional to the flow assumed to result from the rainfall in each unit time period.

It will be seen that the unit hydrograph simply distributes the runoff in time. The volume of runoff must be estimated separately and this generally involves deriving a rainfall-runoff correlation.

The simplest rainfall-runoff correlation is a plot of average rainfall over the catchment and resulting runoff. Typically the relationship is slightly curved indicating a somewhat higher percentage runoff with higher rainfall totals. The scatter of the points about the regression line is often large but may be decreased by introducing additional variables such as antecedent catchment wetness and intensity of rainfall.

The difference between the total rainfall and runoff is assumed to be water held on the surface of the ground or vegetation, which subsequently evaporates, or infiltration which does not appear in the stream flow until the storm runoff has effectively finished. These losses tend to be greater at the start of the storm, although to simplify the estimation of the rainfall input to the unit hydrograph they are often assumed to occur at a constant rate.

Their acceptance as standard practice shows that such methods have been remarkably successful, considering their simplicity, but they have two drawbacks for the present study.

(a) An adequate rainfall-runoff correlation requires a large number of storm data including many producing high flows.

(b) Storms that can be used to derive unit hydrographs, ie high intensity storms of unit duration, are rare, particularly on small catchments where the unit time is short compared to the typical storm length.

A method where more effective use can be made of the limited data that can be collected in a few years was therefore required for this study.

2.2 Conceptual models

A water balance for a catchment may be envisaged of the form

$$R = P - E - \Delta M - \Delta G \qquad \ldots 1$$

where R = runoff
P = precipitation
E = evaporation
ΔM = change in soil moisture
ΔG = recharge to deep storage

If records are available for precipitation, models can be set up to compute a running sequence of values for the other terms on the right hand side of the equation thus giving a running sequence of runoff values which can be suitably distributed in time. The best known example of this type of model is the Stanford watershed model[3].

Complicated computer programs and considerable data for calibration purposes are required which make this approach inappropriate for the present study.

2.3 Analogue models

The similarity between the response of a catchment to storm rainfall and the flow through a series of reservoirs has been noted by many workers. Zoch suggested the concept of linear reservoir routing as early as

1934. The response of cascading flows through a number of linear reservoirs both in series and parallel was studied by Sugawara and Marayama[5].

The purpose of this study is to develop a model which can be applied to ungauged catchments using very few measurements of catchment characteristics such as area, slope, soil type. The model must therefore be simple and have a limited number of parameters. The lumping or averaging of certain catchment mechanisms inherent in a simple reservoir model is thus not a disadvantage. This was therefore considered to be the most suitable type of model for the East Africa study.

3. THE TRRL FLOOD MODEL

3.1 Description of model

The model is made up of two parts. A linear reservoir model is used for the period between the rain hitting the ground surface and the floodflow entering the stream system (the "land phase") and a finite difference routing technique is used for the passage of the flood wave down the river to the catchment outfall.

A reservoir is said to be linear if the outflow (q) is related to the water stored in the reservoir by the linear relationship

$$q = \frac{1}{K} S \qquad \ldots 2$$

where S is the reservoir storage
K is the reservoir lag time

The flow from a linear reservoir with zero inflow decreases exponentially. The lag time may be conveniently thought of as the time required for the recession curve to fall to approximately $\frac{1}{3}$ of its initial value.

The reservoir storage contains the rain falling during the storm which contributes to the flood hydrograph. This can either be surface runoff or rapid subsurface runoff (interflow). Runoff does not occur uniformly over a catchment. Parts of the catchment are less permeable than others due to variation in soil type, or, in low lying areas, to slower drying out after previous rain. A uniform runoff coefficient can therefore be misleading and recently the concept of catchment contributing area (C_A) has been substituted[6].

Between storms, drying of the soil takes place due to the combined effects of evaporation and plant transpiration. The drying takes place principally in the surface layers. This accounts for the high infiltration rate at the start of a storm and also for the lack of subsurface runoff until this loss has been made good by infiltration. This is modelled by a storage called initial retention (Y), which must be filled before flood runoff occurs.

The simple land phase model may therefore be summarised as:

(a) Early rain fills the initial retention (Y). Runoff at this stage is zero.

(b) Subsequent rain falling on the parts of the catchment from which runoff will occur (C_A) enters the reservoir storage.

(c) Runoff is given by equation (2).

This simple model, and derivations from it, were compared with the unit hydrograph using data from a small catchment[7]. The results were very promising.

With small catchments the attenuation of the flood hydrograph during travel down the stream system is negligible. For larger catchments it can be considerable. It has been suggested that these translation effects can be allowed for by varying the reservoir lag time (K)[8].

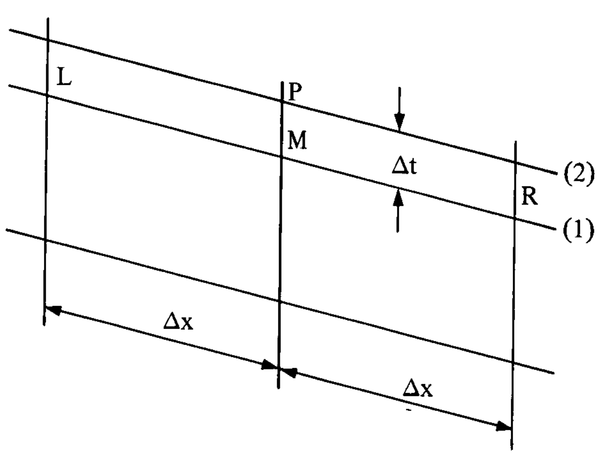
This was attempted when data from the larger catchments were analysed but poor results were obtained. In addition, if the value of the lag time is dependent on catchment size, values obtained for small catchments are difficult to apply to large catchments.

The approach finally adopted was to divide the catchment into a number of sub-catchments, the runoff from which was simulated using the land phase model. The translation of this runoff down the stream system to the catchment outfall was modelled using a modification of the finite difference technique developed by Morgali and Linsley[9].

3.2 Finite difference equations for channel flow

An exact mathematical solution of the equations describing the generation of a flood hydrograph in a stream system is not possible. Finite difference techniques can be used to get approximate solutions. The principle by which they operate is that very simple equations are adequate to describe the flow over very short distances and times. It follows that accurate solutions by this method for whole catchments would involve many repetitions of the calculations. This would be very tedious if done by hand but is quite feasible by digital computer.

Diagrammatic view of a reach of the stream:



SECTION OF STREAM

Lines 1 and 2 represent the water surface at times separated by a time increment Δt. L, M and R are subscripts which define three stations on line 1. P is the subscript of the middle station on line 2.

If the depth and velocities at the stations on line 1 are all known together with the lateral inflow from the linear reservoir, by moving progressively down the stream, the depths and velocities along line 2 can be computed using the momentum and continuity equations. The derivation of the appropriate finite difference equations is given in Appendix 1.

From the diagram it will be seen that values for the extreme upstream and downstream stations are not calculated and have to be estimated.

For the uppermost reach the upstream velocities and depths are assumed zero.

For lower reaches the upstream values are assumed to be the same as the downstream values for the previous reach.

At junctions the following equations apply

$$A_1 + A_2 = A_3$$

$$Q_1 + Q_2 = Q_3$$

where A = cross sectional area
Q = flow

and subscripts 1, 2 and 3 refer to the three reaches forming the junction.

The values for the downstream station of each reach are calculated by extrapolation of the values for the two immediately upstream stations.

A flow diagram for the computer program is shown in Fig 1.

3.3 Stability

Finite difference equations are only approximations and the accuracy of the solutions is dependent on the incremental length and times chosen. Morgali and Linsley[9] show that the equations will become unstable and errors will be introduced unless the following equation is satisfied.

$$V + \sqrt{g y} \leqslant \frac{\Delta x}{\Delta t} \qquad \ldots 3$$

where V = velocity
y = depth
g = acceleration due to gravity
Δx and Δt are the chosen distance and time increments.

Typical values for the initial incremental distance and times were 200m and 30s.

The instability takes the form of oscillations or waves in the recession part of the predicted hydrograph.

In the discussion of Morgali and Linsley's paper [10] several people pointed out that satisfying equ (3) did not automatically ensure that the equation would be stable. This was found to be correct. In the analysis, instability did occur on occasions, even if equ (3) were satisfied. This was particularly so for very steep catchments or for catchments with very large reservoir lag (K) values.

Tests were therefore incorporated in the program to establish when instability was occurring. The run was terminated and repeated with the time increment Δt reduced by a factor of two.

3.4 Model calibration and proving

To run the program for a given storm the required input is:

(a) The recorded rainfall for each 15 minute period

(b) The recorded streamflow hydrograph ordinates

(c) For each reach - area, length, slope, K, Y, C_A, Manning's "n", Δx, Δt.

The model works on Δt time intervals but assumes that the rainfall intensity is constant over a 15 minute period. For a single gauge, rainfall can be measured to a finer time scale, but when several raingauges are being averaged this is unjustified.

3.5 Error functions

For each combination of values of the parameters, a hydrograph is predicted. The ordinates of this hydrograph are compared with the ordinates of the recorded hydrograph and an error function calculated. A small error function indicates close agreement between predicted and recorded hydrographs. The error function adopted was the usual sum of the squares of the ordinate differences.

$$\text{ERF} = \Sigma (y - yo)^2$$

where y is a predicted ordinate and
y_o is a recorded ordinate.

To avoid undue significance being given to the larger storms a second error function was calculated. This compared the mean weighted ordinate error to the mean recorded ordinate.

$$\text{Per cent ordinate error} = \frac{\sqrt{\frac{\text{ERF}}{n}}}{\overline{y}_o} \times 100$$

where n = number of ordinates
$\overline{y}_o$ = mean of recorded ordinates

3.6 Model calibration

Approximately 4 years of data were available for each catchment. For most catchments these included a number of large storms, but, inevitably with such short periods of research, on some catchments only relatively small storms were recorded. The model was run for each large storm on a catchment and for a variety of values of the parameters K and C_A. The combinations giving the best agreement between recorded and predicted floods are listed in Table 1.

4. GENERALISATION OF FLOOD MODEL

4.1 Form of model

The flood model had now been calibrated for each catchment. To develop a general flood model the differences in catchment response to rainfall shown by the individual catchments was next examined. As the recorded storms varied in severity it was necessary to use the model for each catchment to simulate a flood of a known recurrence interval before a comparison could take place. A 10 year flood was selected for comparison. This was simulated by using a 10 year storm profile (see ref 11) and appropriate values for the model parameters K, C_A and Y (see 4.2 – 4.4). The results appear in Table 2.

Hydrographs, recorded and predicted for the largest storms and for 10 year floods are shown for each catchment in Figs 2 – 13.

Once estimates of the parameters Y, C_A, and K have been made for a catchment a design flood could be estimated by routing a design storm through the computer program. This can be a time consuming process and for many purposes a simpler technique is required. From C_A and Y the volume of runoff from any given design storm can be calculated and if the hydrograph shape can be related to the catchment lag time (K), the peak flow can also be estimated.

Many research workers have published "dimensionless hydrographs" and it has been shown[17] that in the United States these approximate closely to the equation

$$\frac{Q}{Qm} = \left[\frac{T}{Tm} \quad \exp\left(1 - \frac{T}{Tm}\right)\right]^{4.0} \qquad \ldots 4$$

where Q = discharge at time T after start of rise
Qm = peak discharge
Tm = time to peak

The most widely used dimensionless hydrograph is that of the US Soil Conservation Service[17]. For arid areas Hickok et al[18] suggest a somewhat more peaked shape.

For all of these the ratio of time to peak to base time are very similar. This was not found to be true for the East African catchments studied. For consistency the base time was assumed to be the time from 1 per cent of peak flow on the rising limb to 10 per cent of peak flow on the falling limb of the hydrograph. Defined this way, the ratio base time : time to peak is approximately 3.0 for the US hydrographs. For the East African catchments it varied between 2.7 and 11.0. The use of a single hydrograph based on time to peak was therefore not appropriate.

A much more stable ratio was found to be the peak flow (Q) divided by the average flow measured over the base time ($\bar{Q}$) (Peak Flow Factor F)

$$F = \frac{Q}{\overline{Q}} \qquad \ldots 5$$

This is the factor used by Rodier and Auvray[19] in West Africa. For very short lag times ($K \simeq 0.2h$) F was 2.8 ± 10%. For all lag times greater than 1 hour, F was 2.3 ± 10%. These figures hold true for the catchment results and were confirmed by a simulation exercise in which area, slope, lag time and contributing area coefficient were systematically varied.

The peak flow can therefore be simply estimated if the average flow during the base time of the hydrograph can be calculated.

The total volume of runoff is given by:

$$RO = (P - Y)\, C_A \,.\, A \,.\, 10^3 \ (m^3) \qquad \ldots 6$$

where P = storm rainfall (mm) during time period equal to the base time
Y = initial retention (mm)
C_A = contributing area coefficient
A = catchment area (km^2)

If the hydrograph base time is measured to a point on the recession curve at which the flow is $\frac{1}{10}$th of the peak flow then the volume under the hydrograph is approximately 7 per cent less than the total runoff given by equ (6).

The average flow ($\overline{Q}$) is therefore given by:

$$\overline{Q} = \frac{0.93 \,.\, RO}{3600 \,.\, T_B} \qquad \ldots 7$$

where T_B = hydrograph base time (hrs)

Estimates of Y and C_A are required to calculate RO and lag time K to calculate T_B. These will now be discussed in turn.

4.2 Initial retention (Y)

For the model fitting to the storms listed in Table 1 an appropriate initial retention was calculated from the balance of evapotranspiration and rainfall since the last storm to give significant runoff. This procedure could not be applied to the Mudanda catchment. Here a value of approximately 5mm was found appropriate. This is typical of arid zone catchments[12].

The probability of the soil on a catchment anywhere in East Africa being at field capacity has been studied by Huddart and Woodward[13]. For convenience their results are reproduced in this paper (Table 3, Figs 14 and 15). It will be seen that in the wet zones the 7 day antecedent rainfall easily exceeds the potential evapotranspiration during the same period. In the dry zones the figures are much closer but only in Western Uganda is there a high probability that the surface layers will be below field capacity. Here as in the semi arid zone, a 5mm initial retention is recommended for design purposes. Elsewhere assume zero initial retention.

4.3 Contributing area coefficient (C_A)

When the catchment surface is very dry, runoff is small and only occurs from areas very close to the stream system. For storms following a wet period a larger area contributes and larger volumes of runoff occur. If the catchment were sufficiently wet, the whole area would contribute and the value of C_A would approach unity. However, except on very small solid clay or rock catchments there is a practical upper limit to C_A which is well below unity. Evidence for this from the USA has already been referred to[6] and there is further confirmation in a recent TRRL study[14].

For simplicity it is assumed that the contributing area coefficient varies linearly with soil moisture recharge until the soil reaches field capacity when the limiting value of C_A is attained. Similar assumptions are made in the recent UK Flood Studies Report[15].

Four factors influence the size of the contributing area coefficient. These are soil type, slope, type of vegetation or landuse (particularly in the valley bottoms) and catchment wetness. The network of catchments had been selected to cover the range of these factors to be found in East Africa. The results could therefore be used to give indications of their effect on C_A.

The effect of slope and soil type was studied by comparing the results of the catchments with grass cover and the storms falling on soil at field capacity.

The effect of antecedent wetness was studied by comparing the runoff volumes resulting from storms occurring at different stages of the rainy season. The reduction in value of C_A was assumed to vary linearly with the soil moisture deficit. Using Tables 1 and 3, appropriate values of the reduction in C_A for design conditions for the various zones were arrived at.

The effect of landuse was calculated by comparing the recorded volumes of runoff with those that would have occurred with a standard grassed catchment.

The design value of the contributing area coefficient is therefore given by:

$$C_A = C_s \cdot C_w \cdot C_L \qquad \ldots 8$$

where C_s = the standard value of contributing area coefficient for a grassed catchment at field capacity

C_w = the catchment wetness factor

C_L = the land use factor

Tables for estimating these factors are given in Tables 4, 5 and 6.

4.4 Catchment lag times (K)

In Table 2 it will be seen that there is a very large range of lag times (K). Attempts were made to obtain correlation of K with various catchment characteristics such as overland slope, contributing area and drainage density, but the only factor found to show a strong relationship was vegetation cover. The same conclusion was drawn in a similar study by Bell and Om Kar, involving results from 47 small catchments located throughout the United States[16].

The appropriate value of lag time can be estimated by reference to Table 7. In assessing which category to place a given catchment it should be remembered that generally only small areas either side of the stream are contributing to the flood hydrograph. It is these areas, therefore, which must be assessed.

4.5 Base time

The method of estimating the base time was derived from the study referred to in section 4.1. It is made up of three parts:

(a) The rainfall time

(b) The recession time for the surface flow

(c) The attenuation of the flood wave in the stream system.

The rainfall time (T_p) is the time during which the rainfall intensity remains at high level. This can be approximated by the time during which 60 per cent of the total storm rainfall occurs. Using the general East African depth duration equation.

$$I = \frac{a}{(T + \frac{1}{3})^n} \qquad \ldots 9$$

where I = intensity
T = duration
a and n are constants

The time to give 60 per cent of the total storm rainfall is given by solving the equation

$$0.6 = \frac{T}{24} \left(\frac{24.33}{T + 0.33}\right)^n \qquad \ldots 10$$

Values for the various rainfall zones of East Africa are given in Table 8.

The time for the outflow from a linear reservoir to fall to $\frac{1}{10}$th of its initial value is 2.3K where K is the reservoir lag time. The recession time for surface flow is therefore 2.3K.

In the simulation study, values for base time were calculated for various areas, slopes, lag times and contributing area coefficients. Knowing the rainfall time and the surface flow recession time, the additional time for flood wave attenuation (T_A) can be found by difference. It was found that this could be estimated from the equation

$$T_A = \frac{0.028\ L}{\bar{Q}^{1/4}\ S^{1/2}} \qquad \ldots 11$$

where L = length of main stream (km)
$\bar{Q}$ = average flow during basetime (m^3/s)
S = average slope along mainstream

The base time is therefore estimated from the equation

$$T_B = T_p + 2.3K + T_A \qquad \ldots 12$$

The average flow ($\overline{Q}$) can be estimated. It will be noted that $\overline{Q}$ appears in equ (11) so an iterative or trial and error solution is required. If initially T_A is assumed zero, two iterations should be adequate.

Knowing $\overline{Q}$ and F the peak flow is calculated using equ (5).

5. SUMMARY OF DESIGN METHOD

The steps involved in estimating the peak flow for a design storm are as follows:

(a) Locate catchment on a large scale map and measure catchment area, land slope and channel slope. The land slope is estimated by superimposing a grid over the catchment and measuring the minimum distance between contours at each grid point. From these slopes are calculated and averaged to give the mean catchment value. The channel slope is the average slope from the bridge site to the uppermost part of the stream. Where information is sparse this may be taken as 85 per cent of the distance to the watershed.

(b) From site inspection establish catchment type in Table 7 and hence lag time (K).

(c) From site inspection or using Fig 15 establish soil type and with land slope estimate the standard contributing area coefficient (C_S) in Table 4

(d) Using Fig 14 fix antecedent rainfall zone. Check in Table 3 to see if zone is wet, dry or semi-arid.

(e) Estimate catchment wetness factor from Table 5.

(f) From site inspection decide on type of vegetative cover, paying particular attention to areas close to the stream. Using table 6 estimate land use factor (C_L).

(g) Contributing area coefficient (C_A) is given by:

$$C_A = C_S \cdot C_w \cdot C_L$$

(h) If antecedent rainfall zone in (d) above is semi-arid or West Uganda, initial retention (Y) is 5mm. For all other zones, Y = 0.

(i) Using Fig 16 and Table 8, estimate rainfall time (T_p).

(j) Using methods outlined in ref (11) calculate the design storm rainfall to be allowed for during time interval T_B hours (P mm).

(k) The volume of runoff is given by equ (6).

$$RO = C_A \cdot (P - Y) \cdot A \cdot 10^3 \ (m^3)$$

(l) The average flow is given by

$$\bar{Q} = \frac{0.93 \ . \ RO}{3600 \ . \ T_B} \qquad \ldots 13$$

(m) Recalculate base time

$$T_B = T_p + 2.3K + T_A$$

where $$T_A = \frac{0.028 \ L}{\bar{Q}^{1/4} S^{1/2}}$$

(n) Repeat steps (j) to (m) until $\bar{Q}$ is within 5 per cent of previous estimate.

(o) Design peak flow (Q) is given by

$$Q = F \ . \ \bar{Q}$$

where peak flood factor (F) is:

F = 2.8	K less than 0.5 hour
F = 2.3	K more than 1 hour

Table 9 indicates the accuracy achieved in using the method for the network catchments.

A worked example is given in Appendix 2.

6. DISCUSSION AND CONCLUSIONS

A reservoir analogue model has been developed which can be used to improve the usefulness of limited catchment data.

This has been used to derive a simple method of flood prediction for small rural catchments in which the most critical factors are landuse and soil type.

The catchments used to develop the method covered a range of landuse and soil type but inevitably could not cover all the combinations to be found throughout East Africa. In areas not already covered, very simple on site measurements by engineers would greatly improve the accuracy and usefulness of the design technique. If a note is kept of how many hours a flood resulting from a notable rainstorm lasts, an estimate can be made of the appropriate lag time to apply using equation 12. This is the factor that has the greatest influence on peak flow.

The most commonly specified recurrence intervals for small hydraulic structures are 5 or 10 years. The method has been designed to provide these figures easily.

Sometimes, for larger structures, longer recurrence intervals are specified. Using the method to estimate these introduces two elements of uncertainty.

(a) The shape and volume of the appropriate storm profile.

(b) The catchment response to exceptional storms. Of these the second is the more problematical.

There are a number of other problems in designing structures for very unusual floods. These include:–

(a) If the total flow is channelled through a bridge opening very high scouring velocities occur which can endanger the pier foundations.

(b) Very high flows uproot trees or islands of papyrus. Very often structure failure is due to spans being blocked by such debris rather than to the flow of the water itself.

For the size of catchment being considered here (up to 200 km^2) high flood flows only last for a few hours. If the structure itself can be safeguarded, designing for flows of greater than 10 year recurrence interval could well be uneconomic.

From the above considerations it is recommended that where possible designs are standardised at a 10 year recurrence interval with provision for larger flows to bypass the structure with safety. An example would be to provide approach embankments at a level lower than the bridge deck, thus giving a safe flood spillway and effectively limiting the velocity of flow through the bridge opening.

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TABLE 1

Details of storms and summary of analysis

Catchment	Date of Storm	Rainfall mm	Peak flow $m^3\ sec^{-1}$	Rainfall in previous 30 Days mm	Rainfall in previous 7 Days mm	Assumed Initial Retention mm	Optimum Values for Modelling: Lag Time K Mins	C_A	Error Function	% Age Ordinate Error	Predicted Peak Flow $m^3\ sec^{-1}$
TIWI	21/4/70	104.7	0.289	Record incomplete but Negligible Rainfall		95	95	0.050	0.0359	50.4	0.286
	9/5/72	58.0	1.428	74.3	38.8	25	80	0.050	0.9842	69.6	1.263
	11/5/72	98.9	5.836	277.5	242.3	0	70	0.090	7.1360	98.7	5.717
	6/7/72	27.4	0.354	11.2	11.2	15	100	0.050	0.2061	47.5	0.363
MUDANDA	26/11/69	14.9	0.680	108.8	69.8	3	3	0.081	0.2000	75.5	0.680
	14/4/71	27.5	0.810	6.2	1.5	7	3	0.080	0.1800	79.2	0.810
	1/12/71	17.2	0.370	48.9	19.6	5	3	0.047	0.0200	59.7	0.370
	2/12/71	20.5	0.710	89.6	60.3	8	3	0.105	0.1500	84.4	0.700
	4/11/72	15.8	0.540	20.8	14.4	5	3	0.067	0.0800	85.6	0.540
	21/11/72	10.8	0.340	66.8	10.7	4	3	0.067	0.0400	90.2	0.330
	29/11/72	22.1	0.370	74.9	0.9	7	3	0.037	0.0200	54.2	0.380
MIGWANI	24/11/69	60.4	159.770	129.2	89.8	3	30	0.280	11944.9	67.1	163.190
	23/11/71	96.7	241.930	12.0	9.0	30	20	0.430	26713.7	57.0	233.440
	2/12/71	66.2	159.770	155.4	65.3	3	20	0.380	24021.6	50.2	158.230
	20/12/71	35.4	75.640	238.4	31.9	3	20	0.250	2692.1	51.6	72.590
	15/11/72	53.9	139.320	247.4	75.5	3	20	0.350	5659.9	33.5	135.480
KAJIADO	2/6/70	24.2	0.771	85.0	41.4	15	140	0.380	0.1846	22.3	0.767
	1/6/71	26.4	0.431	88.5	14.9	15	140	0.130	0.0799	30.8	0.439
	22/12/71	71.5	2.011	74.7	37.0	32	35	0.100	0.9810	39.6	2.034
	25/12/71	64.6	4.181	140.1	90.9	0	120	0.380	4.5280	17.0	4.074
					Two storms on 23/4						
ESERET	23/4/70	58.2	2.560	113.3	0.0	0	5	0.065	4.3300	114.6	2.400
	16/2/73	43.5	0.960	33.8	25.6	5	5	0.029	0.2300	56.6	0.950
	19/2/73	67.0	3.260	104.9	100.8	0	5	0.059	1.1800	36.7	3.580
	22/2/73	20.6	0.940	215.1	189.5	5	5	0.062	0.3100	81.1	0.950
KIAMBU I	3/12/68	63.2	0.097		100.7	0	500	0.035	0.0033	12.3	0.100
	1/5/71	45.0	0.018	338.0	117.2	0	500	0.006	0.0008	43.0	0.012
	16/5/71	37.7	0.038	140.4	67.2	0	400	0.016	0.0017	34.0	0.031
KIAMBU II	3/5/72	16.1	0.053	76.5	49.2	0	200	0.017	0.0024	26.6	0.055
	11/5/72	12.1	0.068	111.2	18.9	0	175	0.022	0.0019	20.1	0.071
	3/6/72	53.6	0.435	102.8	0.5	35	150	0.080	0.1807	35.2	0.383
SAOSA	16/3/58	116.6	0.642	0.0	0.0	0	450	0.030	0.6090	34.2	0.643
	20/5/60	25.0	0.140	0.0	0.0	0	500	0.035	0.0122	16.3	0.144
	4/7/60	32.2	0.250	0.0	0.0	0	375	0.035	0.0124	11.9	0.247
	12/5/61	28.1	0.156	0.0	0.0	0	450	0.030	0.0556	32.8	0.167
	29/12/62	45.4	0.140	31.5 mm on 28/12/62		0	450	0.022	0.0056	9.5	0.135
	16/8/66	49.5	0.171	0.0	0.0	0	450	0.014	0.0535	39.5	0.151
BARABILI	19/8/70	74.8	0.235	199.9	63.9	0	600	0.030	0.2855	45.3	0.203
	28/8/71	101.8	0.751	200.4	56.6	0	400	0.065	0.3620	20.6	0.815
MUNYERE SUB	18/7/71	41.1	0.310	8.4	8.4	15	7	0.047	0.0100	35.3	0.300
	26/10/71	15.5	0.190	111.5	32.7	2	7	0.052	0.0000	44.0	0.190
	2/11/71	24.2	0.300	136.0	48.8	8	7	0.061	0.0200	73.5	0.290
	28/11/71	52.3	0.680	158.2	29.8	10	7	0.120	0.1900	65.4	0.670
	4/3/72	17.3	0.110	76.2	19.7	9	7	0.036	0.0000	38.6	0.110
RUBAARE	25/4/70	89.6	3.200	195.9	88.2	0	50	0.021	8.1900	52.5	3.160
	26/2/72	105.2	8.300	113.2	11.9	15	50	0.038	31.4000	56.5	8.150
LUGULA	21/11/69	28.7	0.364	155.4	51.8	0	450	0.140	0.2717	37.1	0.363
	27/3/70	42.9	0.767	241.7	37.1	0	500	0.280	0.8892	24.0	0.803
	1/2/72	30.8	0.116	124.8	22.8	0	600	0.060	0.0304	27.9	0.137

TABLE 2

10 year flood details

Catchment	Rainfall (mm)	Retension (mm) Y	Lag time (min) K	Contributing Area Coef Ca	Base time (hr) TB	Time to peak (h) TP	Peak flow (m^3/s) Q	Average flow (m^3/s) $\bar{Q}$	$F = Q/\bar{Q}$
Tiwi	102.5	0	70	0.09	6.00	1.35	5.66	2.70	2.10
Mudanda	88.4	5	5	0.20	1.60	0.60	6.79	2.41	2.82
Migwani	108.5	0	50	0.38	5.50	1.50	377.03	143.77	2.62
Kajiado	67.0	0	120	0.50	7.40	1.35	9.03	4.19	2.16
Eseret	67.0	0	5	0.06	3.00	0.85	2.95	1.04	2.84
Kiambu I	80.3	0	500	0.04	22.75	2.25	0.14	0.06	2.25
Kiambu II	80.3	0	175	0.15	9.75	1.65	3.55	1.63	2.18
Saosa	81.0	0	450	0.04	20.50	2.00	0.63	0.27	2.34
Barabili	88.0	0	500	0.07	22.00	2.00	0.67	0.28	2.42
Munyere sub	81.4	5	7	0.08	1.50	0.50	1.11	0.43	2.56
Rubaare	81.4	5	50	0.03	4.18	0.90	4.63	1.81	2.56
Lugula	88.8	0	500	0.30	21.75	2.00	2.27	0.95	2.39

Note: Base time is measured from 0.01Q on rising limb to 0.1Q on falling limb

TABLE 3

Antecedent catchment conditions for storms of greater than 50mm

	Potential Evaporation mm/day	2 day antecedent Rainfall (mm)		7 day antecedent Rainfall (mm)		Soil moisture Recharge (mm)	
		Mean	Std. Dev.	Mean	Std. Dev.	Mean	Std. Dev.
SEMI ARID							
North Eastern Kenya	6.9	20.4	32.0	46.5	56.5	45.0	62.6
DRY ZONES							
Western Uganda	5.2	10.2	14.6	32.6	28.3	40.2	39.0
Central Uganda	4.6	10.0	15.6	42.9	44.1	66.9	61.0
Northern Uganda	5.3	12.0	16.3	39.5	31.9	65.3	57.0
Nyanza	5.6	21.1	29.0	48.4	46.0	60.9	53.0
Central Tanzania	5.6	23.6	38.5	68.5	70.2	54.0	59.0
WET ZONES							
Kenya Coast	5.9	32.9	40.6	76.9	85.5	81.1	76.0
Tanzania Coast	6.0	25.6	45.5	56.9	58.4	90.1	64.0
Kitui	5.2	31.4	42.1	83.4	84.6	101.8	84.5
Nairobi	4.9	21.2	27.8	81.7	67.1	117.0	67.0
Lake Malawi	4.4	41.0	49.8	125.5	121.0	170.3	74.0

TABLE 4

Standard contributing area coefficients
(Wet zone catchment, short grass cover)

Catchment slope		Soil type		
		Well drained	Slightly impeded drainage	Impeded drainage
Very Flat	< 1.0%		0.15	0.30
Moderate	1-4%	0.09	0.38	0.40
Rolling	4-10%	0.10	0.45	0.50
Hilly	10-20%	0.11	0.50	
Mountainous	> 20%	0.12		

Note:

The soil types are as in Fig 16 and are based on the soils map contained in the Handbook of Natural Resources of East Africa (see ref 13).

TABLE 5

Catchment wetness factor

Rainfall zone	Catchment wetness factor (C_W)	
	Perennial streams	Ephemeral streams
Wet zones	1.0	1.0
Semi arid zone	1.0	1.0
Dry zones (except West Uganda)	0.75	0.50
West Uganda	0.60	0.30

TABLE 6

Land use factors (C_L)

(Base assumes short grass cover)

Largely bare soil	1.50
Intense cultivation (Particularly in valleys)	1.50
Grass cover	1.00
Dense vegetation (particularly in valleys)	0.50
Ephemeral stream, sand filled valley	0.50
Swamp filled valley	0.33
Forest	0.33

TABLE 7

Catchment lag times

Catchment type	Lag time (K) hrs
Arid	0.1
Very steep small catchments (slopes > 20%)	0.1
Semi arid scrub (large bare soil patches)	0.3
Poor pasture	0.5
Good pasture	1.5
Cultivated land (down to river bank)	3.0
Forest, overgrown valley bottom	8.0
Papyrus swamp in valley bottom	20.0

TABLE 8

Rainfall time (Tp) for East African 10 year storms

Zone	Index "n"	Rainfall time (Tp) (h)
Inland zone	0.96	0.75
Coastal zone	0.76	4.0
Kenya-Aberdare Uluguru Zone	0.85	2.0

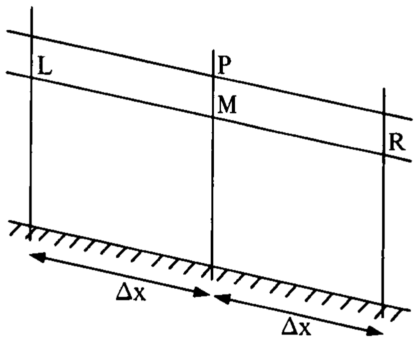
TABLE 9

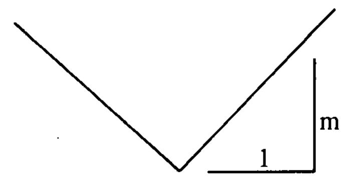
Comparison between predicted 10 year floods using computer and short methods

Catchment	Storm rainfall (mm)	Assumed Catchment Parameters									10 YEAR PEAK FLOWS	
		Ca	CL	Cw	y(mm)	k(hr)	Area (km^2)	Land slope %	Channel slope %	Channel length (km)	Computer method (m^3/sec)	Short method (m^3/sec)
Tiwi	122	0.09	0.75	1.00	0	1.0	6.7	2.2	1.2	5.08	5.7	6.6
Mudanda	89	0.10	1.50	1.00	5.0	0.3	1.7	8.2	5.0	2.67	6.8	8.0
Migwani	105	0.38	1.00	1.00	0	1.0	83.5	3.0	1.3	19.05	377.0	430.2
Kajiado	68	0.50	1.00	1.00	0	1.5	3.6	8.8	2.7	3.43	9.0	12.0
Eseret	57	0.12	0.50	1.00	0	0.1	3.2	22.0	9.2	3.70	3.0	3.1
Kiambu I	112	0.10	0.50	0.75	0	8.0	2.0	4.0	3.6	2.75	0.14	0.27
Kiambu II	112	0.10	1.50	1.00	0	3.0	5.2	6.6	2.8	6.03	3.6	4.5
Barabili	96	0.15	0.50	0.75	0	8.0	3.9	0.7	0.3	2.03	0.67	0.61
Munyere sub	54	0.11	1.00	0.60	5.0	0.1	0.5	27.0	23.5	0.83	1.11	1.12
Rubaare	65	0.10	1.00	0.30	5.0	1.0	13.7	6.0	4.9	6.99	4.6	4.3
Lugula	103	0.45	0.50	0.75	0	8.0	3.1	9.0	0.7	2.29	2.27	1.74
Saosa	103	0.11	0.33	0.75	0	8.0	6.8	12.0	3.4	4.54	0.63	0.55

APPENDIX 1

Derivation of finite difference equations





SECTION

The equations are developed from the continuity and momentum equations.

Continuity equation:

$$A \frac{\partial V}{\partial x} + V \frac{\partial A}{\partial x} + \frac{\partial A}{\partial t} = q$$

(q = inflow per unit length)

For simplicity the cross-section is assumed triangular.

Therefore $\quad A = my^2$

Therefore $$\frac{\partial (y^2 V)}{\partial x} + 2y \frac{\partial y}{\partial x} = \frac{q}{m} \qquad \ldots 1$$

Momentum equation

$$\frac{1}{A} \frac{\partial (\bar{y} A)}{\partial x} + \frac{V}{g} \frac{\partial V}{\partial x} + \frac{1}{g} \frac{\partial V}{\partial t} = So - Sf \qquad \ldots 2$$

Symbols y, v, x, t, g have their usual meaning,

$\bar{y}$ is the depth to the centroid of area measured from the water surface.

So is the channel bed slope

Sf is the energy line slope.

In finite difference form

$$\left.\frac{\partial V}{\partial x}\right/_M = \frac{VR - VL}{2\Delta x}$$

$$\left.\frac{\partial y}{\partial x}\right/_M = \frac{YR - YL}{2\Delta x}$$

$$\left.\frac{\partial V}{\partial t}\right/_P = \frac{VP - VM}{\Delta t}$$

$$\left.\frac{\partial y}{\partial t}\right/_P = \frac{YP - YM}{\Delta t}$$

$$q = \frac{Qin}{\Delta x}$$

Substituting in Equ (1)

$$\frac{YR^2 VR - YL^2 VL}{2\,\Delta\,x} + \frac{(YP + YM)(YP - YM)}{\Delta t} - \frac{Qin}{m\Delta x} = 0$$

Simplifying

$$YP = \sqrt{YM^2 - \frac{\Delta t}{2\Delta x}\,YR^2\;VR + \frac{\Delta t}{2\Delta x}\,YL^2 VL + \frac{\Delta t}{\Delta x}\,\frac{Qin}{m}} \qquad \ldots 3$$

From Equ 2

$$YM\,\frac{(VR - VL)}{2\Delta x} + \frac{VP - VM}{\Delta t} + \frac{g}{2\Delta x}\,(YR - YL) = g(So - Sf) \qquad \ldots 4$$

From the Manning equation

$$Sf = \frac{Vp^2\;n^2}{Rp^{4/3}}$$

$$\text{and} \quad RP = \frac{mYP}{2\sqrt{1 + m^2}}$$

where n = Manning's "n".

Substituting in (4) and simplifying:

$$VP^2\,\frac{g\,n^2}{4/3} + \frac{VP}{\Delta t} - \frac{VM}{\Delta t} + \frac{VM}{2\Delta x}\,(VR - VL) + \frac{g}{2\Delta x}\,(YR - YL) - g\;So = O \qquad \ldots 5$$

Equations (3) and (5) are the equations used in the computer program.

APPENDIX 2

Worked Example

A 10 year flood design is required for a catchment having the following details:

(a) Area: 10 km^2

(b) Land slope: 6%

(c) Channel slope: 3%

(d) Channel length: 4.0 km

(e) Grid reference 5°S 35°E

(f) Catchment type: poor pasture

From Table 7, lag time (K) = 0.5 h

From Fig 16 and Table 4, standard contributing area coefficient C_S = 0.45.

From Table 5, catchment wetness factor (C_W) = 0.50.

From Table 6, land use factor (C_L) = 1.0.

Therefore, design value for C_A = 0.23.

Initial retention (Y) = 0.

From Table 8, T_p = 0.75 hrs.

Using equ (11) with T_A = 0.

$$T_B = 0.75 + 2.3 \,.\, 0.5 = 1.15 \text{ hrs}$$

Rainfall during base time is given by:

$$R_{T_B} = \frac{T_B}{24} \left(\frac{24.33}{T_B + 0.33}\right)^n \,.\, R^{10}/_{24} \quad \text{(see ref (11))}$$

where $R^{10}/_{24}$ = 10 year daily rainfall

and $n = 0.96$ (see Table 8).

Using rainfall maps in ref (11),

2 yr daily point rainfall = 63 mm

10:2 yr ratio = 1.49

10 yr daily point rainfall = 94 mm

$$R_{1.15} = \frac{1.15}{24} \left(\frac{24.33}{1.48}\right)^{0.96} . 94 = 59.0 \text{ mm}$$

Area reduction factor is given by

$$ARF = 1 - 0.04\ T^{-\frac{1}{3}}\ A^{\frac{1}{2}} = 0.88$$

Average rainfall (P) = 59.0 x 0.88 = 51.9 mm

$$RO = C_A . (P - Y) . A . 10^3$$

$$\overline{Q} = \frac{0.93 . RO}{3600 . T_B} = 26.23 \text{ m}^3/\text{sec}$$

$$T_A = \frac{0.028\ L}{\overline{Q}^{\frac{1}{4}} S^{\frac{1}{2}}} = 0.29 \text{ hrs}$$

T_B (2nd approximation) = 1.15 + 0.29 = 1.44 hrs

$$R_{1.44} = \frac{1.44}{24} \left(\frac{24.33}{1.77}\right)^{0.96} . 94 = 69.9 \text{ mm}$$

ARF = 0.89

Therefore, P = 62.4 mm

$\overline{Q}$ = 25.11 m^3/sec

T_A = 0.29 hrs (no change)

Therefore, Q = F . $\overline{Q}$

F = 2.8

Therefore, Q = 2.8 . 25.11

= 70.3 m^3/sec

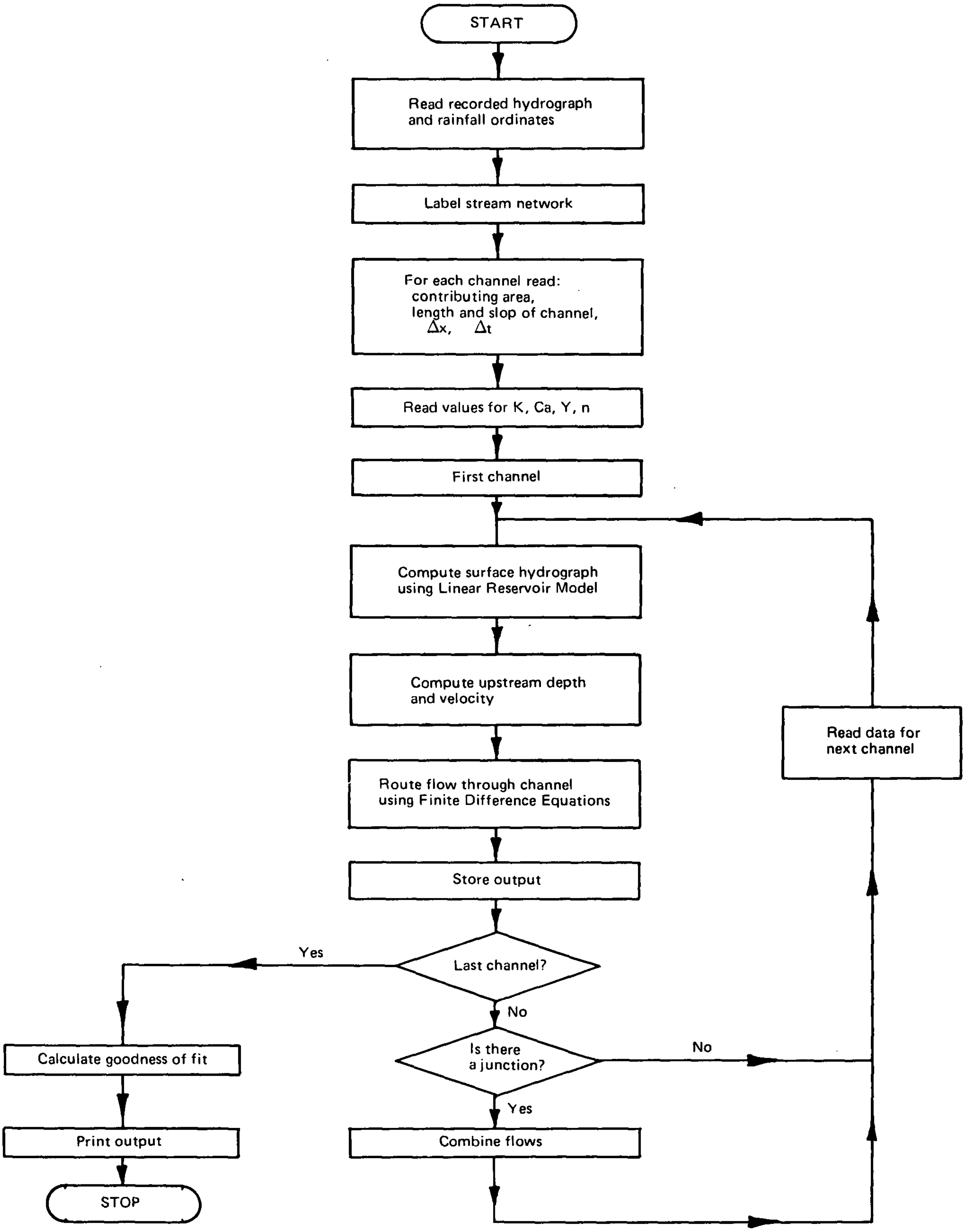


Fig. 1 FLOW DIAGRAM FOR RURAL FLOOD MODEL

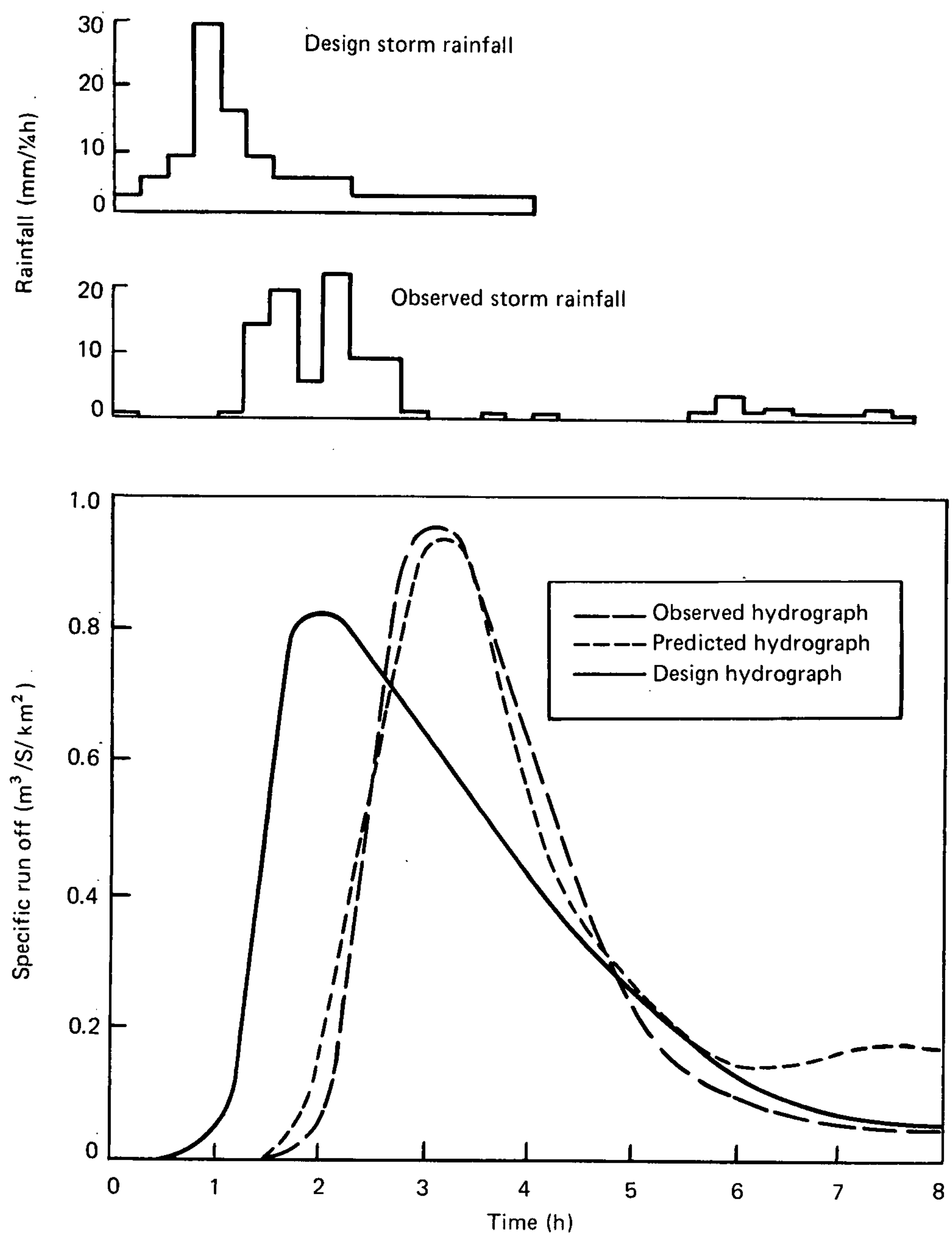


Fig. 2 TIWI CATCHMENT HYDROGRAPHS

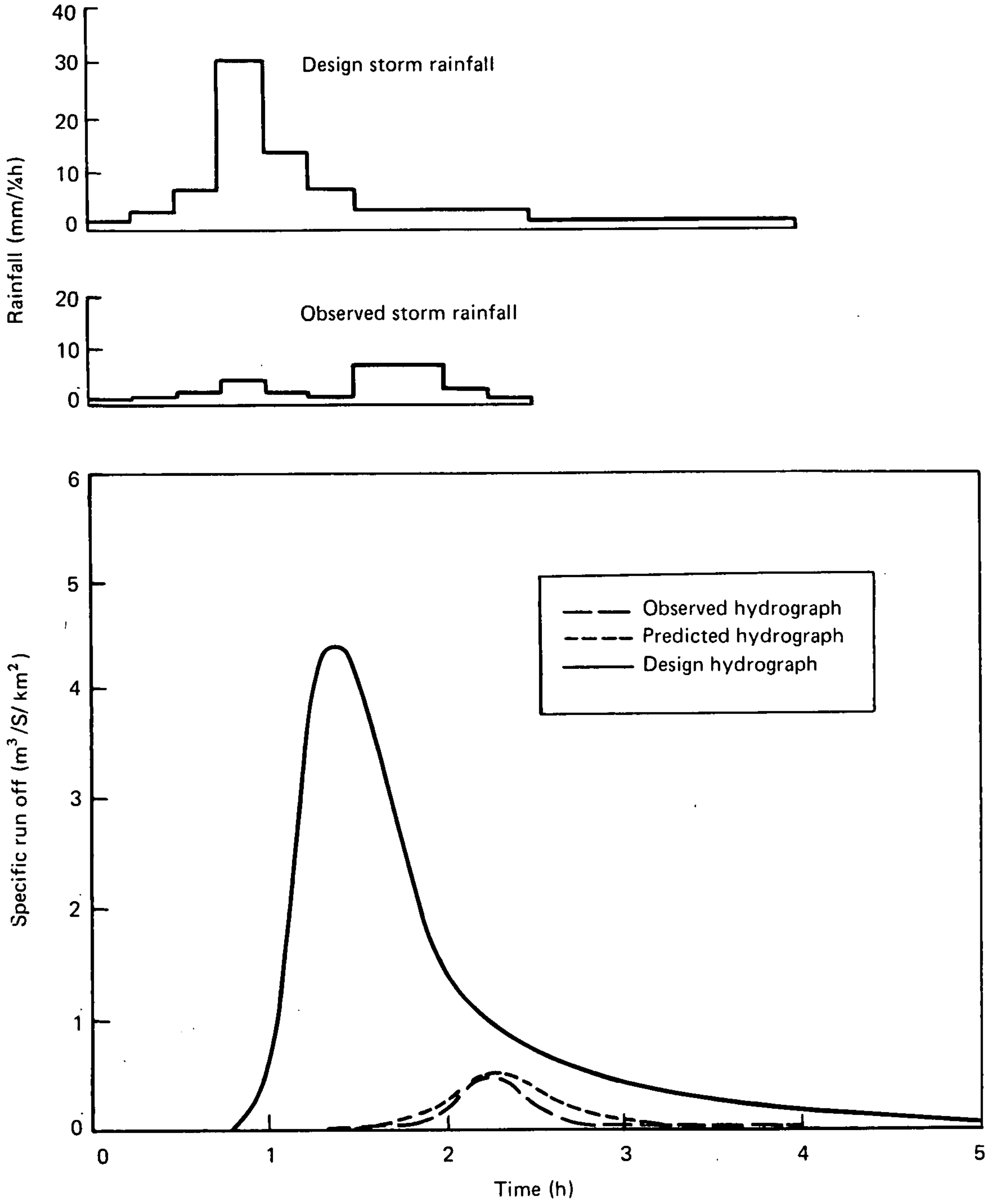


Fig. 3 MUDANDA CATCHMENT HYDROGRAPHS

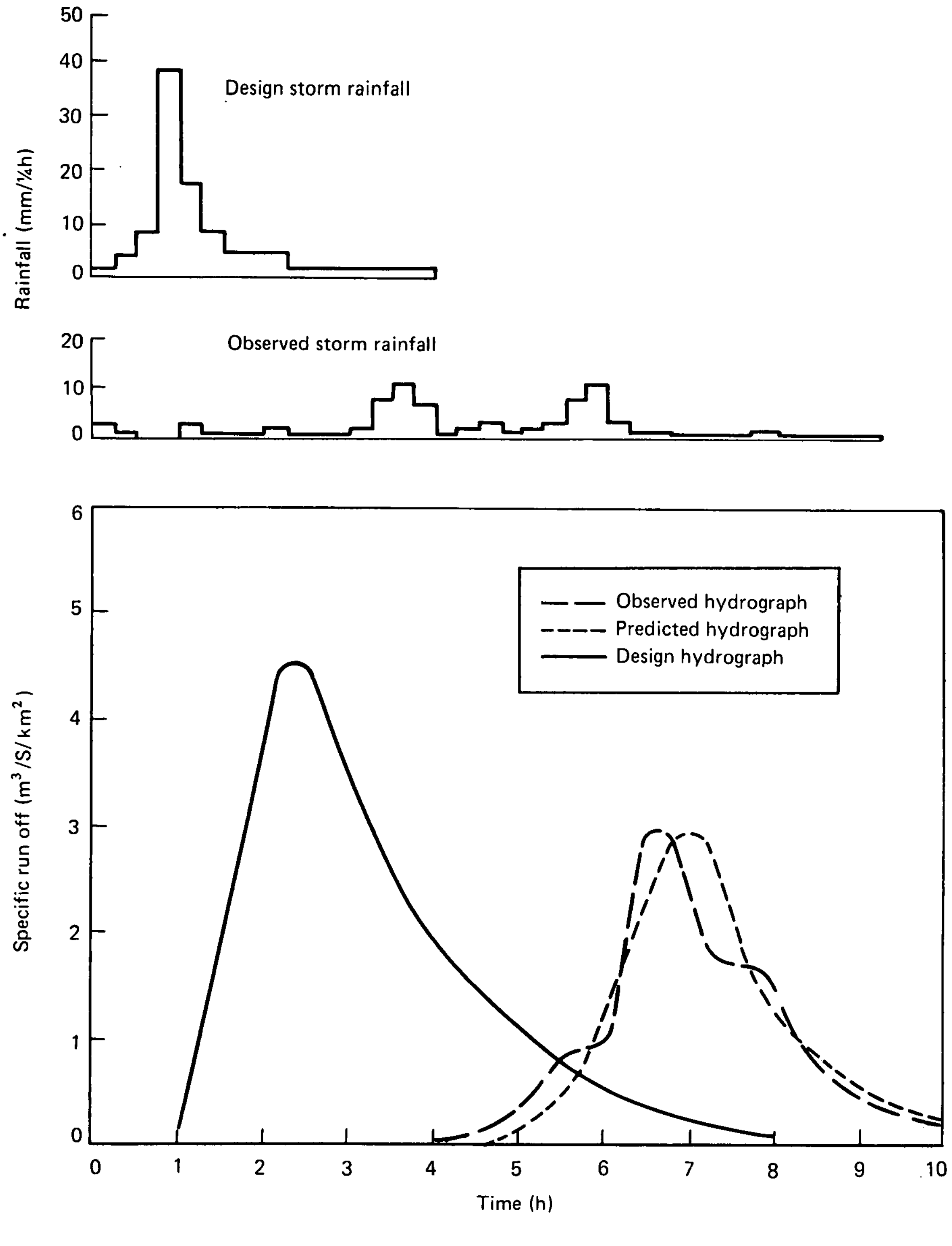


Fig. 4 MIGWANI CATCHMENT HYDROGRAPHS

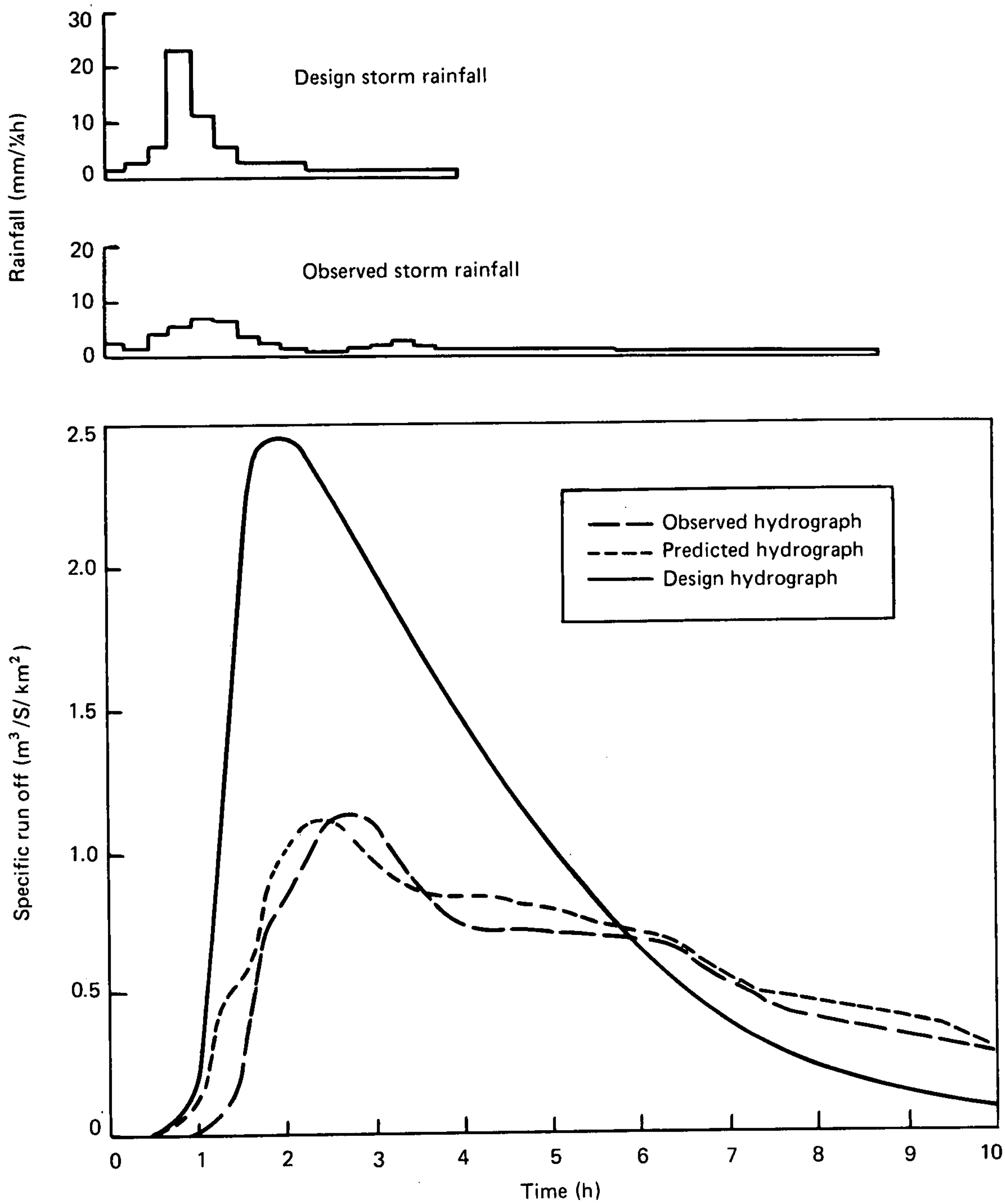


Fig. 5 KAJIADO CATCHMENT HYDROGRAPHS

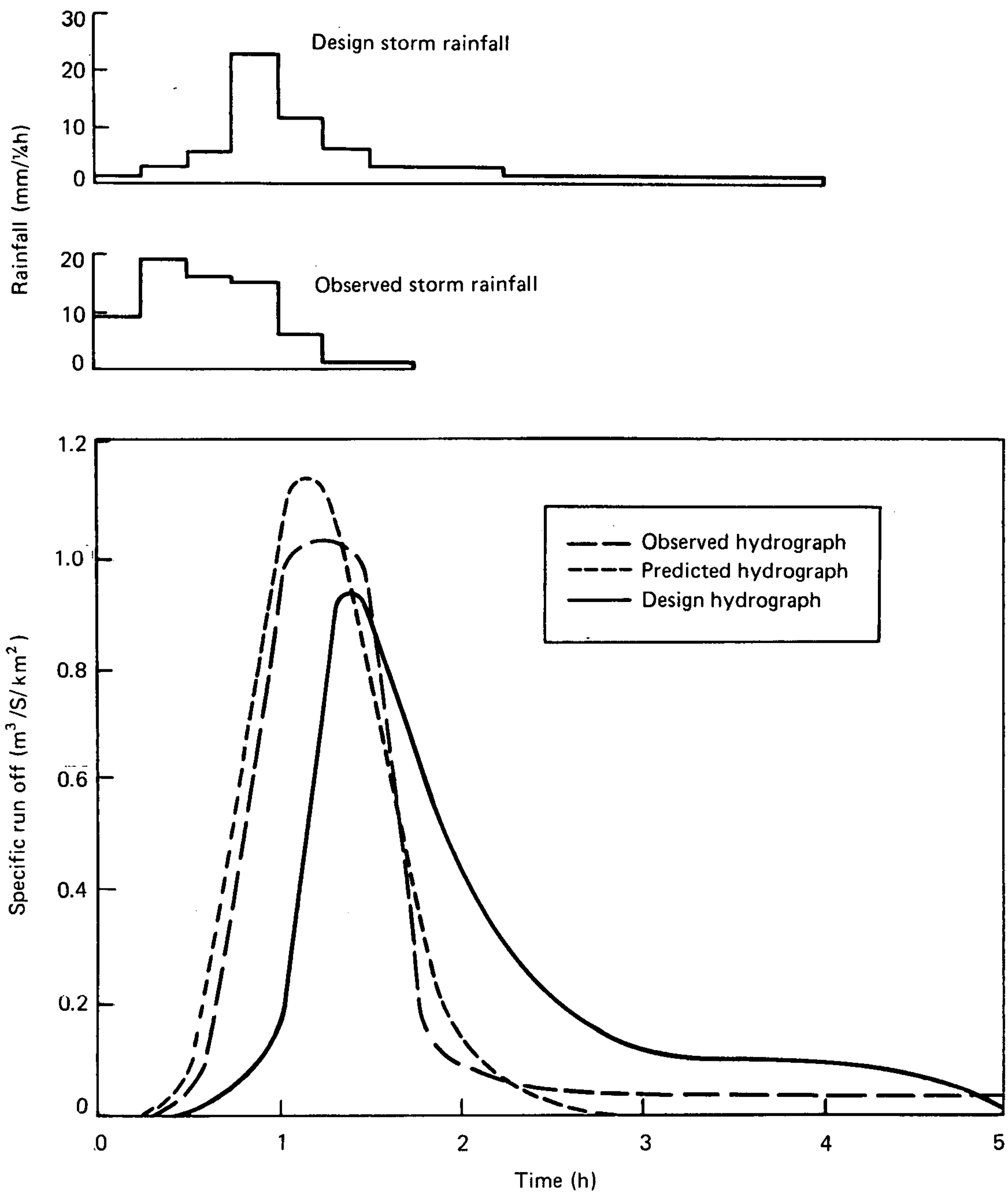


Fig. 6 ESERET CATCHMENT HYDROGRAPHS

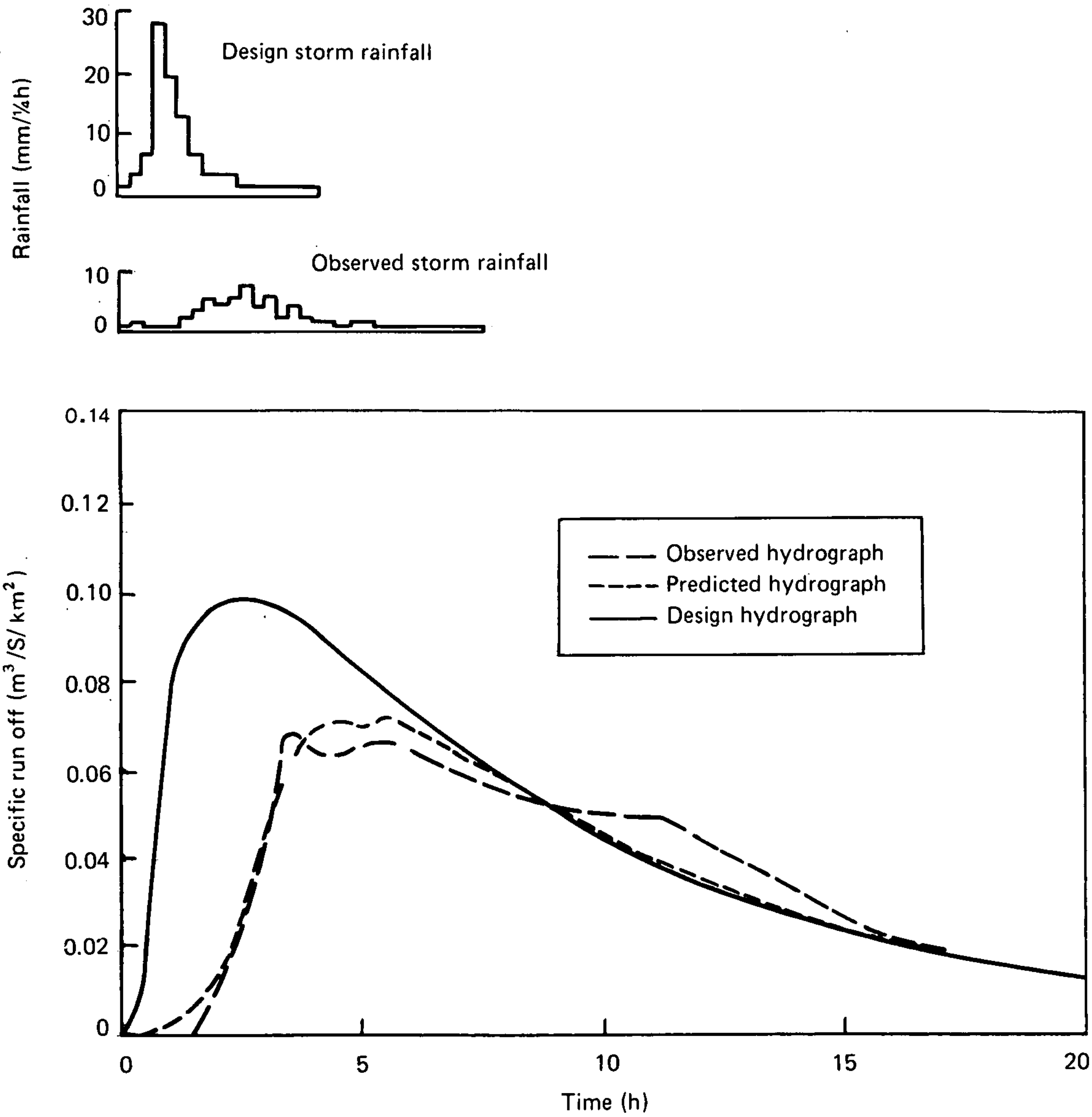


Fig. 7 KIAMBU I CATCHMENT HYDROGRAPHS

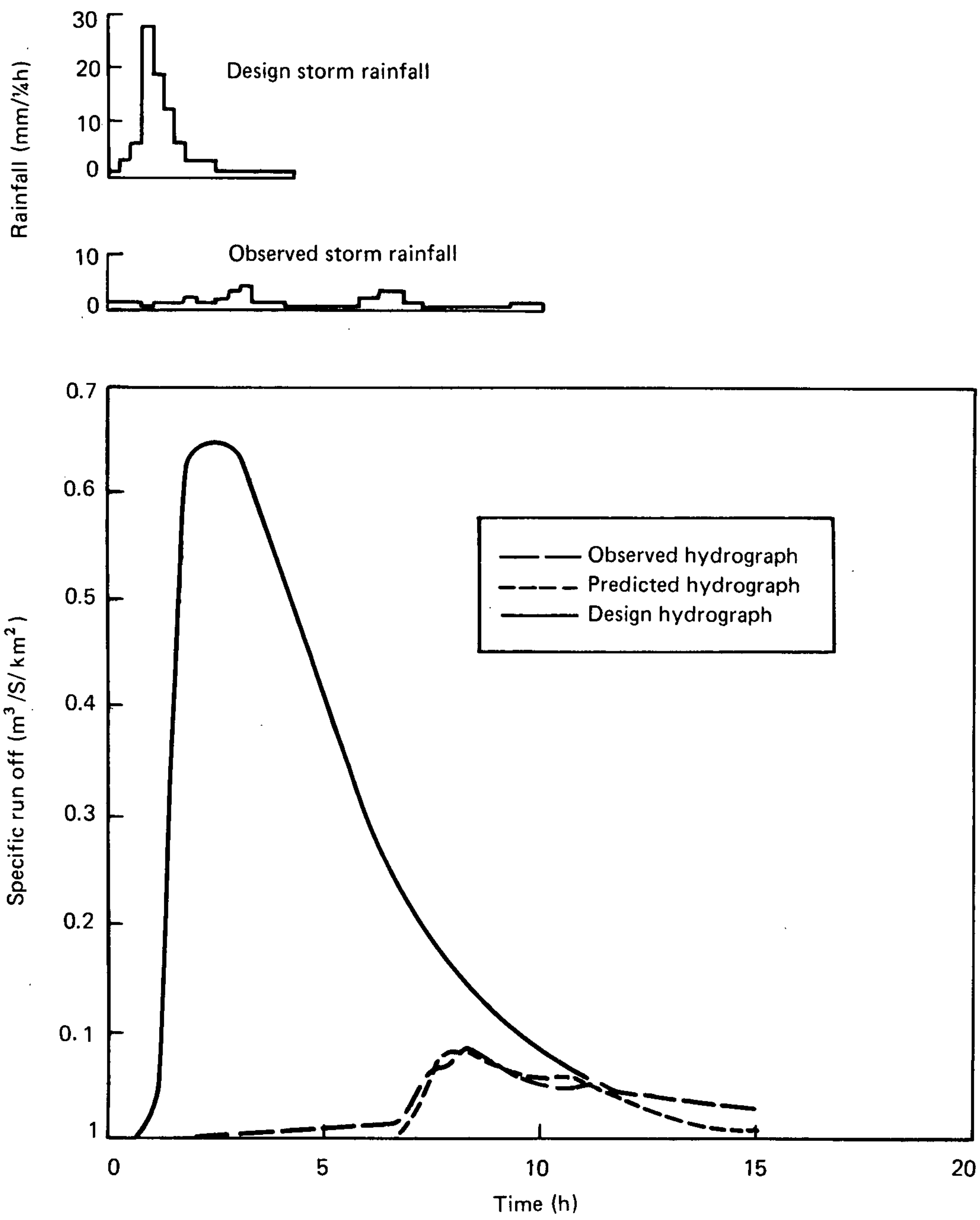


Fig. 8 KIAMBU II CATCHMENT HYDROGRAPHS

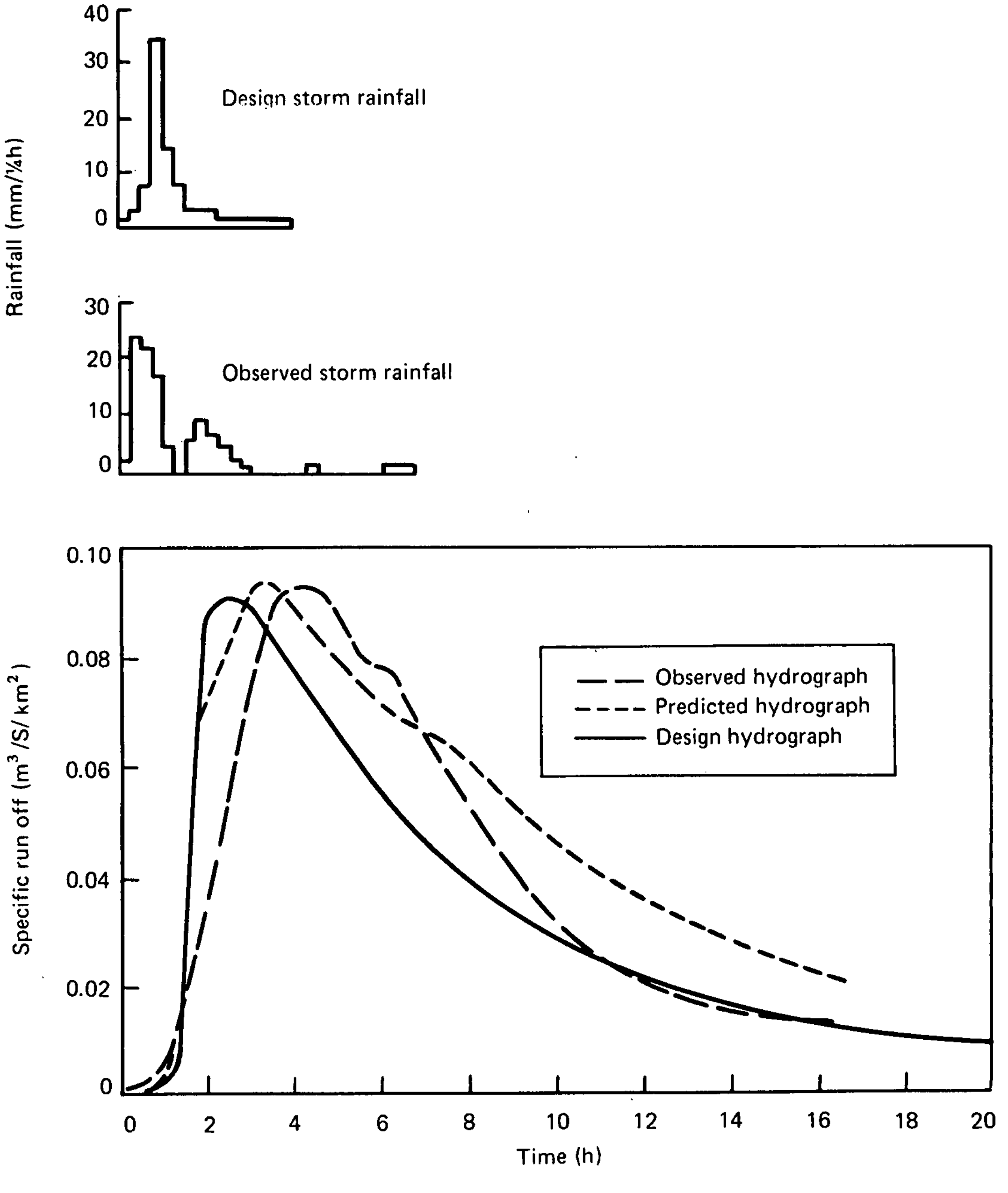


Fig. 9 SAOSA CATCHMENT HYDROGRAPHS

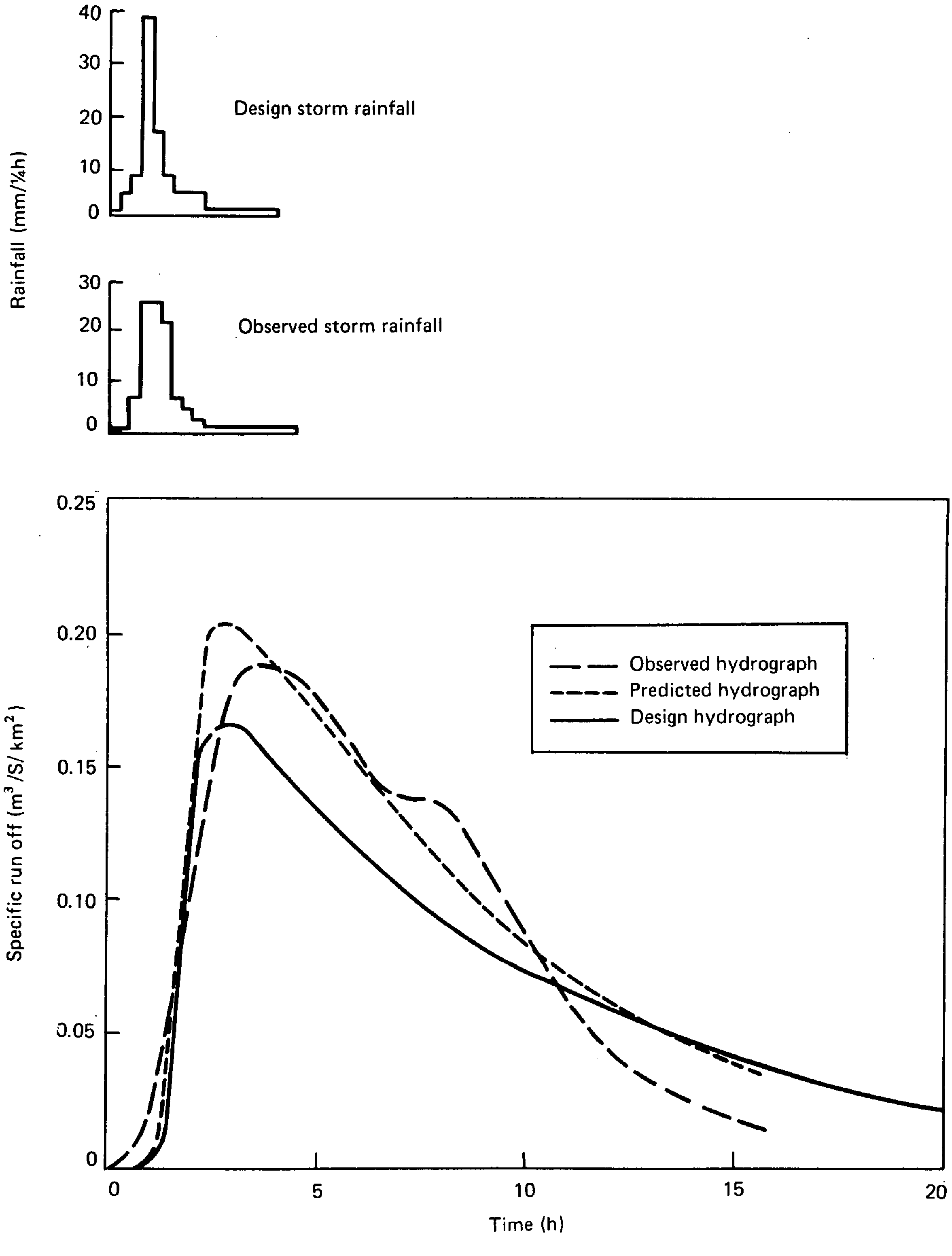


Fig. 10 BARABILI CATCHMENT HYDROGRAPHS

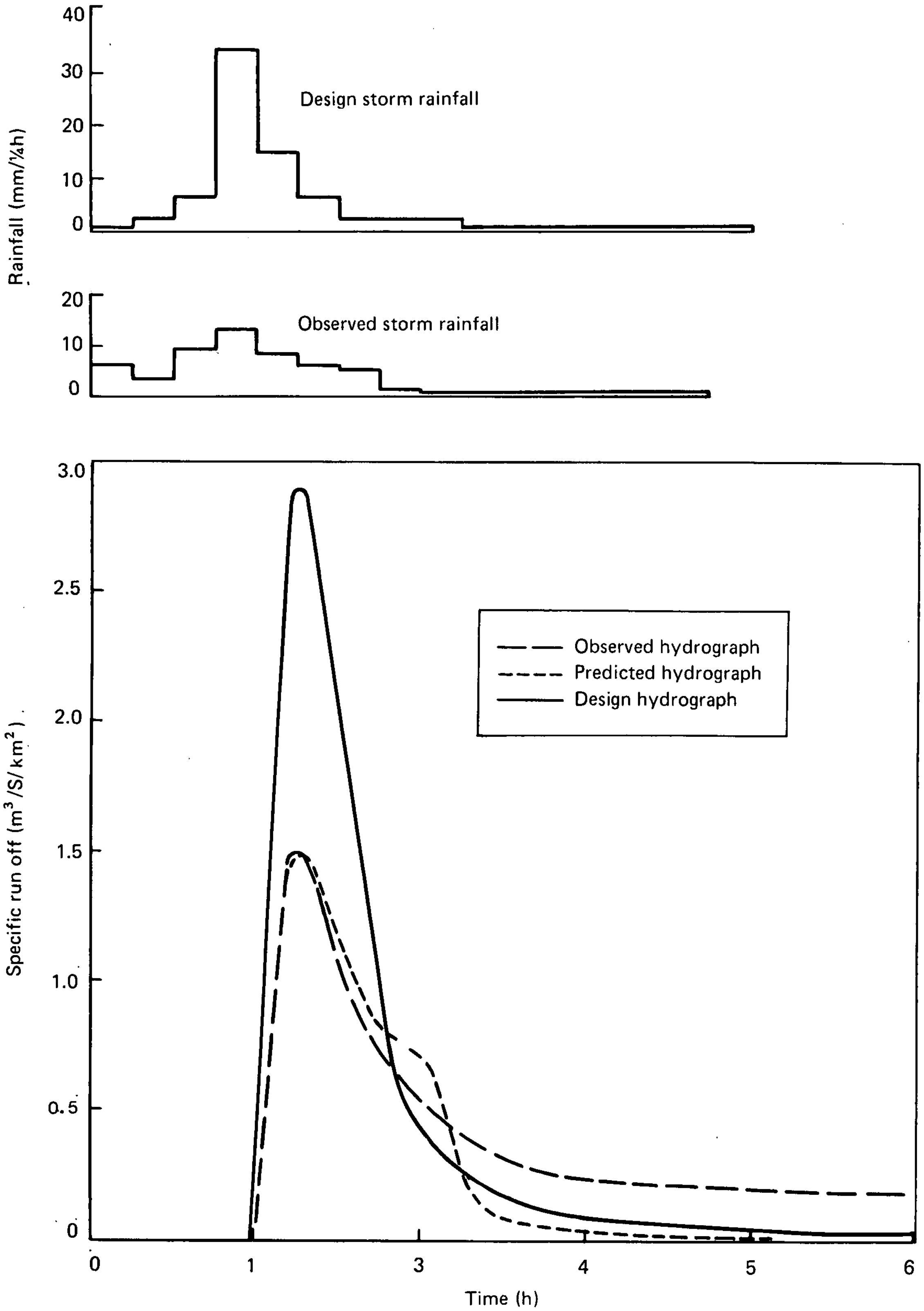


Fig. 11 MUNYERE CATCHMENT HYDROGRAPHS

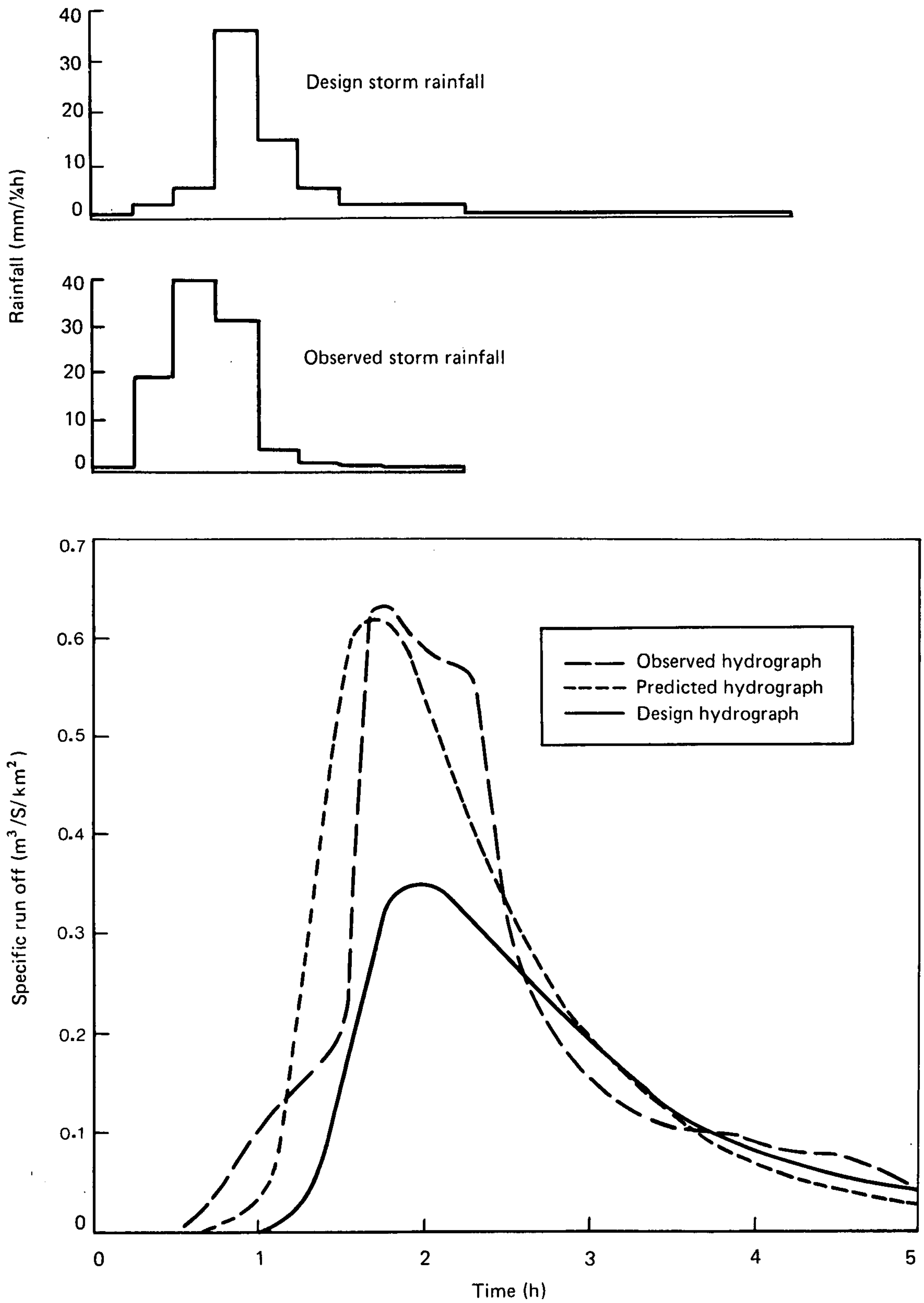


Fig. 12 RUBAARE CATCHMENT HYDROGRAPHS

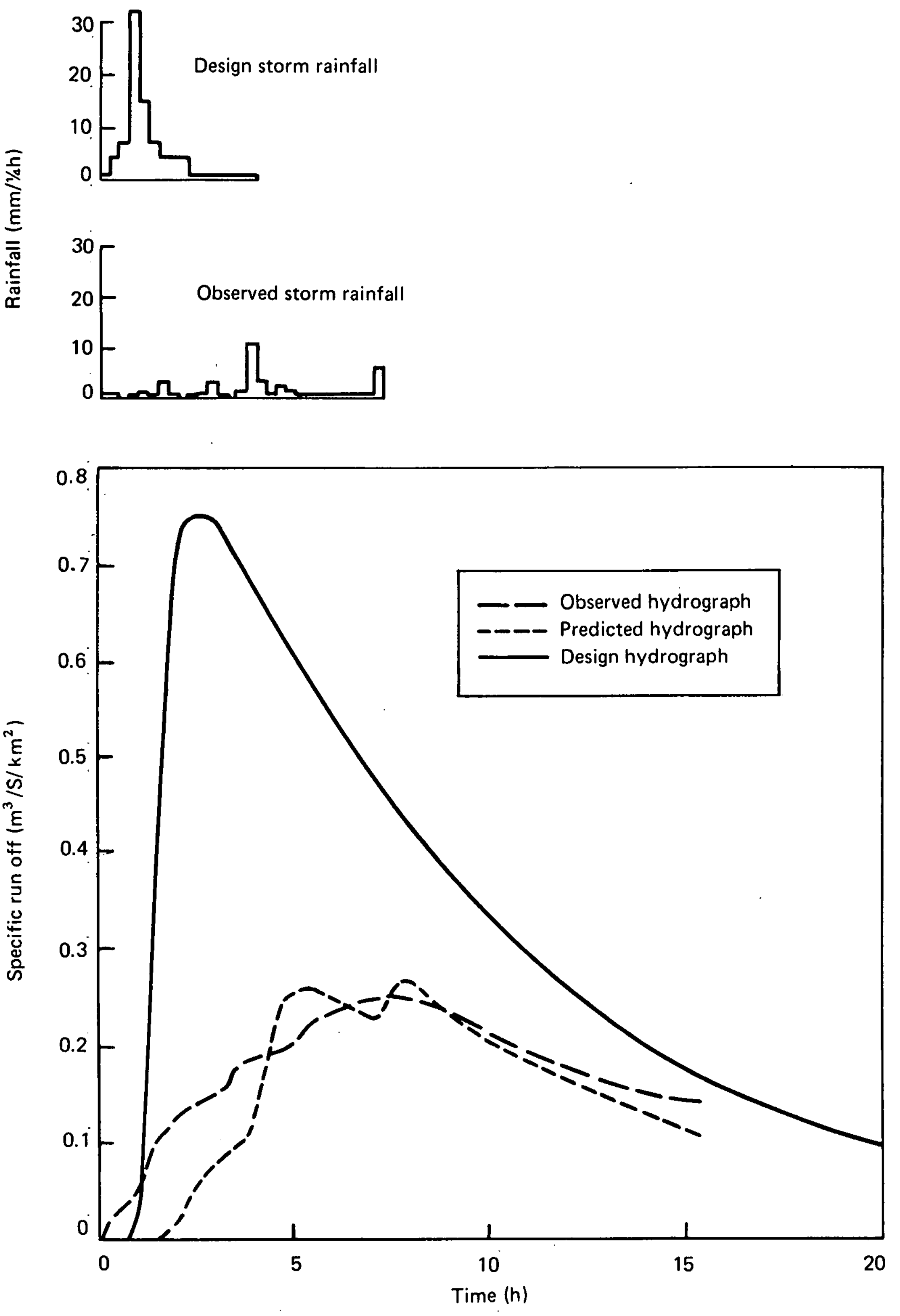


Fig. 13 LUGULA CATCHMENT HYDROGRAPHS

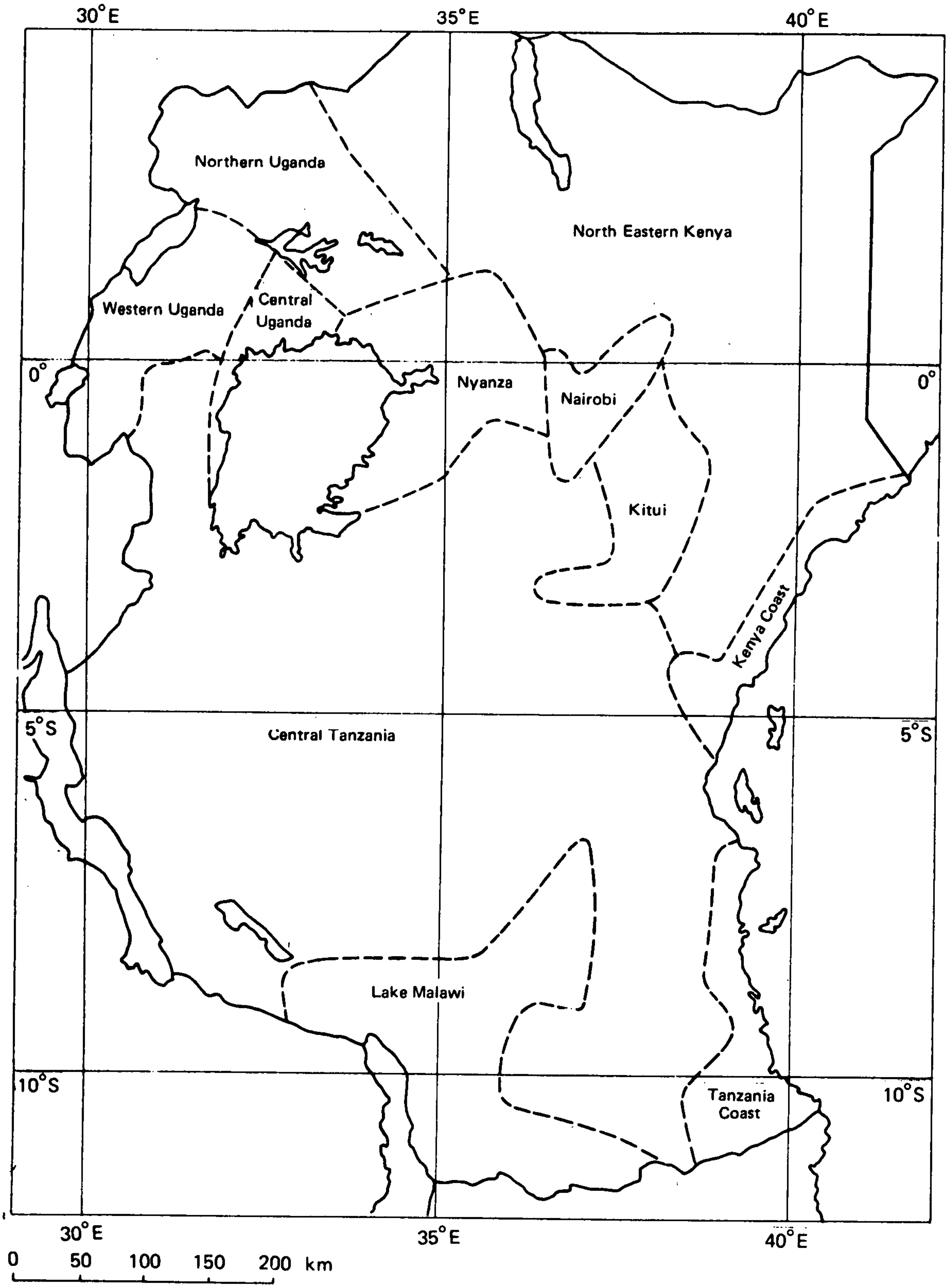


Fig. 14 AREAS FOR CALCULATION OF 2 AND 7 DAY ANTECEDENT RAINFALL, AND 30 DAY SMR

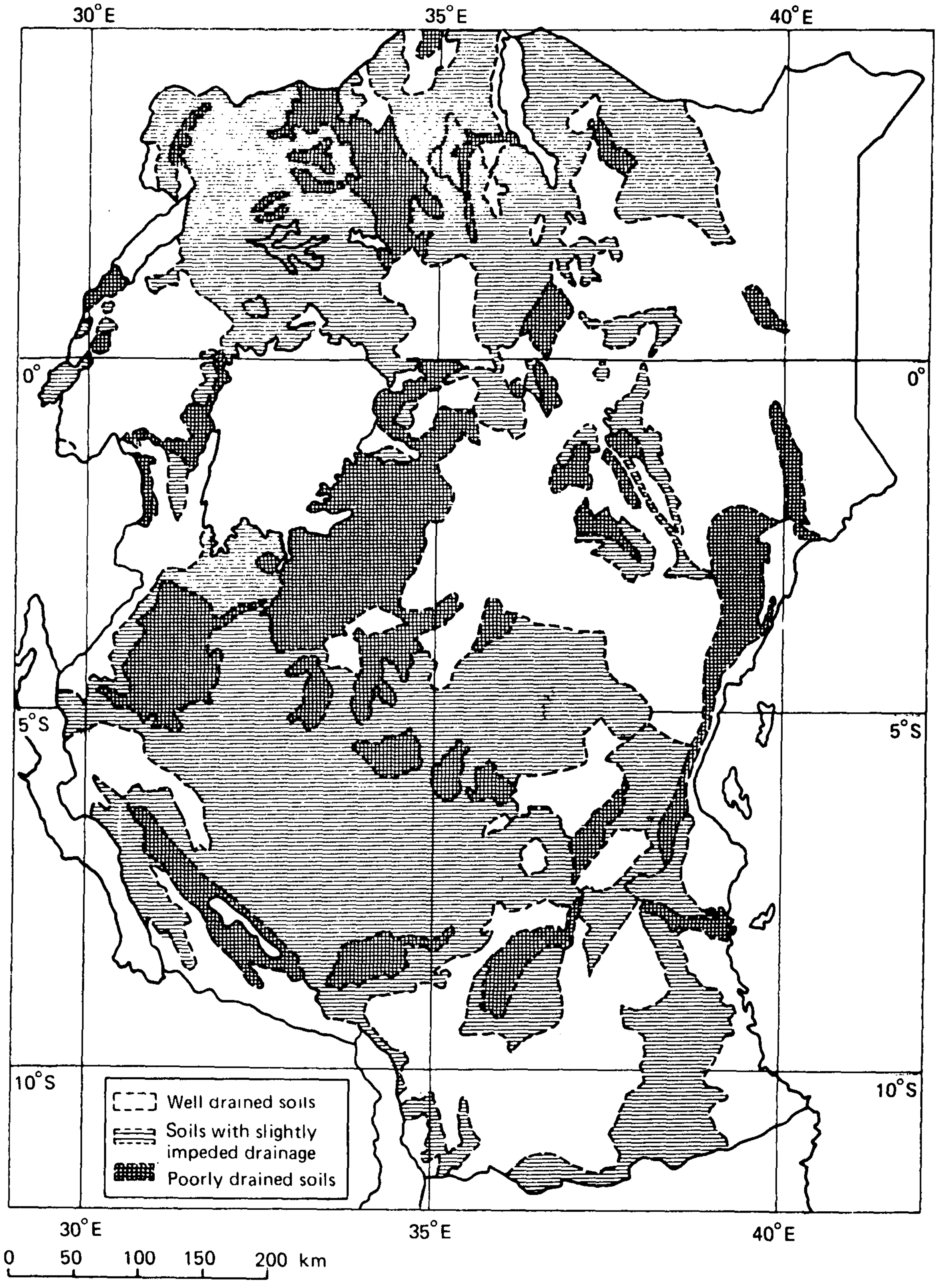


Fig. 15 SOIL ZONES

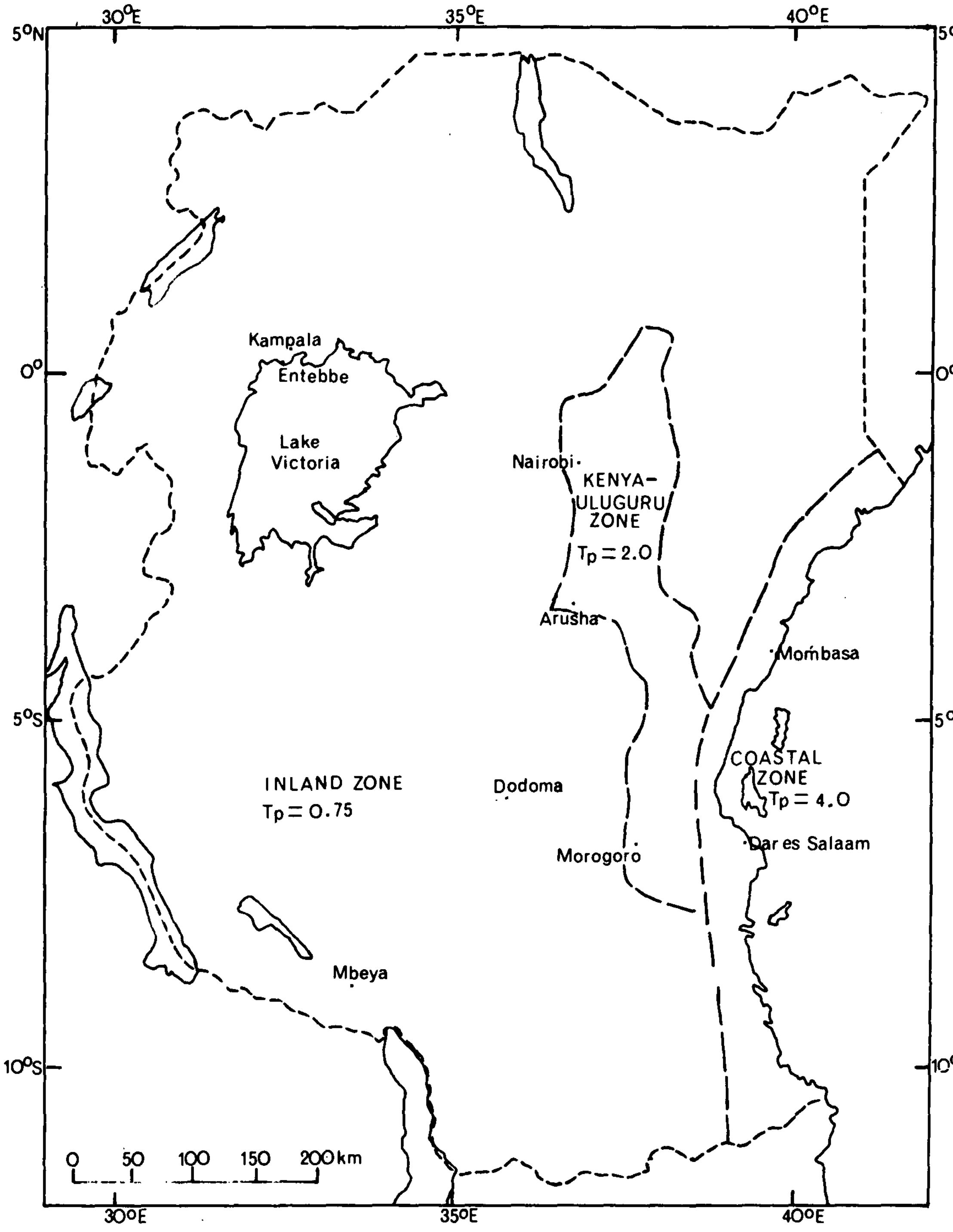


Fig. 16 RAINFALL TIME (Tp) ZONES

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ABSTRACT

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