STATISTICAL ANALYSIS OF PEAK FLOWS WITH SPECIAL REFERENCE TO KERALA RIVER BASINS

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By

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CERTIFICATE

This is to certify that the work reported in this thesis entitled **STATISTICAL ANALYSIS OF PEAK FLOWS WITH SPECIAL REFERENCE TO KERALA RIVER BASINS** is being submitted by Sri Soney John for the award of the Doctor of Philosophy, to the University of Calicut, is based on the bonafide research work carried out by him under my supervision and guidance in the Department of Statistics, University of Calicut. The results embodied in this thesis have not been included in any other thesis submitted previously for the award of any degree or diploma.

Dr. K. Kumaran Kutty

C U Campus
December 20, 1999.

DECLARATION

I hereby declare that the matter embodied in this thesis is the result of investigations carried out by me in the Department of Statistics, University of Calicut, under the supervision and guidance of Dr. K. Kumaran Kutty, Department of Statistics, University of Calicut and it has not been submitted for award of any Degree/Diploma of any other University or Institute.

SONEY JOHN

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INTRODUCTION

Soney John "Statistical analysis of peak flows with special reference to Kerala river basins" Thesis. Department of Statistics, University of Calicut, 1999

1. INTRODUCTION

1.1 SCOPE

Statistical flood frequency estimation, based on annual maxima of streamflow data has been attempted for the river basins of Kerala region of the Malabar Coast. The data of homogeneous flood regions have been combined to increase the sample size as is done in similar studies elsewhere (NERC, 1975). The investigations also extended to finding out a relationship between mean annual peak flow and certain basin characteristics, thereby helping in estimation of the flood frequencies of ungauged sites.

Estimating accurately the magnitude and frequency of relatively rare flood events are important because these events cause greate economic losses and loss of life. This has lead hydrologists to search for flood records outside at site; systematic records are required to improve their estimates. Thomson et al (1964) obtained prerecord information from documents, news papers and information from long term residents and Costa (1978), Baker (1983), and Sigafoos (1964) obtained prerecord flood information from examining sediment deposits and botanical evidence to improve their estimates.

Another method of improving the estimate is regionalisation. Regionalisation has for many years been a standard hydrological tool, used to facilitate extrapolation from sites at which records have been collected to others at which data are required but unavailable (Riggs,1973). By defining region within which catchments are considered to behave in similar fashion, records may be extrapolated with more precision, and regression equation to predict hydrologic variables in terms of catchment characteristics may be used with greater confidence. However, there is surprisingly little guidance in the literature regarding the details of regionalisation. Many text books such as 'Water in Environmental Planning' (Dunne et al, 1978) mention the benefits of regionalisation without discussing the procedures or the pitfalls.

While applying regional approach, it has to be ensured that the streams behave as nearly similar in hydrologic characteristics as possible; they should have as far as possible similar vegetal cover, landuse, topographic conditions and geologic characteristics. Large basins should not be grouped with very small ones. They should also have similar rainfall and evaporation regions (Linsely et al, 1975). Benson (1952) found that 48 years of record were needed to define the 100 year flood within 25% and 115 years of record to define the same flood within 10% with a 95% probability in both cases. It was considered dangerous to extrapolate a 100 year flood using a 20-year record from a single station (Cole,1965); even if the distribution can be linearised on probability paper, hydrologists are warned against extrapolation because the effect of sampling errors get magnified (Chow,1964).

The flood frequency analysis is the commonly used approach for estimation of design flood, particularly for small and medium structures. When data of sufficient length is available, flood frequency analysis can be based almost exclusively on that record alone. However, when data available is limited or when there is no record available at a site, regional frequency approach provides reasonable estimates of expected floods. The data from a single station is a random sample from an unknown population. It is also noted that not only are the parameters of this population unknown, but the frequency distribution that generated the sample are also unknown as well. Recognising the variability inherent in a random sample of this nature, many hydrologic frequency studies are done on a regional basis (Hann, 1977). Extension of the results of the frequency analysis of the point data to an area requires regional analysis. Various methods of regional analysis have been developed for flood studies (Dalrymple, 1950).

Experience has shown that regional flood frequency curve, as it is known, is a reliable means of predicting flood frequencies and is to be preferred to estimates made from data of a single station (Drayton,1980). Regional analysis overcomes sampling error. Its essential feature is the grouping of data to improve the quality of result. Regional analysis has been developed with variations by the United States Geological Survey in collaboration with a number of State agencies in the period since 1945 and appears to have been widely accepted in the United States (Dalrymple,1960).

The index-flood method of regional flood frequency analysis has proved to be a popular way of providing flood information at ungauged sites (Dalrymple, 1960). The derivation of regional distributions consists of three stages:

i) Identification of one or more homogeneous regions;
ii) Selection of an index-flood and derivation of regressions relating the index-flood to basin characteristics;
iii) Selection and fitting of a regional flood frequency distribution to the standardised flood data of each region.

One of the first steps in a regional study is to define the region itself. Some are based on the location of watershed divides, political boundaries, land resource regions and physiographic regions. Regional boundaries are also defined in terms of similarity of flood frequency curves or flow duration curves. Dalrymple (1960) discusses a test for determining whether flood frequency curves for a region can be considered as homogeneous. Some investigators, including DeCoursey (1973), Mosley (1981), Tasker (1982), and Gottschalk (1985), have used multivariate analysis techniques such as cluster analysis, discriminent analysis and principal components analysis for regionalisation of hydrological data. In practice, many hydrologic regions are based upon administrative or similar boundaries which may cut across geologic, climatic and topographic boundaries. The U.K Flood Studies Report (NERC, 1975) commenced its regionalisation with groupings of hydrometric areas. The theory and practice of regionalisation has received the attention of many workers on the geographical sciences but no general methodology for identifying regions is available. However, Grigg (1967) has presented a number of principles of regionalisation on the assumption that it is analogous to classification and should therefore be based upon the principles of formal logic. Monte Carlo studies by Wallis et al (1985) indicate that while regional analysis improves the mean square on average, some specific sites in a region may be adversely affected by regionalisation.

The peak flow data for frequency analysis should satisfy the following assumptions in order to have meaningful estimates:

i) the characteristics of the sample are true representatives of the characteristics of the population;
ii) the statistical characteristics derived from the sample are time invariant;
iii) the data are random;
iv) the data are homogeneous; and
v) the data are accurate and reliable.

For carrying out flood frequency analysis, usually, the following procedures are followed. The historical data on annual peak flow from the gauged streamlets and rivers of Kerala river basins have been processed. Data from stations which have long periods of data and which are free from influences of man-made structures such as dams, diversions etc. are selected. Simple quality control techniques are applied on "suspect-data" based on enquiries with concerned authorities and by correlating with rainfall data and such other possible methods. The graphical method and analytical method have been used in the flood frequency analysis. In the graphical method, curves were developed for each of the nine flood frequency regions using Gumbel distribution; to get a general idea, a flood frequency curve is developed for the entire Kerala State. In analytical method, some of the theoretical frequency distributions are identified, which are likely to provide good fit to the data series; the chosen frequency distributions are fitted to the historical flood records and the parameters of the distributions estimated adopting one or more parameter estimation techniques. The best-fit distribution is selected on the basis of some goodness of fit criteria; the floods of different return periods are estimated using the parameters of the best-fit distribution. Mean annual peak flow is correlated with basin characteristics, separate relationships for nine regions evolved and correlation matrix for each region formed. This relationship is useful for estimating the T-year flood.

In order to predict the runoff from rainfall, two mathematical models namely, Simple Linear Model and Linear Perturbation Model (Nash et al,1983) have been tested and validated. River flow models are used in flood forecasting schemes. Forecasts of the discharge are obtained in real time, by using the model to transform the input function into a corresponding discharge function of time. The forecasts may subsequently be modified or updated, in accordance with the errors observed in previous forecasts, upto the time of making the new forecast.

1. 2 NEED FOR THE PRESENT RESEARCH PROGRAMME

The problem of floods and their computation is one of the main concerns of hydrologists and engineers, because it is an economically important problem. Estimation of design flood is one of the most important components of planning and design of any water resources project. Flood estimates are required for design of a variety of engineering works including dams, spillways, bridges and flood protection works. Correct estimates of the incoming floods are also required for flood control operations, maintenance of flood levees and for evacuation operations in emergent situations. Use of flood frequency analysis is very common for estimation of design floods of different return periods. Flood frequency analysis deals with univariate process comprising of maximum peak flow values.

The flood frequency studies are not only useful in flood loss prevention but also for purposes of planning and design of hydraulic structures, urban areas and human settlement, and for planning other facilities like transport, communication, etc. Floods affect many of the engineering structures such as bridges, embankments, tanks, levees, reservoirs etc. Varshney (1979). While designing these, proper precaution must be taken for the safe passage of the maximum flood expected. The problem of safe design would call for an estimate of the probable maximum flow, which would occur during a specified period of time. It is only possible to have an estimate of the maximum flow, which can reasonably be expected with a particular probability. This can be achieved through statistical methods using the techniques of frequency analysis.

Streamflow data are being collected in Kerala since 1964, but detailed flood frequency study based on historical data was not carried out. According to CWRDM (1983), a detailed flood frequency based on historical data could not be attempted so far in the region, since actual measured data on streamflow was not available for such a purpose. However, since river gauging has been carried out in Kerala since about thirty years by different agencies like Public Works Department (PWD) (now Irrigation Department), Kerala State Electricity Board (KSEB), Central Water Commission (CWC) and of late by CWRDM, it is time to attempt flood frequency studies.

Since a number of streamlets and rivers are not yet gauged in the region, the necessity for regionalising the study has been realised. This is significant because some of the ungauged rivers are awaiting exploitation for the development of the region (CWRDM, 1983, PWD, 1974).

The present regional flood frequency study is aimed at preparing a material which will provide a basis for decision-making in water resources planning. This study will also serve as guideline for similar studies in future.

1. 3 LIMITATIONS OF FLOOD STUDIES IN GENERAL

The objective of statistical flood frequency analysis is the inference of form and parameter values of the statistical distributions that characterise the population on the basis of the sample of recorded data.

The primary purpose of flood frequency analysis is to define a relation or distribution that conform well to the data and represent an orderly variation of probability rather than the chance erratic variation usually found in a flood series (Singh et al, 1972).

It will be difficult if not impossible, to assure total security or economy in planning protection works against floods at great costs (Varshney, 1979). Therefore, while planning any system of flood control, it is desirable to ascertain the magnitude and occurrence of all great floods in the past and their likelihood of being exceeded in the future.

The unpredictability of floods will be clear from the examples given below:

(i) On the Kaveri river(south India), at the time of construction of Krishnassagar dam, an examination of nearly a 100-year record revealed that the maximum flood was 5925 cumec. Based on this, the spillway was designed for 7100 cumec. In July 1924, when the dam was still under construction, a flood of 8200 cumec was observed, and therefore the design flood had to be revised. The spillway capacity finally adopted was 11330 cumec.

(ii) The flood in Republican river in Nebraska (U.S.A) with a catchment area of 31,800 sq.km., mostly semi-arid, had not exceeded 710 cumec in 80 years, but it suddenly experienced 7950 cumec (i.e, 10 times the earlier maximum) in May 1935.

(iii) The Devils river in Texas (U.S.A) had a 30–year record showing a maximum of 1000 cumec (catchment area of 10530 sq.km.). Two dams were built on the river in 1930 and two years later a flood of 17000 cumec occurred. This was classified by Creager as a flood of 10000 year frequency and yet they had to wait only two years for it.

A few aspects of flood problem are capable of being formulated:

(i) There is proof that small floods occur more frequently than large ones;

(ii) The probability of occurrences of major floods is the greatest at a certain time of the year and in the case of Indian rivers, it is the middle of monsoon season (i.e. in the month of July or August);

(iii) More or less definite relation exists between heavy rainfall, the rate of its runoff from the land and the flood discharges in the river; and

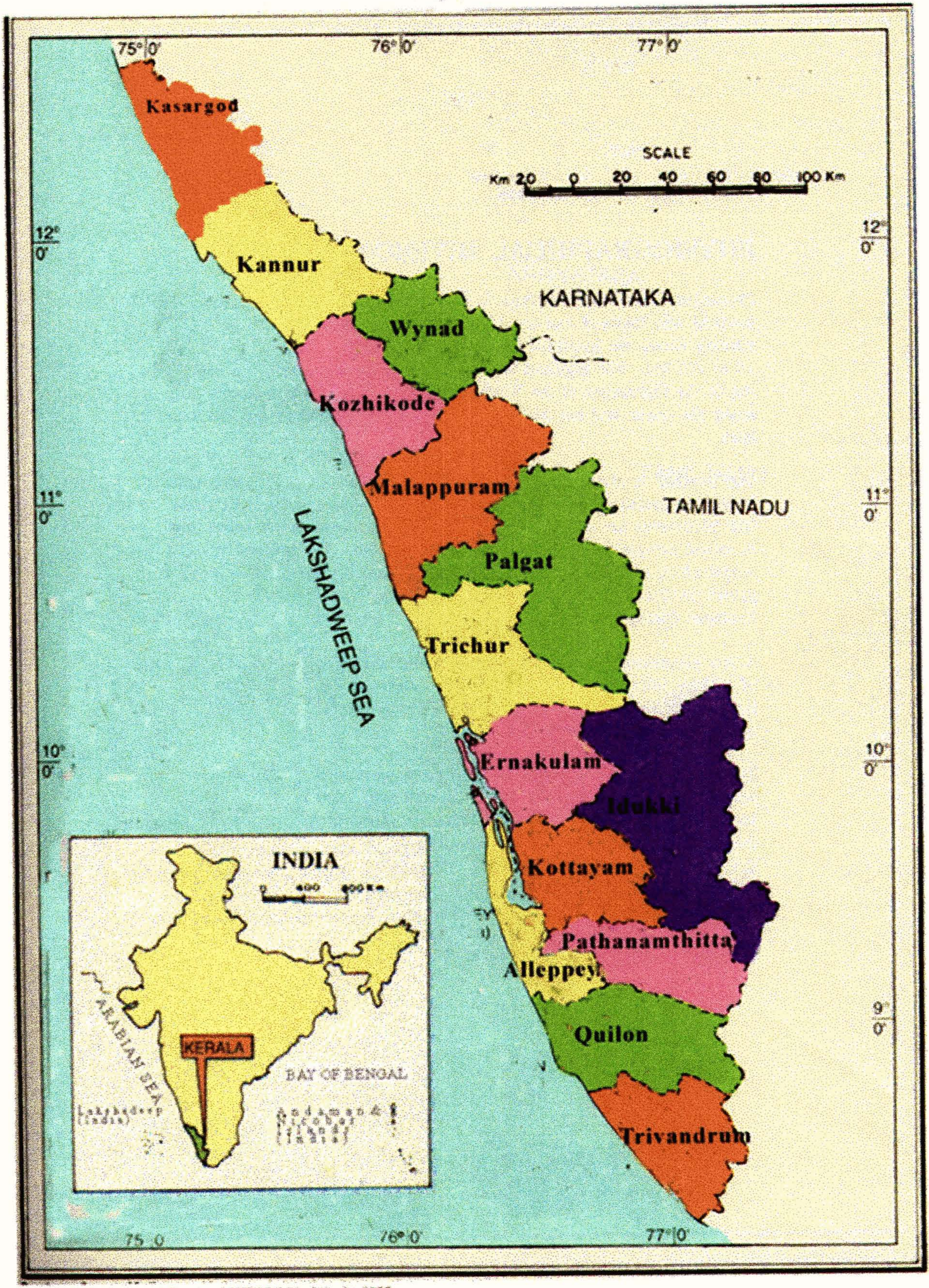
(iv) The longer the period of flow observation in a large region, the greater is the largest observed flood event.

No method is perfect in predicting with certainty the time and size of any flood that may take place in future. Their occurrence and magnitude are as uncertain as the meteorological phenomena and the contributing factors which cause them. Since rainstorms are the principal cause of floods, a study of greatest storms of past is usually of assistance in estimating the frequency of large floods.

1.4 AREA OF STUDY : KERALA COAST

General

The present flood frequency studies concentrate mainly in the Kerala region of the Malabar Coast, situated in the Indian Peninsula between 8°18' and 12°48' N latitude and between 74°52' and 77°22' E longitude (location map is given in Fig.1). As far as possible, the river basins of the region were considered in total. However, this was not always possible because of non-availability of data. The total area of the region considered is about 38864 sq.km, as per the records of Surveyor General of India, with a coastline of about 600 km length. Rivers, along with length, catchment areas are given in the Table 1.



Source: Water Atlas of Kerala, CWRDM, Kozhikode, 1995

Fig. 1 : LOCATION MAP OF KERALA

TABLE 1: RIVERS AND RIVER BASINS OF KERALA

SL No (1)	NAME OF RIVER (2)	LENGTH km (3)	CATCHMENT AREA, km^2			
			KERALA (4)	KARNATAKA (5)	TAMILNADU (6)	TOTAL (7)
1.	MANJESHWAR	16	90	-	-	90
2.	UPPALA	50	76	174	-	250
3.	SHIRIYA	67	290	297	-	587
4.	MOGRAL	34	132	-	-	132
5.	CHANDRAGIRI	105	570	836	-	1406
6.	CHITTARI	25	145	-	-	145
7	NILESHWAR	46	190	-	-	190
8	KARIANGODE	64	429	132	-	561
9	KAVVAYI	31	143	-	-	143
10	PERUVAMBA	51	300	-	-	300
11	RAMAPURAM	19	52	-	-	52
12	KUPPAM	82	469	70	-	539
13	VALAPATTANAM	110	1321	456	-	1867
14	ANJARAKANDY	48	412	-	-	412
15	TELLICHERRY	28	132	-	-	132
16	MAHE	54	394	-	-	394
17	KUTTIYADI	74	583	-	-	583
18	KORAPUZHA	40	624	-	-	624
19	KALLAI	22	96	-	-	96
20	CHALIYAR	169	2535	-	388	2925
21	KADALUNDI	130	1122	-	-	1122
22	TIRUR	48	117	-	-	117
23	BHARATHAPUZHA	209	4400	-	1786	6186
24	KEECHERI	51	401	-	-	401
25	PUZHAKKAL	29	234	-	-	234
26	KARUVANNUR	48	1054	-	-	1054
27	CHALAKUDY	130	1404	-	300	1704
28	PERIYAR	244	5284	-	114	5398
29	MUVATTUPUZHA	121	1554	-	-	1554
30	MEENACHIL	78	1272	-	-	1272
31	MANIMALA	90	847	-	-	847
32	PAMBA	176	2235	-	-	2235

(contd...)

Table 1 (contd.)

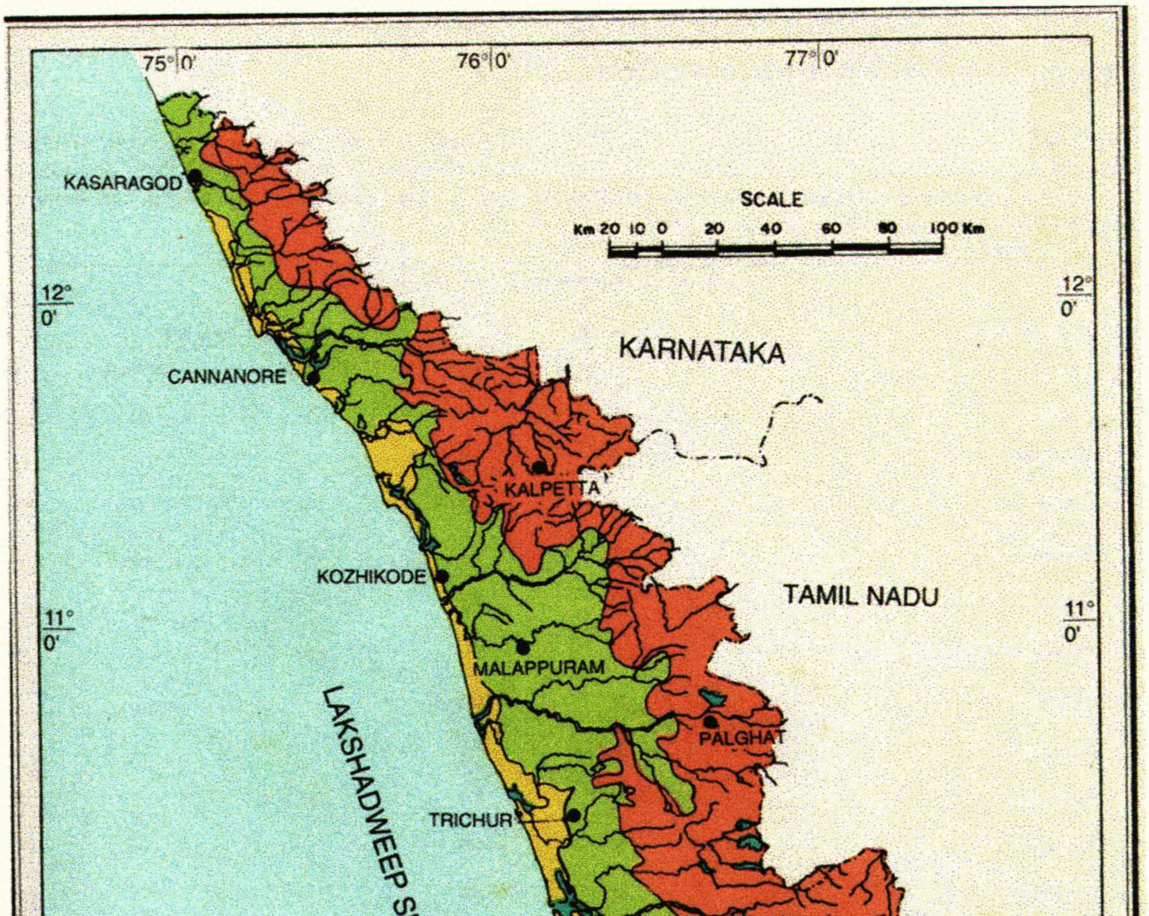
(1)	(2)	(3)	(4)	(5)	(6)	(7)
33	ACHENCOIL	128	1484	-	-	1484
34	PALLICKAL	42	220	-	-	220
35	KALLADA	121	1699	-	-	1699
36	ITHIKKARA	56	642	-	-	642
37	AYROOR	17	66	-	-	66
38	VAMANAPURAM	88	687	-	-	687
39	MAMOM	27	114	-	-	114
40	KARAMANA	68	702	-	-	702
41	NEYYAR	56	497	-	-	497
42	KABBINI		1920			
43	BHAVANI		562			
44	PAMBAR		384			

*Source : Water Resources of Kerala, PWD (Kerala), Trivandrum, 1974

Physiography

Physiographically, the Kerala State is divided into three natural zones: the lowland, the midland and the highland as given in Fig.2 (PWD,1974). These zones form parallel belts running across the length of the State; the width of the State varies from 15 to 120 km. The highland lies on the eastern boundary comprising of the high ranges of the Western ghat; the lowland is a narrow strip along the coast; the midland lies between the highland and the lowland (CWRDM, 1995).

The lowland is characterised by numerous lagoons and backwaters such as the Vembanad, the Ashtamudi, etc., which receive drainage from the rivers. In order to drive out flood waters from the Vembanad backwater system with five rivers draining into it and with only one mouth at Cochin, a spillway was constructed at Thottapalli by operating which flood waters could directly be discharged to the Arabian sea through a short cut. The backwaters of Kerala are connected up by a network of artificial canals, which help in inland navigation for a length of about 558 km from Trivandrum in the South to Tirur in the North. The lowland is often subjected to salinity intrusion (James,1985).



Source : Public Works Department, Kerala (1974)

Source: Water Atlas of Kerala, CWRDM, Kozhikode, 1995

Fig 2 : Physiographical divisions - Kerala

A few kilometres from the sea to the east, the surface gathers into slopes and clustering hills with numerous valleys in between formed due to floods and sediment transport. This undulating midland is characterised by a number of small cultivated watersheds which are locally known as elas (CWRDM,1982).

The Western Ghat, which form the eastern parts of the State, rise from very low altitudes of a few hundred meters upto 2000 m on an average. Most of the reserve forests of the State are in the highland region. The important peaks in the Western Ghats are the Anamudi (2690 m), Mukunti (2550 m) and Nilgiris (2470 m). The Palghat Gap with a width of about 32 km is the largest gap in the Western Ghats. In addition, there are a few other passes in the ghats such as Aramboli, Kumili, Kambam, Thevaram, Bodinaikannur, Karkkur, Periya, Perambadi, etc. The main crops cultivated in the highland are tea, coffee, rubber, pepper and cardamom (CWRDM,1995).

Morphology and Drainage

The ancient drainage pattern in the region belongs to the break-away from East Africa and apparently upto Miocene times been low relief drainage, probably most of the times with a savannah-like or semi-arid climatic region (Neilson,1972). The Bhavani river in the Nilgiris was supposed to have drained westward, but sharply turned eastward during uplifts. There would never have been enough water from the very small catchments with rather dry pre-Miocene climates to have cut the deep gorges in solid charnakite or gneissic rocks, and neither could they have been cut to that extent during the later uplift (Lake,1891). From an examination of the stretch between Ponnani river and Palghat gap, it is seen that the elevated mountain areas are drained in a more regular pattern, and foothills are developed, but the monsoon being a geologically 'young' wind, the drainage development of the lower slopes and foothills are also 'young' (IMD,1958). The geographical changes have played a major role on the climatic changes, and these have again influenced denudation and sedimentation.

There are 44 rivers in Kerala flowing within the region with a minimum length of 15km (Fig.3). Out of the 44 rivers, 41 originate from the Western Ghats region and flow towards the west through midland and lowland and join the Arabian Sea (Laksha Deepu Sea) while three of these rivers originate from the Western Ghats and join the Bay of Bengal, flowing through the neighbouring States (CWRDM, 1983). The State receives a fairly heavy rainfall annually from the South-West and North-East Monsoons. The rapidly falling terrain, the heavy precipitation and the narrow width of the State have given rise to numerous rivers (PWD,1974).

Meteorological Aspects

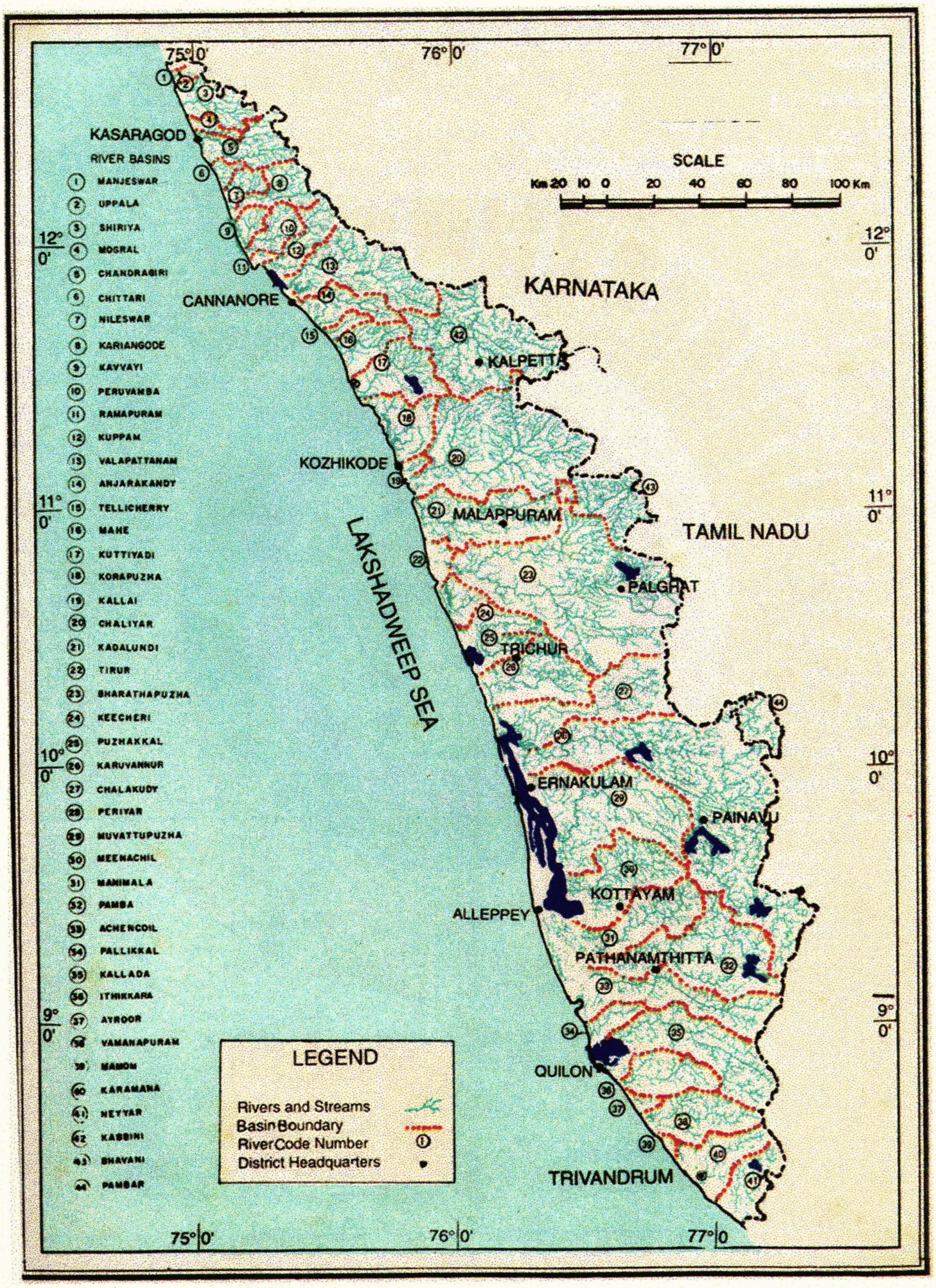
Rainfall

There are two distinct rainfall seasons in Kerala viz. the South-West (locally known as Edavapathy - June to August middle) and North-East monsoon (locally known as Thulavarsham - September to November end). During these two seasons, about 80% of the total annual rainfall is precipitated in the State. The South-West monsoon accounts for 60% of the total annual precipitation. The rains are occasionally accompanied by thunder and lightning.

Isohyetal map of Kerala is given in Fig.4. The average annual rainfall of the State is 3000 mm. The annual rainfall in the lowland ranges from 900 mm in the south to 3500 mm in the north. In the midland, annual rainfall ranges from 1400 mm in the south to 4000 mm in the north. In the highland, the annual rainfall varies from 2500 mm in the south to about 5000 mm in the north. The morphology of the ghats is attributed to be one of the major factors deciding the rainfall pattern (CWRDM, 1995).

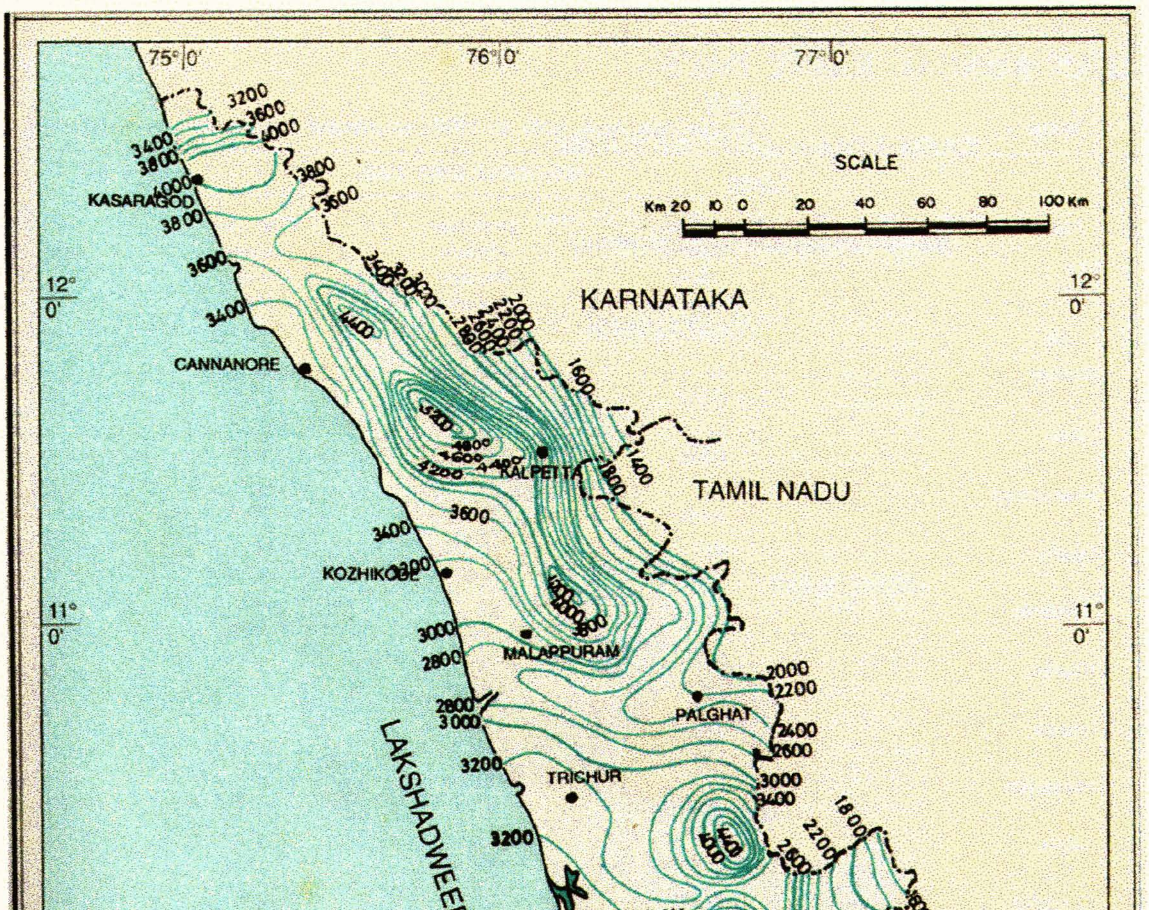
Temperature

The temperature in the region varies between 21°C (69F) and 37°C (99F). The seasonal and diurnal variations of temperature are not uniform throughout the State.



Source: Water Atlas of Kerala, CWRDM, Kozhikode, 1995

Fig 3 : Drainage map of Kerala



Source: Water Atlas of Kerala, CWRDM, Kozhikode, 1995

Fig 4 : Isohyetal map of Kerala

The diurnal monthly and yearly variations are small, so much so that processes of growth and decay are not hampered by temperature, but controlled by rainfall (Nielson,1972). The study of CWRDM shows that the stations located near the coast are influenced by land and sea breeze and here the seasonal and diurnal variations of temperature are almost of the same range (5°C - 7°C). At Palghat, the mean seasonal variation is less than the diurnal variation. But in the high ranges, which are typically sub-tropical, the diurnal variation is very high (>15°C in some months).

The percentage humidity is very high in the coastal region and it varies between 95% in July-August and 60% in January. There is a progressive decline in humidity and temperature from the coastal belt to the Western Ghats.

Geology

The area is mainly covered by crystalline rocks, such as charnockites, khondalites, gneisses, and Dharwar schists of precambrian age. The charnockites are exposed in all districts; khondalites and gneisses are concentrated in the southern part of the State and Dharwar schists and gneisses are exposed in the northern part of the State whereas intrusive granites and dolerites occur in limited areas of the State only. Pegmatite and quartz veins cut across all the rock formations all over the State (CWRDM,1995).

Late Tertiary sedimentary formations, equivalent to the Rajahmundry sandstone, form a linear coastal outcrop to the north of Cannanore and from Kottayam to beyond Trivandrum in the south. These, together with alluvium, cover an area of about 6000 km^2 (CWRDM,1995). The sediments are over 300 m thick in the widest part of the outcrop between Sherthalai and Karunagappally. The top of the beds is 70 m below MSL at Cochin, and rises to 80 m above MSL south of Trivandrum. They are of riverine origin, and comprise from bottom to top (i) Vaikom beds of over 100 m thickness of sandstone-clay with four lenticular granular horizons, (ii) Quilon beds of fossiliferous limestone, marl, sand and clay of about 70 m thickness, and (iii) Varkkallai beds of about 80 m thickness of two persistent interconnected gritty sandstone horizons with interbedded clay and lignite. The Quilon beds do not persist eastward, and thin out 7 km short of the eastern boundary of the sedimentaries.

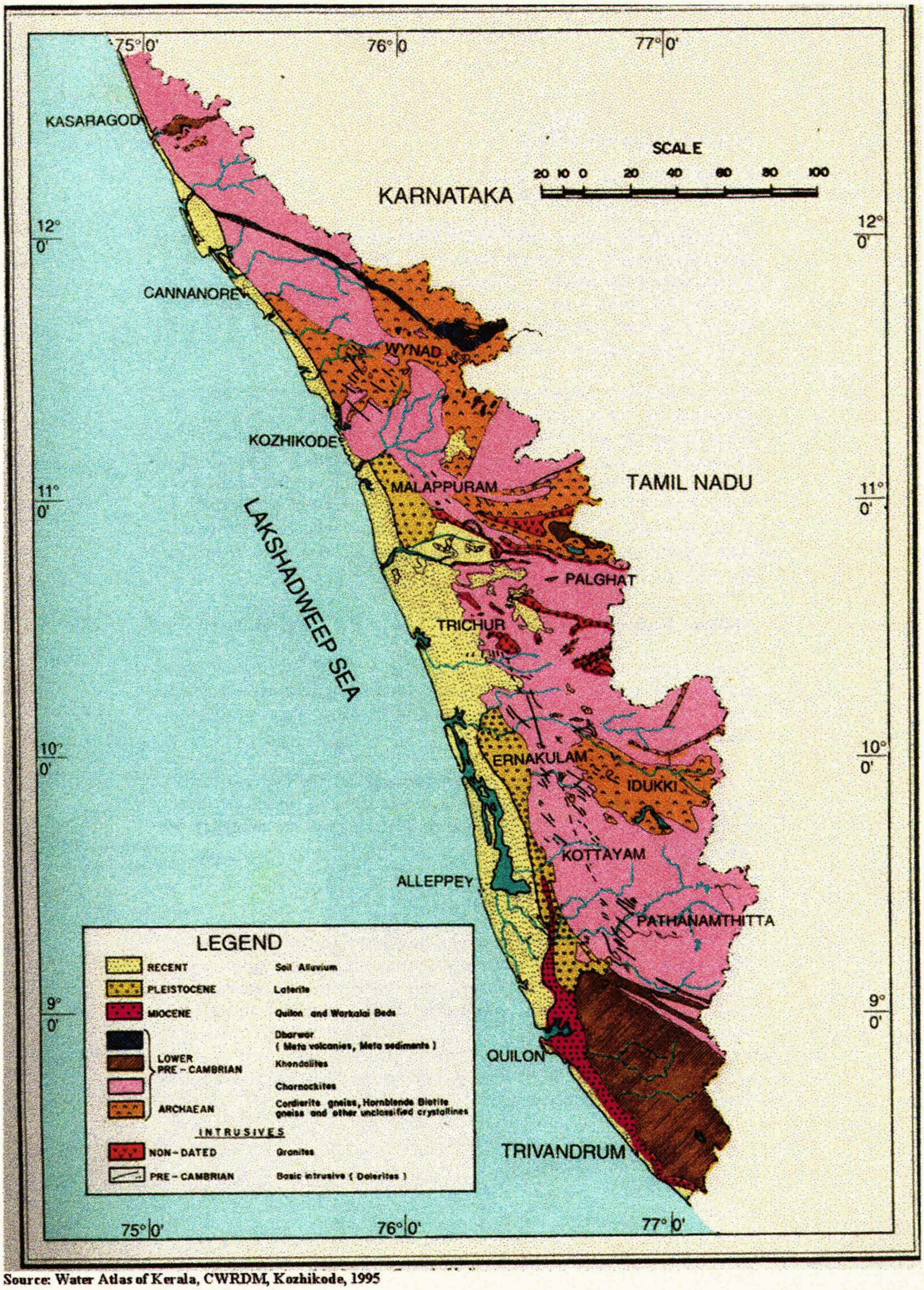
Laterites cover wide areas in Kerala. All along the midland region it forms as a residual deposit due to weathering of either crystalline or sedimentary rocks; the thickness of the laterite generally varies from 5 to 8 m. Plateau laterites of greater thickness is seen in Malappuram, Kozhikode and Canannore districts. Lateritic soil is predominant in the midland regions.

Alluvium overlying laterite is 14 to 20 m thick. It extends all along the coast from Kasargode to Trivandrum. It is essentially sand, clay and silt. The geological formations of Kerala is given in Fig. 5.

Hydrogeology

The groundwater in the coastal region predominantly occurs under water table conditions in the alluvial deposits of recent age and these aquifers are normally a few metres in thickness. In some stretches, groundwater occurs in the upper tertiary sedimentary formations of Vaikom, Quilon and Varkkalai beds capped by laterites. In these areas, the groundwater occurs under artesian conditions. The piezometric surface of these confined aquifers has been found to vary between 0.5 to 14.3 m above MSL. The Vaikom aquifers having thickness of 25 to 79 m are the highest groundwater potential aquifers in the coastal region. The sedimentary tract between Quilon and Shertalai is suitable for medium capacity tube wells.

In the midland region, groundwater is commonly encountered under water table conditions in the lateritic aquifers of about 10 to 20 m thickness. Dug wells of relatively large diameter of about 4 to 6 m are the common groundwater extraction structures in midland. The Laterite are underlain by weathered rock and the two are usually separated by lithomargic clay zone which is prone to caving and hence in such formation, laterite brick lining or concrete rings are necessary for open wells.



Source: Water Atlas of Kerala, CWRDM, Kozhikode, 1995

Fig 5 : Geological formations in Kerala

In the highland region weathered zone forms the aquifer in this region. Groundwater occurs under water table condition and the same can be exploited through dug wells. Deep seated fracture or fault zones in the hard rocks usually also hold groundwater and the same can be exploited through bore wells. Fig.6. shows in detail the hydro-geologic set-up of Kerala.

Soils

In general, the soils are dominated by lateritic soils and forest loams. Other soil type have developed in certain areas due to local physiographic factors. Based on the physico-chemical properties and morphological features, they are classified into ten broad groups , which could be identified with thirteen 'general groups' of soil taxonomy(CWRDM, 1995). Brief description of soil types are presented.

Coastal Alluvium (Tropopsamments – Tropofluvents)

This soil has originated from recent deposits, predominantly marine, with some fluvial sediments along the coast line. The soil is immature with high sand content and low water holding capacity with pH value less than 6.5 in most of the areas.

Riverine Alluvium (Tropofluvents – Eutropepts –Dystropepts)

This type of soil, developed along river valleys, occurs throughout the State cutting across the extensive laterite soils. The soil is very deep with surface texture ranging from sandy loam to clay. It is very fertile having high water holding capacity and plant nutrients which are regularly replenished during floods. It supports cultivation of paddy, arecanut, pepper, tapioca and a wide variety of vegetables.

Red Loam (Tropudalfs – Eutropepts)

This occurs mainly as colluvial deposits in isolated patches in foothills and hillocks being associated with laterites. The red colour is ude to the presence of iron oxides. The soil being highly porous and friable is not fertile.

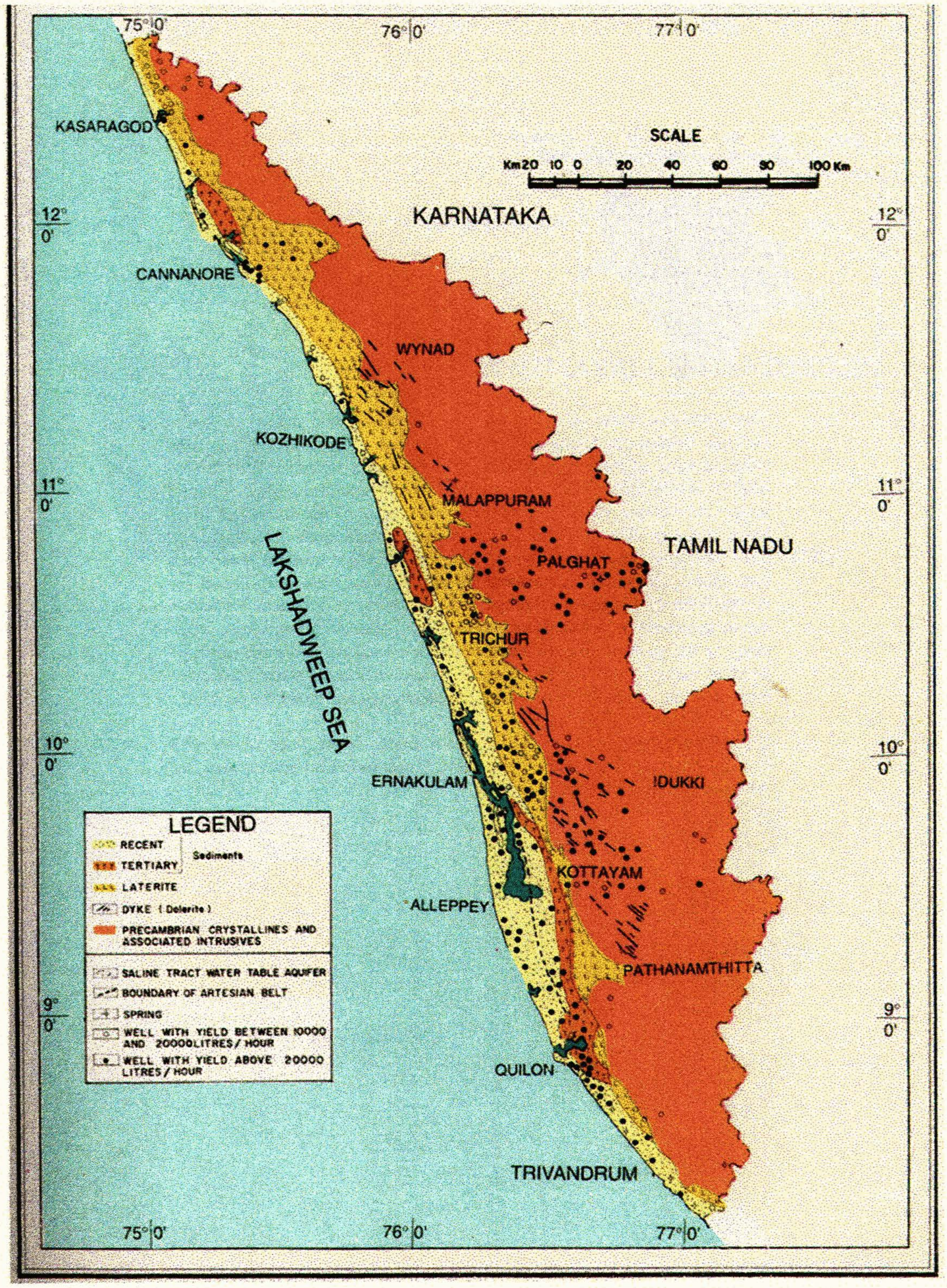


Fig 6: Hydrogeology - Kerala

Laterite Soil (Eutrorthox – Haplorthox – Dystropepets)

This soil, a typical weathering product under humid tropical conditions, occurs throughout the State. It shows the development of AB(c) profiles, which are deep to very deep. The B horizon is well developed in most cases with abundant ferruginous and quartz gravels. Though this soil, in general, is acidic and poor in available nitrogen, phosphorus, potash and organic matter, it is well drained, widely cultivated and responds to management practices. A variety of crops like coconut, tapioca, rubber, arecanut, pepper, cashew etc, can be successfully grown by the proper application of fertilisers and irrigation.

Greyish Onattukara (Troporthents)

This soil with its characteristic grey colour occurs in the districts of Alleppey and Quilon. It is generally coarse grained, highly porous with limited capacity for retaining water and fertilizers. These soils are acidic and are extremely deficient in major plant nutrients. Additions of sufficient organic matter and irrigation facilities improve the water holding capacity for cultivation of paddy, tapioca and other seasonal crops in addition to coconut.

Acidic Saline (Tropaquepts – Fluvaquents)

This soil is found mainly in the Kuttanad region. Developed under hydromorphic conditions, these include the *Kari* soil (black soil with high organic content developed in low lying water logged areas), *kayal* soil (in reclaimed areas with high clay content) and *karappadam* soil (soil along river courses with high silt content). Salinity and waterlogging have put limitations to crop culture but with careful management, these soils can sustain good crop production. Paddy is successfully grown in this soil.

Brown Hydromorphic Soil (Tropaqualfs – Tropaquepts)

This is commonly found in areas of wetlands and is moderately rich in organic matter, nitrogen and potash and deficient in lime and phosphate. Acidity is a problem in some places. For poorly drained areas, provision for drainage is essential.

Hydromorphic Saline Soil (Tropaqualfs)

The saline soil of this group is observed along the coastal strip where inundation by sea causes salinity. The problem of acidity is also observed within this soil group in some areas.

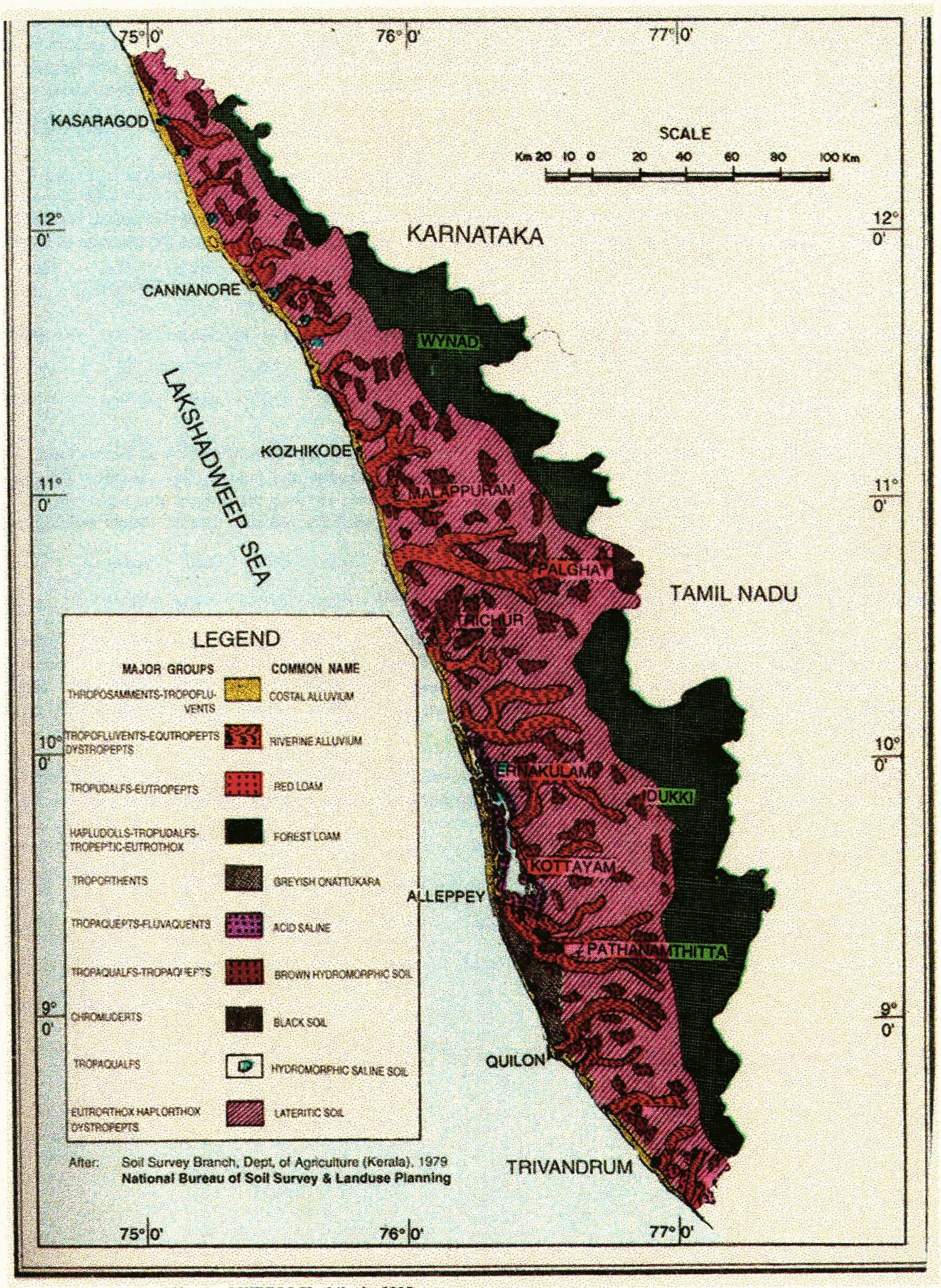
Black Soils (Chromuderts)

Black soil is found in the north-eastern part of Palghat district adjacent to Coimbatore district of Tamil Nadu. This soil is dark in colour, low in organic matter, calcareous, moderately alkaline and high in clay content. This higher proportion of clay makes it sticky and plastic in character. The shrinking-swelling capacity is also high. As this soil promotes cotton cultivation, it is often referred to as black cotton soil. Due to low organic matter and high clay percentage, it is found suitable for a limited variety of crops.

Forest Loam (Hapludolls – Tropudalfs – Tropeptic – Eutrorthox)

This soil is developed in the eastern part of the State within forest area on the weathered crystalline rocks. The upper layer is highly enriched with organic matter derived from the decomposed leaves. Due to the presence of excessive organic matter, the soil is dark reddish brown to black in colour. It is rich in nitrogen, but poor in bases. The soil is quite fertile under forest cover and promotes prolific undergrowth. In denuded areas, protection against soil erosion is recommended.

Soil map of Kerala is given in Fig.7.



Source: Water Atlas of Kerala, CWRDM, Kozhikode, 1995

Fig 7 : Soil map of Kerala

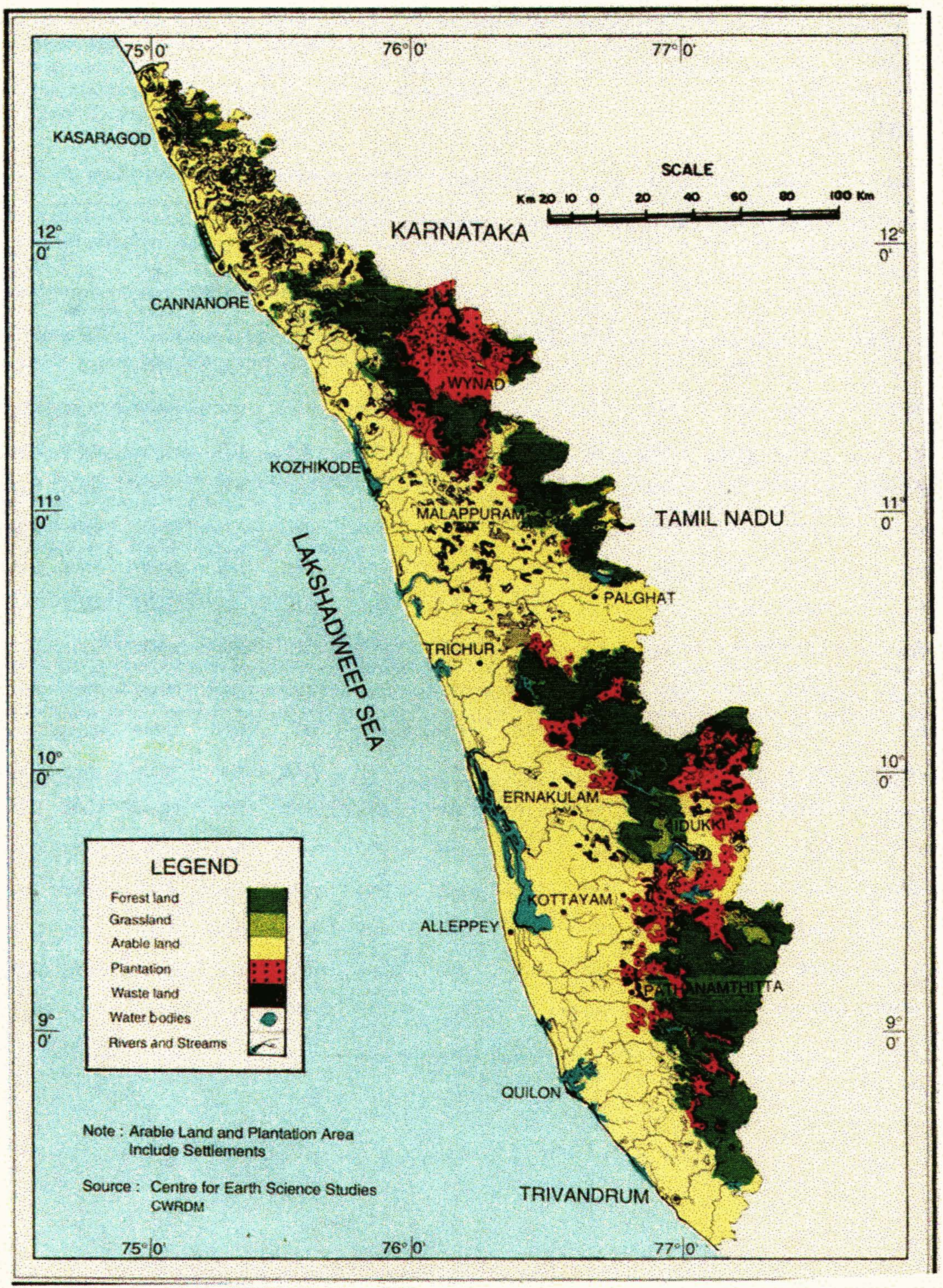
Landuse

The district-wise landuse pattern in Kerala is furnished in Table 2 below and landuse of Kerala is given in Fig. 8.

TABLE 2 : LANDUSE CLASSIFICATION IN DISTRICTS OF KERALA (1985-86)

District	Total Geographical area	Forest	Land Put to Non-agrl.Uses	Barren & un-cultivable Land	Permanent Pastures & Grazing Land	Land under Miscellaneous tree crops	Cultivable Waste	Fallow Other than Current fallow	Current fallow	Net area sown	Area sown more than once	Total Cropped Area in hectares
Trivandrum	218600	49861	17815	2438	31	222	2378	1474	1364	143017	74994	218011
Quilon	251838	81438	23554	882	26	284	801	905	1153	142795	80676	223471
Pathanamthita	268750	155214	9168	948	6	158	512	531	1112	101101	10260	111361
Alleppey	136058		26540	467	10	134	2091	1287	2510	103019	53015	156034
Kottayam	219550	8141	20169	2124	47	280	1259	2255	2702	182573	55933	238506
Idukki	514962	260907	13969	19215	2082	14320	35270	1245	1983	165971	30616	196587
Ernakulam	235319	8163	34628	2433	156	1114	5315	2312	2808	178430	68745	247175
Trichur	299390	103619	22653	2261	136	1361	5503	3087	4891	155879	63102	218981
Palghat	438980	136257	30223	13295	237	8581	24698	4204	5436	216049	102392	318441
Malappuram	363230	103417	19638	7845	320	3054	14463	4343	8876	201274	33921	235195
Kozhikode	233330	41386	17795	1944	111	2849	2949	1376	2451	162469	41876	204345
Waynad	212560	78787	5724	2078	144	3419	4841	15112	1852	114203	31174	145377
Cannanore	296797	48734	22365	14113	530	8575	6464	2348	4167	189501	21941	211442
Kasagod	196133	5625	14360	13064	387	5877	19015	1159	1942	134704	6922	141626
STATE	388497	1081509	278601	83107	4223	50228	12559	28038	43247	2190985	675567	2866552

•Source: Water atlas of Kerala, CWRDM, Kozhikode, 1995



Note : Arable Land and Plantation Area Include Settlements

Source : Centre for Earth Science Studies CWRDM

Source: Water Atlas of Kerala, CWRDM, Kozhikode, 1995

Fig 8 : Land use - Kerala

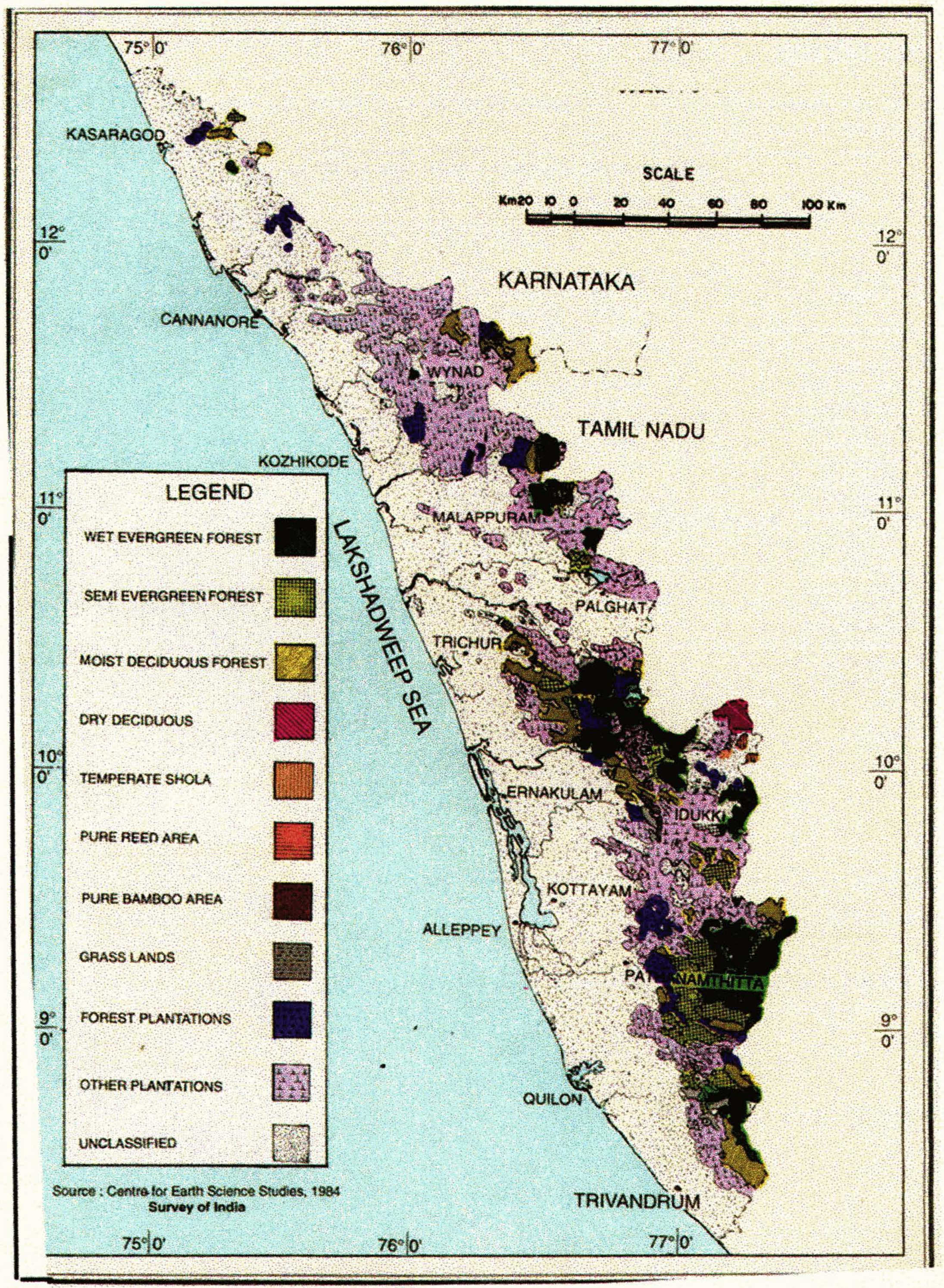
VEGETATION

The details of different forest types of Kerala are given in Table 3. Fig.9 gives the classification of vegetation in the forests of Kerala State.

TABLE 3: DETAILS OF DIFFERENT FOREST TYPES OF KERALA

Type	Total Area (km^2)	Important Species	Remarks
Evergreen and Semi-evergreen	4750	First storey: Acrocarpus fraxinifolius Antiaris toxicaria, Calophyllum, Cullenia exarillata, Dichopsis Ellipticum, Dipterocarpus indicus Hope a parviflora,, Mesua terrea etc.	Locations:Thenmala, Achencoil-Kakki belt Periyar, Sholayar Attappady, Silent - Valley, New-Amarambalam
Sub type: Cullenia-Palaquium Association Palaquium-Mesua Association Poeciloneuron Palaquium Association Mesua-Calophyllum Association Vateraia-Cullenia Association Mesua-Cullenaia Association Reed-Calophyllum Association Reed-Poeciloneuron association		Second storey: Actinodaphne hookeri, Baccaurea courtallensis, Canarium strictum, Cinnamomum Zeylanicum, Elaeocarpus, Holigarna, Nephelium longana etc. Third Story: Euonymus anqulatus, Lea sumbucina, Clerodendrom infortunatum, Veronia etc.	
Moist Deciduous	3140	Tectona grandis, Dalbergia, Dalbergia latifolia, Pterocarpus marsupium Artocarpus hirsutus, Adina cordifolia, Xylia xylocarpa, Lagerstoreemia lanceolata, Grewia Tillifolia, Bombax ceiba, etc.	Important timber species are only given
Dry Deciduous	170	Santalum album, anogeissus bamboos, etc.	Popularly known as sholas, commercially not important, ecologically Significant
Montane Temperate	160		

*source : Water Atlas of Kerala, CWRDM, Kozhikode, 1995



Source: Water Atlas of Kerala, CWRDM, Kozhikode, 1995

Fig 9 : Vegetation - Kerala

LITERATURE SURVEY

Soney John “Statistical analysis of peak flows with special reference to Kerala river basins” Thesis. Department of Statistics, University of Calicut, 1999

2. LITERATURE SURVEY

2 . 1 FlOOD STUDIES

2.1.1 General

Floods may be defined in a variety of ways according to type, origin and magnitude (Wolf,1966). There is a wide range of definitions of floods in terms of their magnitudes of discharge, from the upper quartile of flow duration record proposed by Leslie (1851) to the 'maximum possible flood' resulting from the coincidence of the highest possible precipitation and rate of melting of snow with a minimum loss of runoff. For the purpose of present thesis, the definition is narrowed down to exclude coastal inundations altogether, and flood means high rates of discharge which usually, but not necessarily, cause inundations.

Chow (1956) has defined flood as a relatively high flow, which overtakes the natural channel, provided for runoff. Restvedt et al (1968) defined flood as any high streamflow which overtops natural or artificial banks of the stream. In a strictly hydrological sense, a flood may be any relatively high water level or discharge above an arbitrarily selected flood level or flood discharge.

The main objective of flood estimation is to establish the basis for design and for operation of river works and structures in such a way that the maximum possible flood can be discharged without risk of such failures as may be the cause of loss of human lives.

The flood problem has been discussed at length in publications by Leopold et al (1954) and by Hoyt et al (1955). The problem of obtaining basic data, analysing them for every need, and getting the information to those who have to make decisions is the theme of the book by Langbein et al (1959). Linsley et al (1949) provide useful summaries of the principles used in flood routing. O'Donnell (1960) refers to a number of publications on the use of computers in flood hydrology.

Isaacson et al (1958), and Crawford et al (1960) have reported on the use of digital computers, while Paynter (1952) advocated the use of analogue computers. Matthes (1934), Edger Foster (1948) and Alden Foster (1924) have addressed themselves specifically to the economics of floods and flood protection works.

Water resources engineers are generally concerned with the amount of water, known as design flood, a hydraulic structure is going to face during its economic life. So estimating the design flood is necessary for planning and design of various hydrology related projects. Some of the methods, usually applied in the field, are described in the following paragraphs.

2.1.2 Empirical Relationships

One of the earliest attempts was to determine flood flows by relating peak flows to basin and precipitation characteristics. Much research has been done in relating flood flows of specified probability to the causative factors such as precipitation, duration and intensity, basin size and various hydraulic characteristics. Because of the complexity of the physical interaction that produce flood flows, hydrologic knowledge is not sufficiently advanced as yet to justify a precise mathematical expression for the definition of the maximum flow to be expected from a given basin. Then, empirical equations are initially developed relating extreme floods of observed occurrences mainly to basin sizes (Creager et al, 1945, Mattai, 1969, Crippen et al, 1977). Empirical methods of computing flood discharges have been and are widely used. Over hundred such formulae have been proposed. Most, if not all, of these are inadequate in evaluating the hydrological factors involved. Beardmore (1862), Horton (1924) and Parde (1954) have all attempted to evolve empirical formulae. Lillie (1924) and Inglis (1949) have evolved such formulae for Indian river basins. During the eighty years following Beadmores's work, there appeared a number of formulae for maximum expected flood peaks in which such peaks are assumed to be a function of one or more of the factors, runoff coefficient and exponent characteristic of soil, geology, drainage pattern etc. Chamier (1897), Kuichling (1889), Lloyd-davis (1906) etc. have worked on empirical formulae. Jarvis et al

(1936) and Richards (1955) have summarised some of these important formulae. Varshney (1979) gives list of empirical formulae, developed for Indian catchments. The inconsistent results from the application of such empirical formulae have made their use very limited.

2.1.3 Hydrograph Approach

Unit hydrograph approach gained more popularity among hydrologists with the introduction of the basic ideas of unit hydrograph by Sherman (1932). This approach involves estimation of design storm and derivation of unit hydrograph. The unit hydrograph can be used to estimate design flood from design storm. The idea, on which this method is based, has grown from the proposals of Sherman (1932) and Bernard (1935) with the confirmatory work of a group led by Jarvis (1936) and Hoyt (1936), and with imperative studies by Zoch (1934,1936, 1937), Snyder (1938), Macarthy (1938), Clark (1945), Linsely et al (1949), Johnstone et al (1949), Nash (1957,1958,1959,1960).

The basic ideas of Sherman are that, very nearly,

- (a) the flood runoff, resulting from one inch of effective precipitation of some unit duration T and a given distribution in space, has a unique hydrograph;
- (b) all flood runoffs may be linearly superimposed, which leads to the conclusion that
  - (i) the proportion of any T-hour hydrograph of flood runoff are constant, all the ordinates being directly proportional to the volume of the effective precipitation fallen in T-hours
  - (ii) the hydrograph of successive falls of T-hour effective precipitation are deduced independently from the unit hydrograph, suitably displaced in time and linearly superimposed to give a total hydrograph ordinate.

There are two branches of unit hydrograph practice. One studies the storm on a given catchment and the resulting flood hydrograph, isolates the unit hydrograph for the catchment, and uses it for flood prediction and for the estimation of design flood on the same catchment (Linsely et al, 1949). Then other takes unit hydrograph from various catchments and correlates a number of hydrograph parameters with storm and with catchment parameters. This synthetic unit hydrograph technique was introduced by Snyder (1938). For the purpose of estimation of a design flood, eg., for a spillway intended to discharge the runoff from a computed maximum possible storm, the unit hydrograph technique is very convenient.

2.1.4 Statistical Methods

Frequency Analysis

One of the important problems in hydrology deals with interpreting from a past record of hydrologic events the future probabilities of occurrences. This problem arises in the estimates of frequencies of floods, droughts, storage, rainfalls, water qualities, waves etc; the procedure involved is called frequency analysis (Chow, 1964).

Flood frequency analysis is one of the most active areas of research since last thirty to forty years. Development of flood frequency analysis may be grouped under different phases. In the early phase, it has been recognised that there is no such thing as a single design flood, but rather choice of different return period floods depending on circumstances; in the middle phase, extreme value theory and the algebraic development of an alternative parametric model types and estimation schemes have been introduced. In the recent phase, models have been developed which are stimulated by the outcome of simulation experiment. But till now, the question of finding the most suitable parent distribution and the most suitable method of estimation procedures is remaining. The scope of frequency analysis would have been widened if the parameters of the distribution could have been related with the physical processes governing floods. Such relationships, if established, would have been most useful for studying the effect of non-stationary and man-made changes in the physical process on frequency analysis. Unfortunately, this parent distribution still remains empirical based on the principle of the best fit to the data. However, development of unit hydrograph based on geomorphological features seems to be a good effort towards the physically based flood frequency analysis. Inspite of many drawbacks and limitations, the statistical flood frequency analysis remains the most important means of quantifying floods in systematic manner.

The frequency analysis of streamflow data has perhaps been first applied to flood studies by Hershal and Freeman (Foster, 1934) in 1880-1890 by means of a graphical procedure of using a flow duration curve. Owing to the dearth of long period of records, the use of probability method for flood frequency analysis was apparently hindered until later years. The Gaussian law of probability or the normal law of errors, is the basic and simplest rule for frequency analysis and therefore used in the early stages of flood frequency analysis (Fuller, 1914). Hazen (1914) discovered that if the logarithms representing the annual floods are used instead of the number themselves, the agreement towards the normal law of errors is closer. This is true because the frequency distributions of annual floods are usually skewed or asymmetrical and the distribution can be suitably represented by such frequency distribution laws as the Galton, or log-normal probability law. Hazen (1921,1930) proposed the use of log-normal-probability paper and developed a procedure for the analysis. A table of factors for computing theoretical frequency curves was later prepared by Chow (1954).

In frequency analysis approach, the sample data is used to fit frequency distribution which in turn is used to extrapolate from recorded events to design events either analytically or graphically. The graphical analysis consists of assigning a probability of exceedence or non-exceedence to each ordered observation and selection of an appropriate probability paper. The probability of exceedence or non-exceedence is attached to each observation on the basis of plotting formula. Many distributions and various ways of fitting them are available (Yevjevich, 1963). The selection of an appropriate distribution for any given flood records from among the alternate distributions is still a subject of continuing investigation. A large number of peak flow distributions are available in literature. Among them, the normal, log-normal, Gumbel, General Extreme Value (GEV), Pearson type III, log-Pearson type III and Wakeby distributions are commonly used in most of the flood frequency studies. In flood frequency analysis, it is required to estimate the values of probabilities based on plotting formula. All of the many existing formulae provide different results, particularly at the tail-ends of the distribution. The existing practice in selection of a particular formula is arbitrary and often Weibull's formula is recommended which provides biased and conservative results (Adamowski, 1981). Gringorton plotting formula has been recommended by Gringerton (1963) and Cunnane (1978) for Gumbel distribution on theoretical basis.

In analytical approach following formula is used to estimate the T-year flood:

$$Q_T = \bar{Q} + K_T S \qquad (1)$$

where Q_T is the T-year flood, $\bar{Q}$ the mean of the sample (annual peak discharge series), S the standard deviation of the sample and, K_T the frequency factor (which depends on the return period).

There are essentially two types of models adopted in flood frequency analysis literature, annual maximum model and partial duration series model. In annual maximum model development, Fuller (1914) introduced the concept of return period. Introduction of theoretical frequency curve was done by Foster (1924). Gumbel (1941) used extreme value distribution for flood frequency analysis. Jenkinson (1955,1969) advised generalised extreme value distribution for annual maximum series. Peaks over threshold model was used in flood frequency analysis by Langbein (1949), Borgman (1963), Todorovic (1970), Cervantes (1983), Cunnane (1979), and Ashkar et al (1983b).

The suitability of time series models has been recognised by Quimpo (1967), Hall et al (1972), Weiss (1973); special type of time series model, namely shot-noise model, was used by NERC (1975) for flood frequency analysis.

Procedures of frequency analysis depends on: (i) the amount and type of data used such as at site data, at site / regional data only without at a site data; (ii) type of model; and (iii) form of distribution and estimating procedure used. For the sites having adequate length of records, frequency analysis may be performed either using at site data or at site/regional data. On the other hand, at site data together with regional data can be utilised to provide most consistent and reliable flood estimates for the gauged sites with limited data records. For ungauged sites, however, only regional data can be used for flood frequency analysis.

Flood frequency reports have been prepared by the United States Geological Survey for Albanna (Peiree,1954), Arkansas (Patterson,1961), Columbia river basin (Rantz et al, 1949), Connecticut (Bigwood et al, 1955), Delaware river basin (Tice, 1958), Florida (Pride,1958), Georgia (Bunch et al, 1962), Illinois (Mitchell, 1954), Indiana (Green et al, 1960), Iowa (Schwob, 1953), Kansas (Ellis et al, 1960), Kentucky (McCabe, 1962), Louisiana (Cragvall, 1952), Maryland (Darling, 1960), Minnesota (Prior, 1961), New york (Robinson, 1961), North Carolina (Forrest, 1961), North and South Dakota (McCabe, 1957), Ohio (Cross et al, 1959), Tennessee (Jenkins, 1960), Utah (Berwick, 1962), Washington (Bodhaine, 1960) and Winsconsin (Ericson, 1961). The different regional analysis methods used in U.K. are discussed in detail in Flood Studies Report (NERC, 1975) and Flood Studies Conference proceedings of the Institution of Civil Engineers (London) (1975). Several comparative studies have been conducted on frequency methods by Lettenmaier and Potter (1985), Gries et al (1981,1983), Kuczera (1982), Lettenmaier et al (1987), Nash (1956), Alexander (1957), Dalrymple (1960), Beard (1962), Thomas and Fiering (1962), Benson (1963), Roche (1963), Yevdjevich (1963), Beard (1974) etc.

Some important works were carried out in India in the past to determine certain relationships between rainfall and runoff, which were supposed to be of use in the flood studies. Kapil (1957) discussed flood estimation method by probability approach. CBIP (1957) published a manual on River Behaviour and Training. CW & PRS (1958) published a report on high floods and their return periods. Nagir et al

(1960) reported methods of forecasting floods in the absence of adequate data. Rangarajan et al (1966) have discussed estimation of floods from scanty data. Rao (1971) reported flood damage and prediction techniques for India. CWC (1972) brought out recommended procedures for estimation of floods. Kumra (1975), Ganguly (1979), Dhar et al (1979), Sighal et al (1979), Chabra et al (1979) have studied different aspects of flood problems in the Indian sub-continent.

Regional Approach

Estimation of probabilistic runoff from the watersheds, where no observations have been taken, is one of the most important aspects for the design and economic appraisal of engineering structures. Regional frequency analysis is one of the procedures which make such estimation possible. The main objectives behind this procedure are to reliably extend the shorter hydrologic records and to transfer hydrologic information from gauged to ungauged watersheds within a hydrologically homogeneous region.

Heterogeneity of regions is undesirable as information is lost and the error of quantile estimation will be inflated. Ensuring homogeneity of a region is essential for regional flood frequency analysis. Several approaches are available in literature to identify homogeneity of watersheds. In New Zealand, Mosley (1981) used cluster analysis to form groups of basins characterised by specific mean annual flood and coefficient of variation, while Waylen et al (1984) formed groups of stations in British Columbia on the basis of five different parameters of the Peaks-Over-Threshold (POT) flood series and differentiated between them using discriminent analysis. White (1975) used a factor analysis of a set of basin characteristic data to identify collections of physically similar basins in Pennysylvania and Acreman, and Sinclair (1986) have used cluster analysis on a matrix of basin characteristics to identify similar basins in Scotland. One significant innovation was by De Coursey (1973), who used an iterative algorithm to objectively group 90 gauging sites in Oklahoma by discriminent analysis. The graphs were not spatially continuous so that they cannot strictly be regarded as hydrologic regions. Sorman (1986) used hydrological similarities of flow regimen for regionalisation.

Several other regionalisation techniques have also been used for flood studies. For regional flood estimation, Fuller (1914), Hazen (1932) used station-year method. Dalrymple (1960) introduced the concept of index-flood. In the index-flood method, observed annual peaks are first standardised by dividing each by their sample mean, then all the standardised observations within a region are used to estimate a dimensionless average frequency curve. For estimating an estimate of the T-year peak at a specific site, the standardised T-year event is multiplied by the at-site mean annual peak flow. Thus, the index-flood method is able to combine at-site information (the sample mean annual peak) with regional information (the standardised T-year peak). Several Monte-Carlo studies (Hosking et al, 1985a, Wallis et al, 1985, Lettenmair, 1985) have shown that the use of the index-flood method can improve the average mean square error of flood frequency estimates at the gauged site in a region when compared to other methods. The index-flood method is very efficient when the region is homogeneous except for differences in mean annual peaks (Tasker, 1982). Cole (1965) applied the index-flood method for U.K. data. Nash and Shaw (1965) discussed the relation of regional average dimensionless moments to catchment characteristics.

Benson (1962) pointed out the deficiencies in the United States Geological Survey (USGS) index-flood method, proposed by Dalrymple (1960), and suggested many modifications in the USGS index-flood method. The index-flood method was used by NERC (1975) for British and Irish conditions using regionally averaged, standardised order statistics in a graphical procedure to estimate the portion of the distribution. This procedure was later implemented for New Zealand (Beable et al, 1982). Wallis (1980) recommended the method based on standardised probability weighted moments for regional flood frequency analysis. About GEV distribution recommended in the Flood Studies Report of U.K., Wallis (1980) feels that its regional application is quite specific for U.K. conditions and therefore studies should be made for GEV distribution for other regions also. Gries and Wood (1981) investigated the use of probability weighted moments (PWM) for improving estimates of flood recurrence quantile events in both gauged and ungauged basins.

Stendinger (1983) proposed an approach for regionalisation based on a log space transformation after taking into consideration some theoretical limitations of the standardisation used in index-flood methods. Kuczera (1983) proposed regionalising the parameters of the Box-Cox power transformation, using an empirical Bayes approach. The method accounts explicitly for unequal sample variances and inter-site correlation. Rossi et al (1984) developed a regionalisation procedure for two components extreme value distribution in which annual floods are assumed to come from two distinct EV1 distributions.

Most extensive work has focussed on the application of the probability weighted moments in regional flood frequency studies for various distributional choices including the Extreme Value (EV) type 1,2 and 3 distributions (EV1, EV2, EV3), GEV distribution and the Wakeby distribution. Various issues involved in regionalisation have been investigated by Landwehr et al (1978, 1979a, 1979b, 1979c, 1984), Wallis (1980, 1982b), Gries et al (1981, 1983), Kuczera (1983b), Hosking et al (1985a, 1985b), and Lettenmaier et al (1985).

Different forms of distribution estimation procedures and goodness of fit criteria are used by many investigators in their at site and regional flood frequency studies. A comprehensive review of such studies may be found elsewhere (Gries, 1983, Potter, 1987).

There have been several studies in the area of regional flood frequency analysis in India. Goswami (1972), Thiru Vengadachari et al (1975), Venkataraman et al (1986), Gupta (1987) and many others have conducted regional flood frequency analysis for some typical regions in India. In most of the regional flood frequency studies, the conventional methods such as USGS method, regression based methods and

Chow's method have been used. Some attempts have been made by Perumal et al (1985), Singh (1985), Huq et al (1986), Seth et al (1987) and others to study the applications of new approaches for regional flood frequency analysis for some of the typical regions in India, for which the conventional methods have been already applied.

2.1.5 Mean Annual Peak Flow and Geomorphologic Features

The estimation of response from ungauged catchments is one of the major problem for the hydrologists. The easiest way of solving this problem is by transforming the information from gauged to ungauged catchments with the help of the regional relationship developed between the parameters of a hydrological model and the catchment characteristics for the gauged catchments.

The mean annual peak flows are often correlated with geomorphologic features for purposes of prediction. Geomorphology, the study of 'terra firma', holds a unique position among a host of geosciences. The various parameters under the investigation of geomorphologist are the land forms, the slopes, the stream network and the natural processes. Geomorphology being the study of landforms has direct influence on the flow processes. Geomorphological studies become much more important in the ungauged catchments since different geomorphological parameters help in regionalisation of hydrological models dealing with the runoff estimations.

Several attempts have been made by various investigators in developing of such relationships for different hydrological models. Snyder (1938) related the unit hydrograph parameters with the catchment characteristics. Nash (1959) developed the relationships between the first two moments of instantaneous unit hydrograph and the catchment characteristics using multiple linear regression approach.

Horton (1932,1945) pioneered the hydromorphometric analysis of drainage basin and provided a rational and systematic base, rather a framework of outline of geomorphological characteristics to relate them to various hydrological properties of the system. Many investigators like Langbein et al (1947), Strahler (1952,1954,1956, ,1964), Melton (1957), Schumm (1956), Chroley (1957), Miller (1953) pursued his work. These works have brought forward many laws of fluvial geomorphology connected to hydrology and study of geomorphological characteristics of river basins has become a major area of scientific research for hydrologists throughout the world.

A large number of studies Laten et al (1940), US Army Corps of Engineers (1954), O'Kelly (1955), Gray (1962), Morgan et al (1962), Mirajgaonkar (1963), Askew (1963), Hopkins (1964), Schulz et al (1971), Body et al (1979) have since appeared on synthesis of unit hydrographs in ungauged areas. Recently, Rodriguez-Iturbe et al (1982), Rodriguez-Iturbe (1979), Gupta et al (1980), Singh (1983) and Pristochova (1990) employed bifurcation ratio, length ratio, area ratio, stream lengths and stream areas of synthesising unit hydrograph.

White (1975) classified 112 basins in Pennsylvania and surrounding states according to drainage density, channel slope, shape factors, and geometric factors for the purpose of evaluating their flood potential. Ebisemiju (1979) used drainage density, stream numbers, stream lengths, and relief for identifying morphologically and hydrologically uniform basins.

Attempts to obtain quantitative assessment from geomorphometric parameters by Dobbie et al (1953) did not prove successful. Rastogi et al (1976) presented a quantitative analysis of drainage basin characteristics for developing various geomorphic relationships in northern part of Garhwal district of Uttarpradesh. A few quantitative geomorphological studies based on the approaches of Horton (1945), Strahler (1950) have been carried out for some of the rivers of the Kerala coast, namely Kuttiyadi, Kabini, Meenachil and Karuvannur (James et al,1983).

2.2 MATHEMATICAL MODELLING

Mathematical hydrology is the functional relation between numbers which are a quantitative representation of the descriptive concepts and processes in hydrology.

Fleming (FAO, 1979) has defined a mathematical model as a 'numerical system interrelating in a given time reference a sample of input, cause or stimulus of matter, energy or information and a sample of output, effect or response of information, energy or matter'.

In any model of a physical system, there is a compromise between simplicity of model and its level of accuracy. Though there are a number of mathematical models discussed in hydrological literature, the rainfall-runoff model suitable for any purpose depends on various factors: (i) existence of measurement of physical parameters of the basin, (ii) reliability of the type (conventional, automatic, etc.) of data from hydrometeorological network, (iii) specific use of the model and the purpose of research, (iv) type of available computing facilities and the economics involved, and (v) time scale of modelling. Experience has shown that when the time scale of modelling is long enough, any month or longer, simple models are often adequate when representing the relation between rainfall and runoff without considering the intervening factors (Fleming, 1975).

Derivation of runoff values from the recorded or predicted rainfall events can be done if a satisfactory relationship between the two events is established. Consequently, a substantial amount of effort in hydrological investigation and research has been directed towards formulating this relationship. In the absence of actual gauged data, hydrologists may have to depend on models, particularly mathematical models, for water yield assessment which is essential for most of the water resources development projects and even some of the activities connected with environmental protection, urban development, construction works and so on.

A brief review of works regarding rainfall-runoff modelling is given in the following paragraphs.

Most discrete time rainfall-runoff models operable over long uninterrupted periods can be classified as explicit soil moisture accounting (ESMA) models. One of the famous and early models in this category is the Stanford watershed model (Crawford et al , 1966), developed in 1959. Some of the other models developed for water yield assessment are: Shih Hawkins-Cahambers model (Shih et al, 1972), USDAHL model (Holtan et al , 1971), and Mero model (Mero, 1969).

Several modifications to the Standford watershed model have been attempted. The Kentuky watershed model (Liou, 1970) is one such adaptation for the more humid eastern United States; this version has more computational efficiency. Another version called OPSET (Liou, 1970) uses optimisation techniques for estimating several of the parameters. A continuous streamflow simulation model meant for large basins was developed in 1958 by Army Corps of Engineers (U S) (1963). This model, known as streamflow synthesis and reservoir regulation model (SSARR) is primarily intended for streamflow and flood forecasting and for reservoir design and operation studies.

Dawdy-O' Donnel model (1965) developed by U S Geological Survey has deliberately maintained the simplicity of the structure of their model. Ibbit (1970) has conducted exhaustive comparisons between this model and Standford model. The Boughton model (1966) originally developed in Australia was meant to simulate water yields from catchments in sub-humid to semi-arid regions. The model, using daily rainfall and evaporation data, provides continuous simulation capability for general purpose use.

Some studies have been carried out in the past in India to establish certain empirical relationship between rainfall and runoff. The approaches of Binnie for the rivers of Madhya Pradesh, Barlow for the rivers of Uttar Pradesh, Inglis for the rivers of Maharashtra, and Kanvar Sain for the rivers of Punjab are worth mentioning (Mohan, 1985). Some other empirical relationships between rainfall and runoff were arrived at by Vermeule, Justin, Parker, Khosla and others (Bhalerao, 1978).

The use of stochastic models in hydrology is an attempt to widen and extend the knowledge on hydrological events and improve the decision-making ability, by generating long hypothetical sequences of events based on the statistical and probabilistic characteristics of past records. These generated sequences of data are then used to identify the components that contribute to error and uncertainty in a proposed design, eg. reservoir storage requirements, projected demands and rainfall variations.

A number of models have been reported using the probabilistic approach for the representation of daily river flows. Quimpo et al (1967) and Quimpo (1968) used a stochastic model for daily river flows. Roesner et al (1966) stuided the monthly runoff series using correlation analysis. An earlier study by Yevjevich (1964) on annual runoff sequences showed that the correlation coefficients were less than those for monthly flows. The increase in correlation with shorter time basis was also obtained by Carrigen et al (1967) while analysing annual floods.

The moving average model, in addition to dampening irregularities in a series, may be used to represent the dependence of one hydrologic event at a given time on another event at the same and at preceding times. This model was used by Matalas (1963) to describe the runoff from the effective precipitation.

Stochastic techniques have been used for synthetic generation of hydrologic data by Thomas et al (1962). They generated sequences of monthly and six-hourly flood flows by a stochastic model for the monthly runoff. Singh et al (1974) used an alternate method to generate monthly flows such that the annual totals retain desirable characteristics, the assumption being monthly flows could be described by a mixture of two normal distributions. Thomas-Fiering technique was later extended by Harms et al (1967) using logarithmic transformation. Assuming normal distribution of historical flows, Brittan (1961) generated annual flows based on a Markov process. Fiering (1964 a) used a Markov model for sequential generations of daily flows which was used for lowflow analysis.

Thomas-Fiering model and autoregressive models based on the Gausssian distribution are likely to be less satisfactory than one which preserves the hydrograph characteristics of rapidly rising limb followed by more gradually decreasing recession. A generating model which preserves the characteristics hydrograph shape and which has been applied to the stochastic generation of daily flows, is the 'shot-noise' model. This was described by Parzen (1962) and applied to the daily streamflow generating problem by Weiss (1973).

A bivariate technique was used by Thomas et al (1962) using the cross correlation of different pairs of gauging stations to generate discharges at several sites in a river basin from the recorded and generated flows at one of the stations and successive application to a chain of stations. Matalas et al (1964) developed a procedure to generate and augment data by utilising relationships between two given hydrologic phenomena such as rainfall and runoff.

Multivariate techniques have been applied using spatial and temporal sequential correlation of hydrologic variables from stations having the same hydrologic and climatic conditions. Such a technique was first introduced by Fiering (1964 b) and then developed further by Matalas (1967 a, 1967 b). Benson et al (1967) adopted a regional analysis for hydrologic data generation for sites with short or inadequate records. For approximating discrete fractional noise, two processes have been developed: (i) the broken-line process developed by Ditlevsen (1971) which is adapted to synthetic flow generation by Mejia (1971) and by Garcia et al (1972), and (ii) the ARIMA process (Box et al , 1970) which has been used to discuss streamflow sequences (Carlson et al , 1970) and adapted to synthetic flow generation by O'Connel (1974).

Auto-regressive moving average (ARMA) time series models have been extensively used of late since it has a physically reasonable correlation structure which can reflect long term persistence resulting from long memory (Mandelbrot et al, 1968), although this reasoning has been argued against by Klemes (1974). Multivariate ARMA (p, q) models have been proposed by Salas et al (1980), Loucks et al (1981), Jenkins et al (1981).

2 . 3 PROBLEM IDENTIFICATION

Streamflow data for longer periods were not available since discharge measurements started in Kerala only after 1960. Initially, gauging stations were very limited; even now the density of stations is not satisfactory for detailed research investigations. In this background, the necessity for attempting a regional approach, combining data of homogeneous regions, has been recognised. By combining such data of nearby stations, it would be possible to reliably extend the shorter records and to transfer the flood information from a specific gauged site to a homogeneous region. Wallis (1980) had recommended further studies using the technique developed for UK (NERC, 1975) in other countries / areas to validate its application in other regions. The present study considers the above mentioned points and attempts a regional flood frequency study for the Kerala region with the available gauged data on streamflow.

Once the mean annual peak flow obtained for a region, further studies were carried out to relate the same with the geomorphological parameters of the Kerala region. This exercise will help in predicting the floods of a particular site in a river basin / groups of river basins in Kerala, for which gauged data are not available.

The empirical methods have several limitations; these methods cannot be used for real-time forecasts. Therefore, two mathematical models developed for rainfall-runoff studies, have been attempted for selected river basins of Kerala.

METHODOLOGY

Soney John “Statistical analysis of peak flows with special reference to Kerala river basins” Thesis. Department of Statistics, University of Calicut, 1999

3. METHODOLOGY

3.1 DATA BASE

The peak flows have been taken out from the actual historical data on streamflow for the purpose of flood frequency analysis. The stream gauging operation is of recent origin in Kerala. The necessity for stream gauging was felt in Kerala during the post-independence period, since most of the projects-irrigation and hydroelectric - were planned and designed after independence, though there were a few exceptions. Initially, flow measurements were restricted to sites near which major or medium projects were envisaged. However, since 1963 there was a tendency to establish a basic network of stream gauging stations in the river basins of Kerala State, which is necessary for the planning and execution of many water related works in the State. The stream gauging works in the region are mainly carried out by the PWD and KSEB; of late, CWC and CWRDM have also established stream gauging stations.

An annual peak flow is defined in the present study as the largest instantaneous flow in any given year. The first step in the present flood study has been to select the flow data from the available river gauging stations having well defined ratings. A typical rating curve (stage vs discharge) of a station maintained by Irrigation Department of Kerala is given in Fig. 10. Inevitably, some degree of extrapolation is required with any rating curve, but ‘excessive’ extrapolation of ratings may produce flood flows, which are unacceptably inaccurate.

The second step in this study is the extraction of the peak flow data from the annual flow records. Most of the events would have occurred either in the south-west or in the north-east monsoon periods. The available data have not been in a processed form and generally recorded on data sheets kept in files. Manual selection was done to select the highest value. ‘Water Year’ for the present study starts from June 1 and ends on May 31, as has been followed by Irrigation Department (PWD,1974).

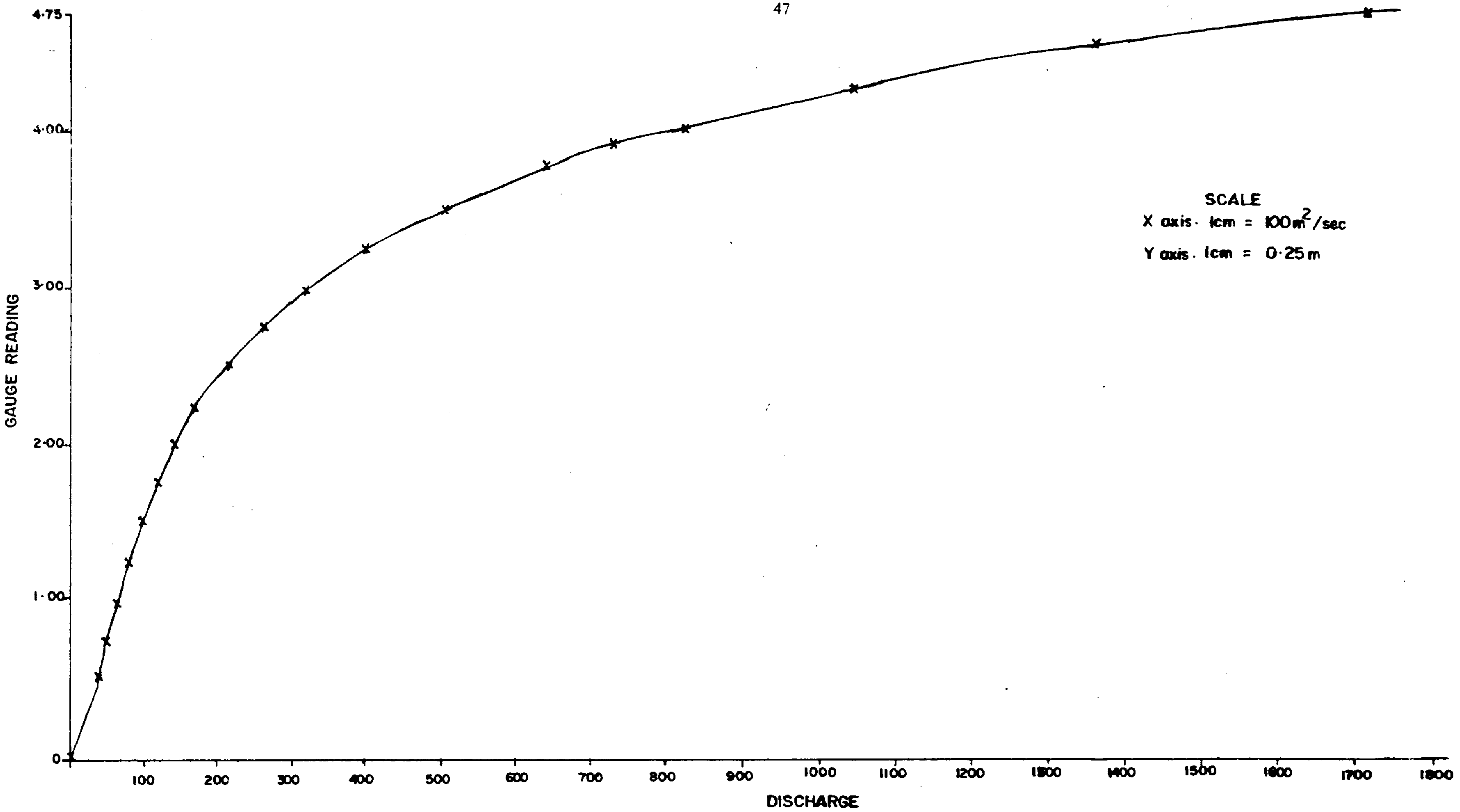


FIG 10 DISCHARGE CURVE OF MANIMALA RIVER — 1994 — LOCATION-THONDRA

SW026/99

The peak flow data available from 98 stations spread out in different river basins of Kerala were collected from the records maintained by PWD, KSEB, CWC and CWRDM. In selecting the stations, the following criteria were considered:

(1) Stations yielding more than eight years data only were taken for the present study;
(2) Stations showing considerable discrepancy in data, when compared to the neighbouring stations, were not considered;
(3) Those stations, downstream of diversions, storage etc. were avoided as far as possible; and
(4) The accuracy of rating curves was another criterion that decided the suitability of a station.

The above precautionary measures taken would not make the data fully foolproof. However, this might have helped in improving the quality of data considerably for the present study. Further, randomness were checked using serial correlation coefficient.

TABLE 4: DETAILS OF STATIONS AND STREAMFLOW DATA

Station no (1)	Station name (2)	Period of data (3)	Depth of data, years (4)
	REGION-1		
1	MANJESHWAR	82-90	09
2	ANNAKAL	82-90	09
3	UPPALA	82-90	09
4	SHIRIY A(U/S)	65-90	26
5	MADHUR	82-90	09
6	PADIATHADUKA	74-90	17
7	PALLANGODE	70-90	21
8	MOONAMKADAVU	65-90	26
9	KAKKADAVU	65-90	26
10	MANGARA	71-90	20

(contd...)

Table 4 (contd)

(1)	(2)	(3)	(4)
	REGION – 2		
11	IRUDE	66-90	25
12	KUILOOR	77-90	14
13	PALA	70-90	21
14	MERUVAMBA I	66-90	25
15	KANNAVAM	70-90	21
16	PATTIAM	66-90	25
17	VALAYAM	65-90	26
	REGION – 3		
18	BAVELI	64-90	27
19	THIRUNELLY	70-90	21
20	THONDAR	72-90	19
21	MANAMTODY	65-90	26
22	CHOORANI	70-90	21
23	PANAMARAM	70-90	21
24	KAKKAVAYAL	71-90	20
25	MUTHANGE	70-90	21
26	VAZHAVATTA	71-90	20
27	MANJAT	72-90	19
	REGION- 4		
30	KADAMTHARAPUZHA	82-90	09
31	KOLLICKAL	74-90	17
32	KOODATHAI	65-90	26
33	MUKKOM	65-90	26
34	AREACODE	65-90	26
35	KANJIRAPUZHA	65-90	26
36	CHALIYAR	65-90	26
37	PUNNAPUZHA	65-90	26
38	MARUTHAPUZHA	64-90	27
39	KARIMPUZHA	64-90	27

(contd...)

Table 4 (contd)

(1)	(2)	(3)	(4)
40	KUTHIRAPUZHA	64-90	27
41	ANAKAYAM	64-90	
	REGION – 5		
42	THIRUVAGAPUZHA	64-90	27
43	THRITHALA	70-90	21
44	CHERUTHURUTHY	64-90	27
45	PAMPADY	64-90	27
46	CHEERAKUZHI	63-90	28
47	MANAKKADAVU	71-90	20
48	KUTTIPURAM	67-90	24
49	SILENT VALLEY	70-81	12
50	PAZHOOR	64-90	27
51	KUNDUKADAVU	64-90	27
52	MANALI	64-90	27
53	PILATHODU	64-90	27
54	CHIMONI	65-90	26
55	MUPPLY	66-90	25
56	KURUMALI	65-90	26
57	KARUVANNUR	64-90	27
58	AMBALAKADAVU	63-90	28
59	KARAPPARA	68-90	23
60	KURIARKUTTY	69-90	22
28	CHITTOOR	71-90	20
29	CHAVADIYOOR	71-90	20
	REGION – 6		
61	KALADY	68-90	23
62	PILANTHODU	68-90	23
63	CHEMMANNAR	68-90	23
64	PERINJANKUTTY	65-90	23
65	KALLAR EASTERN	65-90	23

(contd...)

Table 4 (contd)

(1)	(2)	(3)	(4)
66	THUVALLUR	64-90	27
67	PANNIAR	65-90	26
68	CHAMAPKKAD	73-90	18
	REGION- 7		
69	MUVATTUPUZHA	64-90	27
70	KAKADASSERRY	72-90	19
71	KALAMPUR	68-90	23
72	THODUPUZHA	72-90	19
73	MALANKARA	77-90	14
74	PEROOR	70-90	21
75	PALAI	65-90	26
76	CHERIPPAD	76-90	15
77	TEEKOY	76-90	15
78	THONDRA	70-90	21
79	MANIMALA	70-90	21
80	MUNDAKAYAM	76-90	15
	REGION- 8		
81	ERAPUZHA	70-90	21
82	KURUDAMANNIL	65-90	26
83	KOLLAKADAVU	68-90	23
84	PANDALAM	64-90	27
85	KONNY	82-90	09
86	ANAYADI	81-90	10
87	ENATHU	70-90	21
88	PUNALUR	66-90	25
89	THENMALA	71-90	20
	REGION - 9		
90	AYOOR	70-90	21
91	VALAYANKIL	71-90	20
92	AYIROOR	82-90	09

(contd..)

Table 4 (contd)

(1)	(2)	(3)	(4)
93	MAMEM	79-90	12
94	MYLAMMODE	80-90	11
95	ARAYANAD	74-90	17
96	MARUTHANKUZHY	78-90	13
97	AMARAVILA	73-90	18
98	OTTASEKHARAMANGALAM	70-90	21

Table 4 furnishes the details of stations and also of streamflow data considered in the present study; the table furnishes the data regionwise - the criteria for regionalisation is given in the later part of this chapter. The latitude and longitude of stations are given in Appendix 1. Location of discharge stations are given in Fig. 11.

The quality of data used in the present study might have suffered from one or more of the following limitations:

i) Some gauging stations are inaccessible during heavy monsoon period;
ii) Flood peaks on small catchments, especially those which are flashy, pass the staff gauges without being recorded;
iii) Gauge readings are at times fabricated or taken only once a day;
iv) Floods outflank the control section;
v) Heavy rains are capable of washing away or overtopping the gauge staff;
vi) Data processing at times takes place much after the actual observation;
vii) Frequent occurrences of floods at night often go unrecorded; and
viii) Automatic water level recorders have not become popular in the region.

Rainfall data

The daily rainfall data were collected for all stations from the records of PWD, KSEB, CWC and CWRDM. In most places, rainfall has been measuring using the ordinary raingauge. In some places, automatic raingauge has been using to measure rainfall. In certain locations, rainfall measurements are carried out by private agencies.

3. 2 FLOOD FREQUENCY ANALYSIS

The flood frequency analysis using the conventional procedures can be conducted for the data from the gauging sites, where the recorded peak discharges over number of years are available. However, such analysis will have somewhat limited application for the ungauged sites or sites with short length of record. Such a situation can be overcome by adopting regional approaches and performing flood frequency analysis with data combined from a particular region.

There has been significant developments and studies in the area of regional flood frequency analysis in India as well as abroad. Estimation of the regional flood frequency parameters is preferred to that of the specific site, for two reasons:

(i) Because of the sample variations present in the short hydrologic records, frequency estimates of rare events based on at site frequency analysis are subjected to large error and thus unreliable, this error being reduced by combining data from many more sites;

(ii) There are many more sites in the same region where hydrologic data are not available but design flood estimates are needed for the design of small structures and in such a situation regional flood frequency analysis helps in transferring the knowledge arrived at from gauged sites to ungauged sites.

The present study attempted to provide a flood frequency analysis for nine regions in Kerala State based on which the flood of any return period for any particular region could be estimated irrespective of the availability of the river flow data. While selecting the regions it has been ensured that river basins are taken together (not in part); grouping of such basins are done considering the available data and convenience in handling the data. Homogeneity in rainfall has been one of the important criterions in delineating the regions. A homogeneity test was conducted in the regions, as explained in the subsequent section, to further ensure that delineation is satisfactory. The main purpose is to obtain a region having sufficient gauging stations with data for a comparatively long period. The methods used to derive the relationships are based on those developed during the UK flood studies and as reported in Flood Studies Report (NERC, 1975). These methods have found wide application in different continents, since the report has been brought out, for example in Malawi in Africa (Drayton et al, 1980), Iran, Indonesia and Korea in Asia and Brazil in South America (Sutcliffe et al, 1984).

3.3 SERIAL CORELATION COEFFICIENT

It is necessary to investigate the degree to which the discharge in any one-year is dependent upon the magnitude of the discharge in the years preceding it. Many hydrological sequences exhibit a departure from randomness in that large values tend to be followed by large values, and small values by small values, so that values of similar magnitudes tend to persist throughout the sequence. This tendency can be measured by lag-one serial correlation coefficient (r_1), describing the strength of the relation between a value in the sequence and that preceding it by one time interval. For strictly random sequences, the value of (r_1) necessarily differs from zero only by sampling variation; for sequences showing strong persistence, its value is close to one.

If the data sequence is denoted by $\{y_t\}$, t=1,2....N, the lag-one serial correlation coefficient is calculated by

$$r_1 = \frac{\frac{1}{N-1}\sum_{t=1}^{N-1}(y_t - \bar{y})(y_{t+1} - \bar{y})}{\frac{1}{N}\sum_{t=1}^{N}(y_t - \bar{y})^2} \qquad (2)$$

(r_1) is distributed normally about mean 1/(N-1) with variance $(N-2)^2/(N-1)^3$. If (r_1) lies outside the range

$$\left(-1-1.96\frac{(N-2)}{(N-1)^{3/2}}, \qquad -1+1.96\frac{(N-2)}{(N-1)^{3/2}}\right)$$

then there is evidence that (r_1) is significantly different from zero, unless an unusual event has occurred.

3. 4 REGIONAL HOMOGENEITY

A homogeneous region is essential for the reliable flood estimation. In order to test the homogeneity of a region, a test has been developed by Langbein etc., (1947) and Dalrymple (1960) developed certain methods, which are generally followed in flood frequency studies. Wiltshire et al.(1986) developed a significance test for regional homogeneity based upon the non-exceedence probabilities (termed G points) of individual maxima; this test was applied in the flood studies of UK (NERC,1975). The studies showed that most of the geographical decisions of UK followed in the Flood Studies Report are heterogeneous (Wiltshire et al,1986). The most commonly sought form of homogeneity is homogeneity in the frequency distribution after standardising by an index-flood, which is commonly chosen to be the mean annual peak flow. The test, which is based on the non-exceedence probabilities, was used to check the regional homogeneity in the present study.

For applying this test, a priori selection of distribution of the type of distribution applicable to the region was made and proceeds with an examination of the scatter of site flood data about the fitted region-average distribution. The method is as follows.

If F(x) is the distribution function of the variate X, then F(x) considered as a function of x, the probability integral transform, has a uniform distribution. If random sample values (x_1, x_2, x_3, ... x_N) are transformed into values (G_1, G_2, G_3 ....G_N) in the (0,1) interval by the probability integral transform, then the G values should be approximately uniformly distributed, the departure from uniformity being solely due to sampling effects. On the other hand, if random sample values (z_1, z_2 ..... z_N) from another variate Z, Z is not equal to X, are so transformed their distribution would be different from uniform by more than random amounts. Thus if F(x) is the regionally estimated distribution function of the variate $X = Q / \bar{Q}$ and if (z_{1j}, z_{2j}..z_{ij}, z_{Njj}) are a standardised sample ($Q / \bar{Q}$) from site j in the region and let G_{ij}, i=1,2....N_j, be the F(.) transformation of z_{ij}. Under the condition that z is distributed as x (i.e. homogeneous region), the G_{ij} values are approximately uniform. If the hypothesis is not true, the difference can be amplified by the transformation

$$G_{ij}^{1} = 2|G_{ij} - 0.5| \qquad (3)$$

The statistic R is then defined for the M sites in the region as

$$R = \sum_{j=1}^{M} \frac{(G_{.j}^{1} - G_{..}^{1})^2}{U_j} \qquad (4)$$

where $G^{1}_{.j}$ and $G^{1}_{..}$ are site and regional average values of $G^{1}_{..}$ and U_j is the sampling variance of $G^{1}_{.j}$, defined by Wiltshire (1986b). The sampling distribution of R is approximately χ^2 – distribution, values of R exceeding χ^2 - crit leading to rejection of homogeneity hypothesis.

3.5 REGION CURVE DERIVATION

A reliable flood frequency analysis is possible for a station if it has a long record and distributions can be fitted to the data. But when only a short record is available at a site, the choice of distribution cannot be based on sample alone and prior knowledge about the distribution must be used. This prior knowledge is either in the form of distribution or in the form of region curve, which is the mean distribution of all recorded floods scaled by a single parameter in a region. When no record is available at a site, the region curve may be used together with an estimate of the mean annual flood obtained from catchment characteristics.

The different activities involved in arriving at the regional flood frequency curve are described below in the sequential order:

(i) Assembling the annual peak flow (Q) data for all stations in the selected region.

(ii) Ranking the annual peak flow values of each station in ascending order form i=1 to i=N (N is the number of years of data).

(iii) Estimating the mean of each station's data set as the arithmetic average of the annual maxima Q, the exception to this rule may be when the highest ranking flood exceeds three times the median, i.e $Q_{max}/Q_{med} > 3$. When this occurs, it indicates that the data contains an outlier and the arithmetic average is raised and the $Q / \bar{Q}$ are depressed. Under this condition, an estimator K times the median flood is used as $\bar{Q}$. The adjustment factor K is obtained empirically by arriving at a relation between median and arithmetic average by plotting a curve median versus arithmetic average of all stations in the region with $Q_{max}/Q_{med} \leq 3$. This method avoids biasing the estimate of $\bar{Q}$ when sample includes outliers.

(iv) Dividing Q of each station by $\bar{Q}$ to get the $Q / \bar{Q}$ values and tabulating the data. This is done to make the resulting regional flood frequency curve non-dimensional. By non-dimensionalising, the curve will have application both in gauged section (where $\bar{Q}$ is known from observed data) and in the ungauged section when used with a suitable equation relating $\bar{Q}$ with the geomorphological parameters.

(v) Using Gumbel's extreme value distribution, the corresponding plotting position Y (reduced variate of the return period) for the $Q / \bar{Q}$ values are formed. this is done using the Gringorten's approximation:

$$F(i,N) = \frac{(i-0.44)}{(N+0.12)} \qquad (5)$$

$$Y(i,N) = -\log_e(-\log_e F) \qquad (6)$$

(vi) Merging and assembling the entire Y and $Q / \bar{Q}$ values into class intervals; Y is divided into intervals of width 0.5, eg. -1.5 to -1.0, -1 to -0.5, -0.5 to 0.0, 0.0 to 0.5 etc. The Y values falling within each class interval and their corresponding $Q / \bar{Q}$ values are identified and assembled under respective class intervals.

(vii) Finding out the average Y and $Q / \bar{Q}$ values in each class interval. These sets of average Y and $Q / \bar{Q}$ values are obtained from the co-ordinates of the regional frequency curve. But with these points, the curve arrived at may not give $Q / \bar{Q}$ for higher return periods. These points form the lower part of the curve.

(viii) Extracting the highest three values of $Q / \bar{Q}$, regardless of station, and associating these with the plotting position for i=M-3, M-2, M-1 and M, where M is the total number of station years of data.

(ix) Placing the extracted highest values in class intervals (these give some more co-ordinates for the curve which will facilitate extension of curve for higher return periods).

(x) Plotting the cordinates obtained in steps (vii) and (ix) and drawing the eye-guided best fitting curve.

(xi) In order to arrive at the return period, the following relationship (IMD, 1972) is considered:

$$Y_T = -\log_e \log_e \left(\frac{T}{T-1} \right) \qquad (7)$$

where Y_T is the reduced variate for the return period T.

In the analytical method, distributions including Gumbel, log-normal, Pearson type III and log-Pearson type III have been attempted, because several agencies recommended the importance of these distributions in flood frequency studies.

3.6 DISTRIBUTIONS USED IN THE PRESENT STUDY

3.6.1 Gumbel Distribution

Gumbel distribution or EV1 is a two parameter distribution widely used in hydrology. The probability density function P(x) and cumulative density function F(x) are given below:

$$P(x) = \alpha\, e^{[-\alpha(x-\beta) - e^{-\alpha(x-\beta)}]} \tag{8}$$

$$-\infty < x < \infty, \qquad -\infty < \beta < \infty, \qquad \alpha > 0$$

$$F(x) = e^{-e^{-\alpha(x-\beta)}} \tag{9}$$

α is a concentration parameter, and β is a measure of central tendency

Estimation of Parameters

Method of moments

The two parameters α and β are computed using the following equations:

$$\alpha = 1.2825 / \sigma \tag{10}$$

$$\beta = \mu - 0.45\,\sigma \tag{11}$$

where μ and σ are the mean and standard deviation of the sample.

Estimation of quantiles

The following equations are used to obtain T-year flood events by the method of moments:

$$x_T = \mu + K_T\,\sigma \tag{12}$$

where

$$K_T = -[\,0.45 + 0.7797\, ln\,(-ln\,(\,1 - 1/T\,))] \tag{13}$$

and μ and σ are sample estimates of mean and standard deviation respectively.

Standard error

The standard error of the T-year event is obtained by equations by the method of moments.

$$S_T = \frac{\sigma}{\sqrt{n}}(1 + 1.1396\; K_T + 1.1000\, K_T^2\,)^{1/2} \tag{14}$$

Method of maximum likelihood

The following non-linear equation is solved using Newton's method of non-linear optimisation to compute the parameters α and β by the method of maximum likelihood.

$$\frac{n}{\alpha} - n(\mu - \beta) + e^{\alpha\beta}\sum_{i=1}^{n}(x_i - \beta)e^{-\alpha x_i} \tag{15}$$

$$e^{\alpha\beta} = n / \sum_{i=1}^{n} e^{-\alpha xi} \tag{16}$$

$$\beta = \frac{1}{\alpha}\ln\left[n / \sum_{i=1}^{n} e^{-\alpha xi} \right] \tag{17}$$

Estimation of quantiles

T-year flood events are computed using the maximum likelihood estimates of parameters, α and β in the following expressions

$$\mathrm{x_T} = \beta + \frac{1}{\alpha} y_T \tag{18}$$

$$\mathrm{y_T} = -\ln\left(-\ln\left(\frac{(T-1)}{T} \right)\right) \tag{19}$$

Standard Error

The standard error of estimates for T-year event is computed using the following equation

$$\mathrm{S_T} = \frac{1}{\sqrt{n\alpha}}(1.1086 + 0.514\ y_T + 0.6079\ y_T^2\ )^{1/2} \tag{20}$$

3.6.2 Log-normal Distribution

Log-normal distribution can be applied to a wide variety of hydrologic events especially in cases in where the corresponding variable has a lower bound, the empirical frequency distribution is not symmetrical and the factors causing those events are independent and multiplicative. Chow (1964) has provided theoretical justification for the use of the log normal distribution in flood frequency studies. The causative factors for many hydrologic variables act multiplicatively rather than additively and so the logarithms of the peak flows which are the product of these causative factors follow the log-normal distribution.

If the logarithms of a variable x are normally distributed, then the probability density function p(x) is given by the following equation

$$p(x)=\frac{1}{x\sigma_y\sqrt{2\pi}}e^{\frac{-(\ln x-\mu_y)^2}{2\sigma_y^2}} \tag{21}$$

$$0<y<\infty$$

where μ_y is the mean of the natural logarithms of the variable x, and σ_y is the standard deviation of the natural logarithm of the variable x

Estimation of Parameters

Method of moments

The general equations for first of the probability density about the origin and second moment of the probability density function about the centroid are given as

$$\mu=e^{\mu_y}+\frac{\sigma_y^2}{2} \tag{22}$$

$$\sigma^2=\left(e^{\sigma_y^2}-1\right)\mu^2 \tag{23}$$

where μ and σ are the mean and standard deviation of the variable x

If a sample of annual maximum flood peaks is given, then the mean and standard deviation estimated from that sample provide the estimates of μ and σ. By solving equations (22) and (23), we get values of μ_y and σ_y.

Estimation of quantiles

T-year flood can be computed by method of moments using the equation

$$\ln x_T=\mu_y+t\ \sigma y \tag{24}$$

where x_T is an estimate of T-year flood, and t the standard normal deviate.

An alternate way for calculating the T-year floods involves the use of frequency factor, K, in the frequency equation. The general frequency equation in terms of frequency factor, K, is given by the equation:

$$x_T = \mu + \sigma K_T \tag{25}$$

where μ and σ are mean and standard deviation of the original flood peak series.

The frequency factor K_T for Lognormal two parameter distribution is given by the following equation (Kite, 1977):

$$K_T = \left[\frac{e^{\left[\ln(1+z^2)\right]^{1/2} t - \left[\ln(1+z^2)\right]/2} - 1}{z} \right] \tag{26}$$

where, z is the coefficient of variation of recorded events.

Standard error

The standard error of estimates for T-year flood events are computed by method of moments using the following equation:

$$\Delta_T = [1 + (z^3 + 3z)\, K_T + (z^8 + 6z^6 + 15z^4 + 16z^2 + 2)\, K_T^2 / 4]^{1/2} \tag{27}$$

$$S_T = \frac{\Delta_T \sigma}{\sqrt{n}} \tag{28}$$

where K_T is frequency factor given by equation (26), z the coefficient of variation of the recorded events, σ the standard deviation of the recorded events, n the number of recorded events, and S_T is the standard error of estimate for T-year flood events.

Method of maximum likelihood

By this method, the parameters of the distribution, μ_y and σ_y, are estimated by solving the following equations resulting from differentiating the logarithms of the likelihood function with respect to μ_y and σ_y^2 respectively.

$$\mu_y = \sum_{i=1}^{n} \ln x_i / n \tag{29}$$

$$\sigma_y^2 = \sum_{i=1}^{n} \frac{(\ln x_i - \mu_y)^2}{n} \tag{30}$$

Quantile Estimation

The following relationship is used to estimate T-years flood events by the method of maximum likelihood procedure

$$x_T = e^{\mu_y + t\sigma_y} \tag{30a}$$

where t is the standard normal deviate.

Standard error

The following equations are used to compute Standard error of estimates for T-year flood events by maximum likelihood

$$\delta_T = [(\ln(z^2+1)(1+K_T z)^2(1+t^2/2))/z^2]^{1/2} \tag{31}$$

$$S_T = \delta_T \sigma / \sqrt{n} \tag{32}$$

3.6.3 Pearson type III Distribution

The Pearson type III distribution is a three parameter distribution. This is also known as Gamma distribution with three parameters. The probability density functions of the distribution is given by:

$$P(x) = \frac{1}{\alpha \Gamma \beta} \left(\frac{x-\gamma}{\alpha} \right)^{\beta - 1} e^{-\left(\frac{x-\gamma}{\alpha} \right)} \tag{33}$$

where α, β and γ are the scale, shape and location parameters respectively. From the sample mean , standard deviation and coefficient of skewness , the parameters α, β and γ can be determined. The parameter β is always positive as it is equal to the product of the square of sample standard deviation σ and α. When $\alpha > 0$, the distribution is always positively skewed and $\gamma \leq x \leq \infty$. When $\alpha < 0$, the distribution is negatively skewed and $-\infty \leq x \leq \gamma$.

Estimation Parameters

The parameters of the Pearson type III distribution α, β and γ are obtained using the following equations by the method of moments

$$\beta = \left(\frac{2}{\gamma_1}\right)^2 \qquad (34)$$

$$a = \frac{\sigma}{\sqrt{\beta}} \qquad (35)$$

$$\gamma = \mu - \sigma\sqrt{\beta} \qquad (36)$$

where γ_1 is the unbiased estimate of skewness computed as

$$\gamma_1 = \hat{\gamma}_1\sqrt{\frac{n(n-1)}{(n-2)}}\left(1 + \frac{8.5}{n}\right) \qquad (37)$$

$\hat{\gamma}_1$ = biased estimate of skewness computed from the sample.
n = size of the sample.

Estimation of quantiles

T-year flood events are obtained by the following equations by the method of moments

$$x_T = \mu + \sigma K_T \qquad (38)$$

where μ and σ are the mean and standard deviation estimated from the sample respectively.

$$K_T = t + (t^2 - 1)\frac{\gamma_1}{6} + \frac{1}{3}(t^3 - 6t)\left(\frac{\gamma_1}{6}\right)^2 - (t^2 - 1)\left(\frac{\gamma_1}{6}\right)^3 + t\left(\frac{\gamma_1}{6}\right)^4 + \frac{1}{3}\left(\frac{\gamma_1}{6}\right)^5 \qquad (39)$$

where t is the standard normal reduced variate corresponding to T-year return period and γ_1 is the co-efficient of skewness.

Standard error

The standard error of T-year flood event, S_T is computed by method of moments using the following equations:

$$S_T^2 = \frac{\sigma^2}{n}\left[1 + K\gamma_1 + \frac{K^2}{2}\left(\frac{3\gamma_1^2}{4} + 1\right) + 3K\frac{\partial K}{\partial \gamma_1}\left(\gamma_1 + \frac{\gamma_1^3}{4}\right)\right.$$
$$\left. 3\left(\frac{\partial K}{\partial \gamma_1}\right)^2\left(2 + 3\gamma_1^2 + \frac{5\gamma_1^4}{8}\right)\right] \quad (40)$$

$$\frac{\partial K}{\partial \gamma_1} = \frac{(t^2-1)}{6} + \frac{4(t^3-6t)\gamma_1}{6^3} - \frac{3(t^2-1)\gamma_1^2}{6^3} + \frac{4t\gamma_1^3}{6^4} - \frac{10\gamma_1^4}{6^6} \quad (41)$$

where t is the standard normal reduced variate, and γ_1 the coefficient of skewness.

Method of maximum likelihood

The parameters α, β and γ of the distribution are estimated by the method of maximum likelihood using an iterative procedure of non-linear optimisation, based on Newton's method, to solve the following non-linear equation for γ

$$\frac{-n\Gamma^1(\beta)}{\Gamma(\beta)} + \sum_{i=1}^{n} \ln\ (x_i - \gamma) - n\ \ln \alpha = 0 \quad (42)$$

where $$\Psi(\beta) = \frac{\Gamma^1(\beta)}{\Gamma(\beta)} \quad (43)$$

$$= \ln(\beta+2) - \frac{1}{2(\beta+2)} - \frac{1}{12(\beta+2)^2} + \frac{1}{120(\beta+2)^4}$$
$$- \frac{1}{252(\beta+2)^6} - \frac{1}{\beta+1} - \frac{1}{\beta} \quad (44)$$

$$\beta = \frac{1}{\left[1 - \left(n^2 \Big/ \sum_{i=1}^{n}(x_i - \gamma)\sum\left(1 \big/ (x_i - \gamma)\right)\right)\right]} \quad (45)$$

$$\alpha = \sum_{i=1}^{n}\frac{(x_i - \gamma)}{n} - \frac{n}{\sum_{i=1}^{n}\left(\frac{1}{(x_i - \gamma)}\right)} \quad (46)$$

The optimum value of the parameter, γ, is used in equations (45) and (46) for estimating two parameters, α and β.

Estimation of quantiles

The parameters α, β and γ estimated by the method of maximum likelihood are used in the following equation to compute T-year flood events:

$$\mathrm{x_T} = \alpha\beta\left(1 - \frac{1}{9\beta} + t\sqrt{\frac{1}{9\beta}}\right)^3 + \gamma \qquad (47)$$

where t is the standard normal reduced variate corresponding to T-year return period.

Standard error

The variance of the T-year event , S_T^2 is computed using the following equations by the method of maximum likelihood:

$$S_T^2 = \left(\frac{\partial x_T}{\partial \alpha}\right)^2 \operatorname{var}\alpha + \left(\frac{\partial x_T}{\partial \beta}\right)^2 \operatorname{var}\beta + \left(\frac{\partial x_T}{\partial \gamma}\right)^2 \operatorname{var}\gamma + 2\frac{\partial x_T}{\partial \alpha}\frac{\partial x_T}{\partial \beta}\operatorname{cov}(\alpha,\beta)$$

$$+ 2\frac{\partial x_T}{\partial \alpha}\frac{\partial x_T}{\partial \gamma}\operatorname{cov}(\alpha,\gamma) + 2\frac{\partial x_T}{\partial \beta}\frac{\partial x_T}{\partial \gamma}\operatorname{cov}(\beta,\gamma) \qquad (48)$$

$$\frac{\partial x_T}{\partial \alpha} = \left(\beta^{1/3} - \frac{1}{9\beta^{2/3}} + \frac{t}{3\beta^{1/6}}\right)^3 \qquad (49)$$

$$\frac{\partial x_T}{\partial \alpha} = 3\alpha\left(\beta^{1/3} - \frac{1}{9\beta^{2/3}} + \frac{t}{3\beta^{1/6}}\right)^2 + 18\left(\frac{1}{3\beta^{2/3}} + \frac{2}{27\beta^{5/3}} - \frac{t}{18\beta^{7/6}}\right) \qquad (50)$$

$$\frac{\partial \mathrm{x_T}}{\partial \gamma} = 1 \qquad (51)$$

$$\operatorname{var}\alpha = \frac{1}{\mathrm{n}\alpha^2 D}\left(\frac{\Psi(\beta)}{(\beta-2)} - \frac{1}{(\beta-1)^2}\right) \qquad (52)$$

$$\operatorname{var}\beta = \frac{2}{\mathrm{n}D\alpha^4(\beta-2)} \qquad (53)$$

$$\operatorname{var}\gamma = \frac{\beta\Psi^1(\beta) - 1.0}{\mathrm{n}\alpha^2 D} \qquad (54)$$

$$\operatorname{cov}(\alpha,\beta) = \frac{-1}{\mathrm{n}\alpha^3 D}\left(\frac{1}{(\beta-2)} - \frac{1}{(\beta-1)}\right) \qquad (55)$$

$$\operatorname{cov}(\alpha,\gamma) = \frac{1}{\mathrm{n}\alpha^2 D}\left(\frac{1}{(\beta-1)} - \Psi^1(\beta)\right) \qquad (56)$$

$$\operatorname{cov}(\beta,\gamma)=\frac{-1}{\mathrm{n}\alpha^3 D}\left(\frac{\beta}{(\beta-1)}-1\right) \tag{57}$$

$$\mathrm{D}=\frac{1}{(\beta-2)\alpha^4}\left[2\Psi^1(\beta)-\frac{(2\beta-3)}{(\beta-1)^2}\right] \tag{58}$$

$$\begin{aligned}\Psi^1(\beta)=&\frac{1}{(\beta+2)}+\frac{1}{2(\beta+2)^2}+\frac{1}{6(\beta+2)^3}-\frac{1}{30(\beta+2)^5}\\&+\frac{1}{42(\beta+2)^7}-\frac{1}{30(\beta+2)^9}+\frac{1}{(\beta+1)^2}+\frac{1}{\beta^2}\end{aligned} \tag{59}$$

where D is the determinant of matrix of likelihood derivatives

3.6.4 Log-Pearson type III Distribution

The log-Pearson type III distribution has been widely used in hydrology for fitting the frequency distribution of flood data. The USWRC recommends the use of the log-Pearson type III distribution as an attempt to promote a uniform and a consistent approach for flood frequency studies. As a result, the use of this distribution has become popular in the United States, and several practising engineers from Federal, State and Local governments have started using this distribution.

If the natural logarithms, lnx, of a variable x are distributed as Pearson type III variate, then the variable will be distributed as a log-Pearson type III with probability density function:

$$P(x)=\frac{1}{\alpha\Gamma\beta}\left(\frac{\ln x-\gamma}{\alpha}\right)^{\beta-1}e^{-\left(\frac{\ln x-\gamma}{\alpha}\right)} \tag{60}$$

where α,β and γ are the scale, shape and location parameters respectively.

Estimation of parameters

Method of Moments

The parameters α, β and γ are estimated by the direct application of the moment generating function to the probability density function of the log-Pearson type III distribution by this method. The following equations are used to compute the parameters of the distribution by this method:

$$B = \frac{\ln \mu_3' - 3 \ln \mu_1'}{\ln \mu_2' - 2 \ln \mu_1'} \tag{61}$$

where $\mu_3^1, \mu_2^1, \mu_1^1$ are the third, second and first moments of the pdf about the origin respectively and these moments can be estimated from the sample. Therefore, B can be estimated directly from the sample:

$$C = 1 / (B - 3) \tag{62}$$

If $3.5 \leq B \leq 6.0$, then

$$A = -0.23019 + 1.65262\, C + 0.20911\, C^2 - 0.04557\, C^3 \tag{63}$$

If $3.0 < B \leq 3.5$, then

$$A = -0.47157 + 1.99955\, C \tag{64}$$

The value C obtained from equation (62) is substituted either in regression equation (63) or (64) depending upon the value of B. If the value of B does not lie within the above stated ranges, then this method is inapplicable for that particular sample data. If the value of B lies within one of the above stated ranges, then the parameters α, β and γ are estimated by substituting corresponding A, B and C values in the following equations:

$$\alpha = 1 / (A + 3) \tag{65}$$

$$\beta = (\ln\mu_2^1 - 2\ln\mu_1^1) / (\ln(1-\alpha)^2 - \ln(1-2\alpha)) \tag{66}$$

$$\gamma = \ln\mu_1^1 + \beta\ln(1-\alpha) \tag{67}$$

the parameters α, β and γ are then used to compute mean μ_y, standard deviation σ_y and coefficient of skew, γ_y of the logarithms of x as

$$\mu_y = \gamma + \alpha\beta \tag{68}$$

$$\sigma_y = \alpha\sqrt{\beta} \tag{69}$$

$$\gamma_y = 2.0 / \sqrt{\beta} \tag{70}$$

Estimation of quantiles

T-year events are computed by the method of moments using the following equation:

$$x_T = e^{\mu_y + K_T \sigma_Y} \tag{71}$$

where μ_y and σ_y are the mean and standard deviation of the logarithms of x respectively and computed using the equation (68), (69) and (70) and K_T is computed using the equation (39).

Standard error

The standard error of the T-year event for the log-Pearson type III distribution by this method is computed using equation (40) to obtain $S_{T,Y}$ in log units from the standard normal deviate and the coefficient of skew, computed from equation (41). The standard error is then converted back to linear units as:

$$S_{T,X} = \frac{x_T(e^{S_{T,Y}} - e^{-S_{T,Y}})}{2.0} \tag{72}$$

where $S_{T,X}$ is the average of the positive and negative standard errors in linear units and x_T is the T-year event.

Method of maximum likelihood

The parameters α, β and γ are estimated by this method by solving the following non-linear equations iteratively using Newton's method of optimisation.

$$\sum_{i=1}^{n}(\ln x_i - \alpha) = n\alpha\beta \tag{73}$$

$$n\Psi(\beta) = \sum_{i=1}^{n}\ln[(\ln x - \gamma)/\alpha] \tag{74}$$

$$n = \alpha(\beta - 1)\sum_{i=1}^{n}\frac{1}{\ln x_i - \gamma} \tag{75}$$

Estimation of quantiles

By this method T-year flood events are computed by using the following equations:

$$x_T = \exp\,(\alpha E^3 + \gamma) \tag{76}$$

$$E = \beta\left(1 - \frac{1}{9\beta} + t\sqrt{\frac{1}{9\beta}}\right) \tag{77}$$

where α, β and γ are the maximum likelihood estimates of the parameters and t is the standard normal reduced variate.

Standard error

The standard error in log units is obtained from equations (48) to (59) using the maximum likelihood estimates of parameters α, β and γ. The logarithmic standard error is then converted to linear units using equation (72).

3.7 RELATION BETWEEN MEAN ANNUAL PEAK FLOW AND BASIN CHARACTERISTICS

3.7.1 General

The prediction of flood frequency or hydrograph parameters for ungauged basins relies on statistical relationship between river flow and certain physical characteristics of sample basins. These characteristics may be described as geometric properties or morphometric properties, cover properties (soil and land use) and climatic properties. Since the climatic properties in the regions considered is more or less homogeneous and since sufficient data on cover properties are not available (also, this may not have much impact), they have not been considered.

The morphometric properties have been correlated with mean annual peak flow to arrive at relationships for the purposes of flood prediction.

3.7.2 Prediction of Mean Annual Peak Flow from Basin Characteristics

The geomorphology describes the environment in which the hydrological process operates. A strong mutual correlation exists between geomorphological variables and hydrological characteristics. Such relationship can be applied to surface water regime. Thus, the linking of geomorphological parameters with hydrological characteristics of the basin provides a simple way to understand the hydrological behaviour of different basins and particularly of the ungauged basins.

The geomorphological properties which are important to the hydrological simulation studies include the linear, areal and relief aspects of the basin. Table 5 gives an exhaustive list of geomorphological properties generally considered in hydrologic studies.

TABLE 5: MORPHOMETRIC VARIABLES GENERALLY CONSIDERED IN HYDROLOGIC STUDIES

Properties measured or counted solely from channel network and basin outline horizontal plane

Property (1)	where used or defined (2)
Stream order (Basin order)	Horton, 1945, p.282-282 Strahler, 1952b,p.1120 Strahler, 1954a,p.344. Melton, 1957, Appendix 1
Number of streams Or basins of order u	same as above
*Stream magnitude	Shreve,1967,p.179
Bifurcation ratio	Horton,1945, p.286 Strahler.1952b,p.1137 Schumm, 1954a

(contd....)

Table 5 (contd.)

(1)	(2)
Axial angle (Entrance angle)	Horton,1945.p.349 Strahler,1954,p.345-347 Schumm. 1954a Melton,1957, Appendix 1
Stream azimuth	Strahler. 1954a,p.346
*Stream length (Channel length)	Horton,1955,p.283
Stream length Mean length of segments of order u	Horton,1945,p.286 Strahler,1952b,pp 1134 Strahler,1954a,p.345 Strahler,1957 Miller,1953
Stream length, Total length of order u	Horton,1945,p.291 Strahler,1957
Stream length ratio	Horton,1945, p.286-287
Length of overland flow, or	Horton,1945,p.284
Ratio of stream length ratio to bifurcation ratio	Horton,1945,p.292
Basin perimeter	Smith, 1950, p.657
*Basin length	Schumm, 1954a
B. Properties requiring areal measurement (planimetry)	
*Area of basin or order u	Horton, 1945, p.283
Basin area ratio	Schumm,1954a
Inter-basin area	Schumm, 1954a
*Drainage density	Horton, 1945, p.283
Constant of channel	Schumm, 1954a Strahler, 1957
*Stream Frequency	Horton,1945,p.285

(contd....)

Table 5 (contd.)

(1)	(2)
Drainage intensity	Faniran, 1969
Texture ratio	Smoth,1950,p.657
*Basin circularity	Miller, 1953,p.8
*Basin elongation ratio	Schumm,1954a
*Lemniscate 'k'	Chorley, et al.,1957
A. Properties involving elevation differences (relief aspect)	
*Stream channel slope	Horton, 1945,p.295 Strahler, 1952b,p.1135 Schumm,1954a
*Other stream channel slope, segment of order u	Horton,1945,p.295
Stream channel slope ratio	Horton, 1945,p.295
Ground slope	Horton,1945,p.304
Orthogonal to contour	Strahler,1950
Ground slope,	Strahler,1950
(valley-side slope)	Schum,1954a
Maximum angle of,	Melton,1957
*Overland slope	Nash,1960
Ratio of channel slope to Ground slope	Horton,1954,p.349
Dihedral angle between valley sides	Melton,1957
Relief maximum in basin	Strahler, 1952b,p.1119 Schumm,1954a Melton, 1957
Relief ratio	Schumm, 1954a
Relative relief	Melton, 1957
Ruggedness Number	Strahler, 1958,p.289
Geometry Number	Strahler, 1958, p.295

(contd....)

Table 5 (contd.)

(1)	(2)
Relative basin height (in hypsometry)	Strahler, 1952b, p.1119-1121
Relative basin area (in Hypsometry)	Strahler, 1952b, p.1119-1121
*Hypsometric integral	Strahler,1952b,p.1119-1121
Volume of land mass	Strahler, 1952b,p.1120-1121
Curvature of slope profile	

*Those suitable for consideration as feasible flood predictors are indicated by asterisk

**Source:Malcom Newson, Map work for flood studies, Part 1- Selection and derivation of indices, Institute of Hydrology, Wallingford, Report No.25, 1975.

A description of the important quantitative geomorphologic parameters, which are relevant to drainage basin composition, is given in the following paragraphs.

Linear Aspects of the Channels

Various parameters which involve length of channels in different ways are grouped under this category. Linear aspects of the channel network are listed below:

(i) Length of the main channel
(ii) Length of the channel between the outlet and a point nearer to centre of gravity
(iii) Total length of channels
(iv) Stream order
(v) Wandering ratio
(vi) Basin perimeter
(vii) Fineness Ratio

Stream length

This is the length along the longest watercourse from the outflow point of designated sub-basin to the upper limit to the catchment boundary. The stream length is measured using Chartometer from the GTS map.

Length of channel between the outlet and a point nearer to Centre of gravity

It is the length of the channel measured from the outlet of the catchment to a point on the stream nearest to the centroid of the basin. After finding the centroid, the length is measured using the Chartometer.

Total length of the channels

Total channel length is the sum total of the lengths of channels of all the orders in the basin. This parameter is important as it gives an idea of overland flow and channel flow in the basin. Channel storage also varies with stream length as a simple power function.

Stream order

For all practical purposes, the quantitative study of channel networks used to begin with Horton's (1945) methods of ordering of channels. Later on, Strahler (1952) proposed a modification of Horton's ordering scheme. Strahler's method is now generally preferred due to its simplicity and greater freedom from subjective decisions. There are three steps in Strahlers's ordering procedure.

1) Channels that originate at a source are defined to be first order streams;
2) When two stream of order w joins, a stream of order (w=1) is created; and
3) When two streams of different order join, the channel segment immediately downstream has the higher of the combining streams.

Wandering ratio

Wandering ratio is defined as the ratio of the main stream length to the valley length. Valley length is the straight line distance between outlet of the basin and the farthest point on the ridge.

Basin perimeter

It is measured along the divides between basins and may be used as an indicator of basin size and shape.

Fineness ratio

The ratio of channel length to the length of the basin perimeter is fineness ratio, which is a measure of topographic fineness.

Areal Aspects of the Basin

The parameters which are governed mainly by the area of the drainage basin are classified as Areal aspects of the basin. Following areal aspects are considered for the present study.

Catchment Area

Basin area is a fundamentally important characteristic for the simple reason that large catchments have in general higher values of flow statistics than smaller ones. Drainage area is defined as the collecting area from which water would go to a stream or river. The boundary of the area is determined by a ridge separating water flowing in opposite directions. This parameter is hydrologically important because it directly affects the flood hydrograph and the magnitude of flood peaks in mountainous areas. The larger the size of the basin, the greater is the amount of the rain intercepted and higher the peak discharge that results. The catchment areas were available from the Water Resources Circle of Kerala PWD. However, areas of some catchments were checked from GTS maps, using Planimeter.

Drainage Density

Drainage density is defined as the ratio of the total length of channels of all orders in a basin to the area of the basin. It should, therefore, be measured on the GTS maps of large scales(In the present case the largest scale GTS map available for the region was 1:50,000) so that the first order streams can also be taken into account. Drainage density is a textual measure of a basin, which is generally independent of basin size. It is considered to be a function of climate, lithology, and stage of development. Numerically, this ratio expresses the number of kilometres of channel maintained by a square kilometre of drainage area.

Stream frequency

Stream frequency is defined as the number of streams per unit area. Melton(1956) analysed in detail the relationship between drainage density and stream frequency and gave the relation

$$F = 0.694\, D^{2} \tag{78}$$

Stream frequency F is computed as the number of streams per unit area or

$$F = N / A \tag{79}$$

where, N is the total number of segments of all orders in the catchment area and A the drainage area of the basin.

Circularity ratio

Basin circularity ratio is defined as the ratio of the basin area to the area of a circle having a circumference equal to the perimeter of the basin .The value of this ratio approaches unity as the shape of a drainage basin approaches a circle.

Elongation Ratio

Elongation ratio of a basin is defined as the ratio between the diameter of a circle with the same area as the basin and basin length. The value of the elongation ratio approaches unity as the shape of drainage basin approaches a circle.

Watershed shape factor

Watershed shape factor is defined as the ratio of main stream length to the diameter of a circle having the same area as the watershed.

Unity shape factor

Unity shape factor is defined as the ratio of the basin length to the square root of the basin area.

Relief aspects of catchments

Relief aspects are the functions of the elevation or elevation difference at various points in a catchment along the channels. Contour lines on a toposheet are made use of, while determining the relief aspects. Various parameters involving the relief aspects are furnished below.

Basin relief

Basin relief of a basin is the maximum vertical distance from the stream outlet to the highest point on the dividing ridge. The total relief of a basin is a measure of the potential energy available to move water and sediment down slope.

Relief ratio

The relief ratio is defined as the ratio between the basin relief and the basin length. In normally shaped basins, the relief ratio is a dimensionless height length ratio equal to the tangent of the angle formed by intersection at the basin mouth of a horizontal plane with a plane passing through the highest point on the divide.

Relative relief

Relative relief is defined as the ratio of the basin relief expressed in units of miles to the basin perimeter. Relative relief is an indicator of the general steepness of a basin from summit to the outlet. Relative relief is an indicator of the general steepness of a basin from summit to the outlet.

Stream slope

The slope of catchment has to be indexed by the slope of the mainstream, since overland slope is difficult to measure by techniques such as grid sampling. Benson (1959) has shown that only mainstream slope need be measured because it is closely correlated with tributary slopes, and Strahler (1950) has demonstrated the correlation of channel slope and valley side slopes. The slope measure used is that defined by U S Geological Survey and is the slope between the 10 and 85 percentiles of the mainstream length, measured upstream from the site of interest. This definition excludes the highest and the lowest gradients at either end of the stream. All other things being equal, one would expect steep basins to have higher floods and less lowflows than flat basins.

Geomorphologic Parameters Considered

Using a step-wise regression analysis, the geomorphic parameters, having greater influence on mean annual peak flows have been identified. The parameters initially considered are : drainage area (A), stream slope (SL), main stream length(L), and stream frequency (SF). These parameters were selected keeping in view of the classical works of Horton (1945), Strahler (1952, 1954, 1968) and Schumm (1954, 1977). A matrix of such parameters were considered in the Flood Studies Report (NERC, 1975). Several attempts have been made in different geographical areas of the world to study the relation between mean annual peak flow and geomorphologic parameters. Of all the parameters, the basin area, drainage density, main stream length and stream slope have been most frequently utilised (NIH, 1991-92). Based on the step-wise regression analysis, the parameters to be considered in each region was selected. The parameters are given in Table 6.

This procedure of step-wise regression consists of considering the regression of one variable at a time by adding at each step the variable that explains the largest amount of the remaining unexplained variation. After each step, all the variables in the equation are examined for significance, and discarded if they no longer explain a significant amount of variation. Then the first variable is added in the one with the highest simple correlation with the dependent variable. The second variable added is the one explaining the largest variation in the dependent variable that remains unexplained by the first variable added. At this point, the first variable is tested for significance and retained or discarded depending on the results of the first. The third variable added is the one that explains the largest portion of the variation that is not explained by the variables already in the equation. The variables in the equation are then tested for significance. This procedure is continued until all of the variables in the equation are found to be significant.

TABLE 6: SELECTED CATCHMENT CHARACTERISTICS

Station no (1)	Name of stations (2)	Area km^2 (A) (3)	Stream Length km (L) (4)	Stream Frequency (SF) (5)	Stream Slope (SL) (6)
1	MANJESWAR	25.44	12.5	0.85	0.4
2	ANNAKAL	166.25	15	0.83	9.8
3	UPPALA	256.5	34.5	0.89	1.2
4	SHIRIYA(U/S)	348	35	1.47	1.52
5	MADHUR	15.54	5.8	1.25	5.26
6	PADIATHADUKA	754.17	70.83	4.25	12.56
7	PALLANGODE	527.36	53.86	1.21	5.6
8	MOONAMKADAVU	216.8	30	0.89	22.22
9	KAKKADAVU	480.4	34.85	0.38	11.48
10	MANGARA	109.6	34.85	1.61	15.3
11	IRUDE	337	34.21	0.89	15.59
12	KUILOOR	1040	55.76	0.65	16.74
13	PALA	270	30.41	0.89	13.15
14	MERUVAMBAI	180	39.28	0.62	6.79
15	KANNAVAM	59	14.57	0.8	27.45
16	PATTIAM	46	15.21	2.26	26.29
17	VALAYAM	129	26.61	0.86	12.53
18	BAVELI	174	28	2.51	12.38
19	THIRUNELLY	37.58	10	3.83	44
20	THONDAR	70.13	15	1.81	10.67
21	MANAMTODY	280	34.5	2.56	1.55
22	CHOORANI	40	13	4.8	28.72
23	PANAMARAM	448	48.5	1.97	0.55
24	KAKKAVAYAL	78.08	19.75	2.08	4.05
25	MUTHANGE	172.8	24.8	1.96	8.44
26	VAZHAVATTA	56.58	9.5	1.56	11.22
27	MANJAT	48	16.52	6.9	24.68

(contd...)

Table 6 (contd.)

(1)	(2)	(3)	(4)	(5)	(6)
30	KADAMTHARAPUZA	79	18	2.15	12.45
31	KOLLICKAL	34.82	11	1.18	46.06
32	KOODATHAI	117	20	2.59	13.33
33	MUKKOM	206	31	3.07	44.73
34	AREACODE	1580	85.5	3.03	5.93
35	KANJIRAPUZHA	64	17	4.09	69
36	CHALIYAR	344	43	3.95	26.05
37	PUNNAPUZHA	377	52.5	2.89	23.36
38	MARUTHAPUZHA	65	16	3.92	16.67
39	KARIMPUZHA	656	55	2.92	21.33
40	KUTHIRAPUZHA	220	35	1.24	3.81
41	ANAKAYAM	439	52.5	1.32	2.54
42	THIRUVAGAPUZHA	914.2	83.5	1.34	14.37
43	THRITHALA	2601	186.5	1.84	2
44	CHERUTHURUTHY	2237.75	161.5	2.09	5.12
45	PAMPADY	920	138	4.09	6.96
46	CHEERAKUZHI	982.29	74	0.82	3.6
47	MANAKKADAVU	508.33	67.5	4.32	25.87
48	KUTTIPURAM	3755.5	201.5	1.64	1.85
49	SILENT VALLEY	73.08	15.18	5.64	0.075
50	PAZHOOR	120.35	29.5	1.15	1.81
51	KUNDUKADAVU	28.5	12	3.33	5.56
52	MANALI	244.34	40.5	1.69	2.63
53	PILANTHODU	67.5	15	2.19	1.78
54	CHIMONI	73.5	13	2.45	32.82
55	MUPPLY	62	23.5	4.39	4.54
56	KURUMALI	410	54	2.19	1.03
57	KARUVANNUR	753.4	66	1.74	1.03
58	AMBALAKADAVU	1160	111	2.9	12.25
59	KARAPPARA	43.5	22.5	3.56	21.33

(contd....)

Table 6 (contd.)

(1)	(2)	(3)	(4)	(5)	(6)
60	KURIARKUTTY	126.8	24	2.13	5.56
28	CHITTOOR	96.96	24.5	3.28	23.94
29	CHAVADIYOOR	440	55	4.01	43.15
61	KALADY	4243.98	206.5	3.15	6.07
62	PILANTHODU	3824.68	177.75	3.26	7.05
63	CHEMMANNAR	58.64	15.5	3.39	24.08
64	PERINJANKUTTY	401.81	50.25	3.12	13.93
65	KALLAR EASTERN	107.05	27.25	5.04	12.72
66	THUVALLUR	38.3	12	3.03	21.11
67	PANNIAR	120.75	31.5	5.6	25.82
68	CHAMAPKKAD	251.12	33.5	4.78	58.1
69	MUVATTUPUZHA	1044.35	58.93	0.78	7.69
70	KAKADASSERRY	196.53	38.65	0.66	3.45
71	KALAMPUR	385.52	51.32	0.76	17.67
72	THODUPUZHA	393.5	55.76	0.94	10.52
73	MALANKARA	292.72	34.85	0.98	29.07
74	PEROOR	768	53	2.6	24.6
75	PALAI	429.79	41.5	2.5	28.92
76	CHERIPPAD	147	18.5	3.8	19.56
77	TEEKOY	57	12.5	3.95	12.5
78	THONDRA	816.43	91	2.46	0.88
79	MANIMALA	535.9	52	2.94	2.56
80	MUNDAKAYAM	107.7	21	3.97	30.48
81	ERAPUZHA	1695.72	122	3.83	10.71
82	KURUDAMANNIL	1527.44	101.5	4.16	1.05
83	KOLLAKADAVU	952.71	103.28	1.15	2.07
84	PANDALAM	833.62	91.87	1.25	2.32
85	KONNY	419	52.75	0.63	2.45
86	ANAYADI	82.34	19	0.47	2.36
87	ENATHU	1196.06	93.68	3.22	1.14

(contd....)

Table 6 (contd.)

(1)	(2)	(3)	(4)	(5)	(6)
88	PUNALUR	870	68.34	4.27	0.78
89	THENMALA	561.79	19	0.68	22.46
90	AYOOR	184	26.61	0.69	4.01
91	VALAYANKIL	49.19	37.38	4.84	29.96
92	AYIROOR	66	22	0.53	15.6
93	MAMEM	106	17.5	0.58	2.4
94	MYLAMMODE	84	25	0.95	6.98
95	ARAYANAD	155.33	19.01	0.32	28.05
96	MARUTHANKUZHY	79	15	2.15	8.23
97	AMARAVILA	333.97	49.4	0.34	18.42
98	OTTASEKARA MANGALAM	176.84	34.21	0.64	11.07

3.8 RAINFALL-RUNOFF MODELLING

3.8.1 General

Rainfall-runoff models are commonly used for the purpose of flood forecasting schemes. They are also used, less spectacularly, in the efficient operation of storage reservoirs for hydro-electric, irrigation or other purposes. In most countries, rainfall records are generally available from several stations for long periods with somewhat good reliability. But the streamflow measurements, which are most useful for the assessment of water resources or of damaging flood peaks, are often limited and are rarely available for a specific river under investigation. Hence, evaluating river discharges from rainfall has stimulated the imagination and ingenuity of research workers for many years.

In the context of rainfall-runoff process, it is obvious that the effect of rainfall on a catchment is greatly dependant upon the state of the catchment at the time of its occurrence. The rainfall may be entirely rejected by the soil, or almost all of it may be absorbed, depending upon the soil moisture state and the current infiltration capacity. The fact that rainfall excess is, to some extent at least, a surplus after making up the soil moisture deficiency at the time of occurrence of the rainfall, introduces a complexity into the system which could not possibly be described by a linear system, with the constraints of proportionality and time invariance.

Conscious of the necessity for algebraic complexity in describing the rainfall-discharge system, even if it is physically simple, hydrologists have not generally sought to represent it by the inclusion of quadratic or higher order terms as done by Amorocho (1963), but have resorted to the conceptual modelling approach to represent the generation of runoff volumes, and retained the linear model to represent the subsequent diffusion or routing of the flood wave (Kachroo, 1988). Alternatively, they have resorted to the development of models which apply the linear assumption to the relationship between the modified inflow and outflow (Natale et al, 1976, Nash et al, 1983 and Liang, 1986).

3.8.2 Model Requirements

Ideally, it is expected that the model represent as closely as possible the actual physical process occurring in the catchment. Then it is essential that it should represent accurately the transformation of the input into the output. The utility of the model is reflected in the extent to which it satisfies this practical objective, which is termed "model accuracy". The second requirement is consistency, whereby the level of accuracy and the estimate of the parameter values persist through different samples of the data. The third requirement is that of versatility. A versatile model may be defined as one, which is accurate and consistent, when subject to diverse applications.

Considering various factors, the simple linear model (SLM) and linear perturbation model (LPM) of Nash et al (1983) have been attempted for runoff prediction in the present programme. Various steps involved in calibrating and validating these models are given in the following paragraphs.

3.8.3 Model Calibration

Regardless of what rainfall-runoff model is selected, the key to accuracy of the result is calibration of the model. Calibration simply refers to the process of varying input data until the result match known runoff events. Once model characteristics are determined, which recreate known events, it is possible to predict runoff from other rainfall events within the basin with a reasonable degree of accuracy. Also, once a model is properly calibrated for a basis with known characteristics, it is reasonably safe to transfer the use of the model to nearby basins of similar characteristics.

An examination of the parameter stability and the significance of model parts are necessary. This is achieved by dividing the available record into two periods, in one of which the model is calibrated and in the other it is tested (verification period). The practice has been to use most of the available data in calibration and to continue the verification period to one or two years. Standard errors of estimates of model parameter values would be most useful in assessing the significance of separable model parts and stability of parameter estimation.

3.8.4 Model Equations

Efficiency Criteria.

Criteria which expresses model accuracy are generally linked with the objective function used for optimising its parameters. A commonly used objective function is the sum of squares of the difference between the observed (y) and the estimated ($\hat{y}$) discharges, with the summation taken over the whole of the calibration period.

$$F = \Sigma(y - \hat{y})^2 \tag{80}$$

F is an index of residual error which reflects the extent to which a model is successful in reproducing the observed phenomenon. But Nash et al (1970) introduced a dimensionless quantity. The efficiency R^2 is analogous to the coefficient of determination in linear regression, as one minus the proportion of the initial variance represented by F. Defining the initial variance F_o as

$$F_o = \Sigma(y - \bar{y})^2 \tag{81}$$

where,

$$\bar{y} = \frac{1}{n}\sum_{i=1}^{n} y_i \qquad (82)$$

n is the number of observations

The criterion becomes

$$\mathrm{R}^2 = \frac{F_0 - F}{F_0} \qquad (83)$$

In application to the calibration period, these quantities are all obtained within that period.

The memory length

In most of the perennial streams, discharge continues even for months after the cessation of rainfall. The interval between the occurrences of rainfall and the time when its effect on the streamflow finally ceases is, known as memory length. When the operation is represented by an equation

$$y_\mathrm{i} = \sum_{j=1}^{m} x_{i-j+1}\, h_\mathrm{j} + e_i \qquad (84)$$

where, h_j refers to the j$^{\mathrm{th}}$ ordinate to the pulse response and *m* is the memory length of the system and must be estimated prior to the formation of the equation. The best estimate of m must be obtained by trial and error, choosing initially a large value and successively reducing this in accordance with the indication obtained for the tail values of the pulse response function. The relationship of each of the last few ordinates to their respective standard errors may be used as a guide to the memory length, which may be taken as ceasing when the estimated ordinates are not significantly different from zero. If the pulse response ordinates are estimated by the method of ordinary least squares, then provided the errors are of zero mean, constant variance and pairwise uncorrelated; their variance is given by Johnston (1972) as

$$\mathrm{var}(\hat{H}) = \sigma^2\, [X^T X]^{-1} \qquad (85)$$

An unbiased estimate of σ^2 is given by

$$S^2 = \frac{1}{(n-2m+1)}\sum_{j=m}^{n} u_j^2 \tag{86}$$

which is the mean of the squares of the residuals.

The Gain factor and volumetric Constraint

When the input and output are expressed in the same units, the ratio of the total output to the total input volume is known as the gain factor G. In general, it can be greater or less than unity. In flood routing, when no allowance is made for the inflow due to tributaries or the rainfall on the intervening catchment, one would expect a gain factor in excess of unity. On the other hand, in relating discharge to rainfall, the gain factor would be less than unity. For a conservative, single input, single output system, where the total input equals the total output, the true gain factor must necessarily be unity. The value of the gain factor is given by the sum of the ordinates of the pulse response multiplied by the interval between them.

The equation of the linear model can be written with the consideration of gain factor as

$$\hat{y}_i = G_{ols}\sum_{j=1}^{m} x_{i-j+1}\hat{z}_j + u_i \tag{87}$$

where G_{ols} is the gain factor associated with estimation by the method of ordinary least squares, and z_j are the ordinates of the standardised pulse response obtained by proportionately reducing all the ordinates of the derived pulse response function so that the sum, multiplied by the time interval between them is unity.

In obtaining the ordinate of the pulse response by least square estimation the true gain factor (G) is not necessarily preserved; the volume of the computed output is not necessarily equal to the volume of the observed output. To preserve the gain factor, a constrained least squares estimation procedure must be used.

3.8.5 The Simple Linear Model

The specific features of the simple linear model are presented in this section. In dealing with continuous functions, the input-output relationship for a lumped linear, time invariant, system expressed in terms of the impulse response functions is given by the convolution integral equation

$$y(t) = \int_{\tau=0}^{\infty} x(\tau)\, h(t-\tau)\, d\tau \tag{88}$$

where τ is the dummy variable of integration, h(t) is the impulse response function.

When the input function is expressed as a series of pulses or mean values over successive short intervals T, the response to a unit pulse of duration T is a more convenient expression of the operation of the system than the impulse response. The input-output relationship is expressed in terms of the pulse response by

$$y_i = \sum_{j=1}^{m} x_{i-j+1} h_j + e_i \tag{89}$$

where h_j refers to the jth ordinate of the pulse response ,m is the memory length which implies that the effect of any input, lasts only through m intervals of duration T, e_i is the error term.

Equation (89) can be written out for each output ordinate y_1 to y_n yields 'n' linear equations

$$\begin{bmatrix} y_1 \\ y_2 \\ \cdot \\ \cdot \\ y_m \\ y_{m+1} \\ \cdot \\ \cdot \\ y_n \end{bmatrix} = \begin{bmatrix} x_1 & 0 & \cdot\ \cdot & 0 \\ x_2 & x_1 & \cdot\ \cdot & 0 \\ \cdot & \cdot & \cdot\ \cdot & \cdot \\ \cdot & \cdot & \cdot\ \cdot & \cdot \\ x_m & x_{m-1} & \cdot\ \cdot & x_1 \\ x_{m+1} & x_m & \cdot\ \cdot & x_2 \\ \cdot & \cdot & \cdot\ \cdot & \cdot \\ \cdot & \cdot & \cdot\ \cdot & \cdot \\ x_n & x_{n-1} & \cdot\ \cdot & x_{n-m+1} \end{bmatrix} \begin{bmatrix} h_1 \\ h_2 \\ \cdot \\ \cdot \\ h_m \\ \\ \\ \\ \\ \end{bmatrix} + \begin{bmatrix} e_1 \\ e_2 \\ \cdot \\ \cdot \\ e_m \\ \cdot \\ \cdot \\ \cdot \\ e_n \end{bmatrix} \tag{90}$$

where m is the memory length and n is the number of observations of y. Equation (90) can be rewritten as

$$Y = XH + E \qquad (91)$$

where Y is a [n,1] column vector of the output series, X is a [n, m] matrix of the input series, H is a [m, 1] column vector of the pulse response ordinates, E is an [n, 1] column vector of the errors.

In discovering the operation of the system, the values of input and output are assumed to be known and $H = [h_1, h_2 ...... h_m]^T$ must be found. If the objective is to minimise the sum of squares of difference between the observed and the estimated flows, then the optimum value of H can be determined directly by the method of ordinary least squares from:

$$\hat{H} = [X^T X]^{-1} X^T Y \qquad (92)$$

If new observations are continuously being made, the estimate of H can be updated step by step in the light of the new information. This can be done by the method of recursive least squares, Astrom (1968). In this work, the method of ordinary least squares was used for non-parametric, linear models which could be expressed in sets of linear equation. For certain parametric form, e.g, the gamma function form, which cannot be so expressed, and which are therefore unsuitable for this method, the estimation was done by a direct search technique due to Rosenbrock (1960).

Shape constrained expression of the linear model

The nature of the hydrologic system implies high damping, stability, conservative nature and the absence of feed back. The pulse response functions estimated by the method of ordinary least squares often exhibit unrealistic negative ordinates and oscillation.

Five main factors contributed to the problem are:

(a) Multicollinearity of the input series,

(b) Measurement errors in the input and the output series,

(c) Non-linearities in the relationship,

(d) Instability in the system when viewed in reverse (i.e. outputting the actual input or pulse response), and

(e) Inconsistency in the data.

In hydrological application, the input variable is often serially autocorrelated. This leads to an ill conditioned (X^TX) matrix, which in turn involves instability in inversion, a lack of precision in all the estimates and a tendency towards oscillation or fluctuations in the pulse response ordinates. Because numerical instability in the derivation of the pulse response results from an insensitivity in the output to changes in the input or pulse response, elimination of the fluctuation in the estimated pulse response will normally, produce little change in the output and consequently, little loss of efficiency in the model. Therefore, a constraint may be imposed on the shape of the pulse response to reduce the unacceptable fluctuation, with little loss of efficiency.

Bruen et al (1984) and Liang (1985) showed that the method of ridge regression can be used with success to estimate the ordinates of the smoothed pulse response. But in this study, a more reliable method, parametric modelling, is used.

Parametric solution - The gamma function model

A drastic constraint to the shape and volume of the estimated pulse response function is obtained by parametric modelling, wherein a solution is sought within the constraint of an assumed model form. Based on prior knowledge of the system behaviour, the response function of the system is represented by a suitable mathematical equation involving only a few parameters. These must, however, be estimated by a search in the space of reasonable values, rather by an algebraic method.

If the input x(t) is related to the output y(t) by a differential equation of the form

$$y(t) = \frac{1}{(1+KD)^n} x(t) \qquad (93)$$

(where K is a constant having the dimension of time, and n a numerical constant, D is the differential operator). Then the corresponding impulse response function is given by

$$h(t) = \frac{1}{K\Gamma(n)} e^{-(t/K)} (t/K)^{n-1} \qquad (94)$$

where

$$\Gamma(n) = \int_0^\infty e^{-x} x^{n-1} dx$$

is the gamma function of n. If n is the integral, the system corresponds to exactly to a series of n equal linear reservoirs each of storage S equal to K_y.

It was shown by Nash (1960) that under the constraints of conservation (gain factor equal to unity), stability, high damping and the absence of feed back, the two parameter equation with n integral and K positive, is almost as general a model as the differential equation of unlimited order given by

$$y(t) = \frac{1}{(1 + A_1 D + A_2 D^2 + ...)} x(t) \qquad (95)$$

With the additional flexibility obtained by allowing n to take fractional values, the impulse response of this equation has the ability to represent, adequately, almost all impulse response shapes commonly encountered in the hydrological context.

If the input is presented as a series of pulses (blocks of uniform intensity over short durations T) and the output similarly expressed as ordinates at intervals T, then the linear relationship is most conveniently expressed through the pulse responses. For a system, whose impulse response is given by the gamma function model (94), the corresponding pulse response can be obtained numerically by

$$h(T,t) = (S(t) - S(t-T))/T \tag{96}$$

where,

$$S(t) = \int_0^t h(t)dt = \frac{1}{K\Gamma(n)} \int_0^t e^{(-\tau/K)} (\tau/K)^{n-1} dt$$

Use of equation (94) or (96) for the pulse response implies a unit gain factor. To achieve any other gain factor it is necessary to multiply the equation by the appropriate quantity.

The equation of the single input, linear model can be rewritten as,

$$y_i = G_g \sum_{j=1}^{n} h_j(T,t) x_{i-j+1} + e_i \tag{97}$$

where $h_j(T,t)$ is given by equation (96) and G_g is the gainfactor. The parameters of this model can be estimated by progressive estimation, implying a search recommended by Rosenbrock(1960) in the n, K, G_g space. The parameter pair n and nK should be chosen rather than n and K, because n is a shape parameter and the product nK is a scale parameter.

3.8.6 The Linear Perturbation Model

For rivers with a relatively predictable seasonal variation, a considerable improvement on conventional rainfall-runoff models may be obtained by concentrating on the relationship between the series of departures from the seasonal behaviour in rainfall and discharge as input and output, respectively rather than, as conventionally, treating the total rainfall series as input and total discharge series as output (Kachroo et al, 1988). The linear perturbation model (LPM) may be viewed as an attempt to overcome the basic weakness inherent in the classical unit hydrograph approaches, wherein the departures (storm runoff) from the base flow are related by a linear model to the corresponding departures(rainfall excess) from the corresponding rainfall loss series. The physically undefined base flow and rainfall loss series are replaced, in the case of LPM, by the physically defined seasonal mean daily series of input and output respectively. The LPM concept was developed and exposed by Nash et al (1983), in the context of rainfall-runoff modelling. The model is essentially a combination of two well established concepts, one in time series analysis (seasonal component identification) and the other in deterministic system analysis (classic unit hydrograph identification). This model is based on the following assumptions:

(i) If in a particular year, each input function, is equal, for each day of the year to its expected value for that date, the output will also equal its expectation for that date. That is, if the expected values of rainfall and discharge on each date (d) are denoted by i_d and q_d, respectively, then the input i_d produces the output q_d:

$$\{i_d \rightarrow q_d\}$$

(ii) Perturbation or departures from the date expected input values (i_d), are linearly related to the corresponding perturbations or departures from the date expected output values (q_d):

$$\{(i-i_d) \rightarrow (q-q_d)\}$$

On catchments with highly seasonal variation of discharge, the subtraction of the seasonal means from the original series would remove much of the dependence on linearity, ie., the assumption of a linear relationship between the departures would be less restrictive than the same assumption concerning the actual input and output values. For a catchment, which does not exhibit marked seasonal behaviour, or for which the relationship between total input and total output is nearly linear, it is to be noted that the LPM would be of only similar efficiency to the simple linear model.

For a single input series the LPM may be described by

$$y_i = \sum_{j=1}^{m} x_{i-j+1} h_j + e_i \qquad i = 1,2,\ldots\ldots, n \qquad (98)$$

where $y_i = q_i - q_d$, $x_i = i_i - i_d$, $d = 1,2,....365$. Having determined the seasonal component q_d, the model may be calibrated by the method of ordinary least squares, with or without volumetric constraints. Constraints on the shape of the pulse response function can also be imposed by parametric modelling.

ANALYSIS AND RESULTS

Soney John "Statistical analysis of peak flows with special reference to Kerala river basins" Thesis. Department of Statistics, University of Calicut, 1999

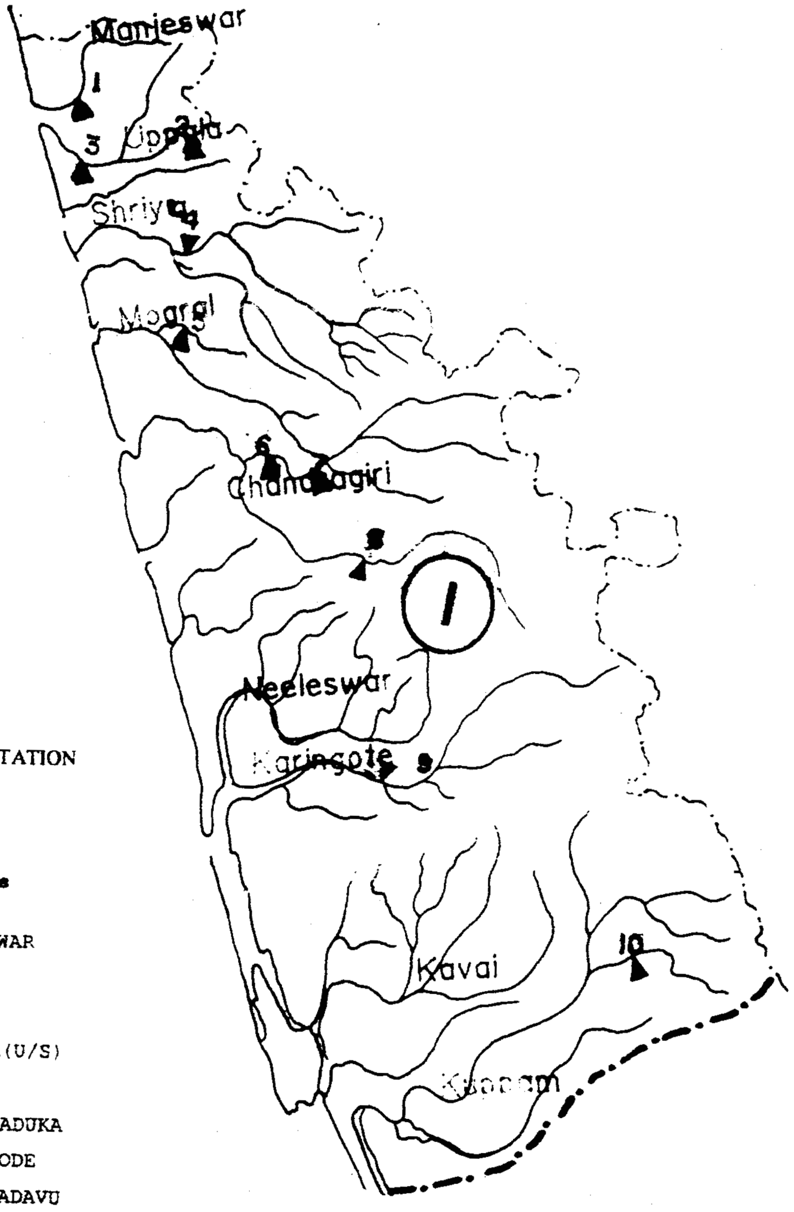
4. ANALYSIS AND RESULTS

4.1 TESTS FOR HOMOGENEITY

All relevant and available data were collected as described in Chapter 3. The present chapter furnishes the analysis of data and results; the detailed methodology for all relevant analysis is given in Chapter 3. The randomness of the data was checked using serial correlation coefficients. In all cases, it is found that the observed (r_1) lies within the ranges, so that the data are consistent with the hypothesis of randomness. River basins were combined together for the purpose of the study, following the procedures described. The homogeneity has been tested; the results of homogeneity tests are given in Table 7. Comparing the calculated values with the values obtained from the standard tables, it is observed that each of the region is homogeneous with respect to the variable under consideration. Regions are given in Figs. 12 – 19.

TABLE 7: RESULTS OF HOMOGENIETY TEST FOR NINE REGIONS

Region No	Number of Sites	Number of site-years	Calculated R	Tabled value
1	10	172	12.007	16.91
2	07	158	5.0161	12.59
3	10	230	3.4493	16.91
4	12	286	3.2084	19.67
5	21	489	8.3532	31.40
6	08	161	11.2638	14.10
7	12	245	2.8192	19.67
8	09	143	8.6423	15.50
9	09	141	6.7853	15.50



LEGEND

▲ - DISCHARGE STATION

REGION-1

Station No	Discharge Stations
1	MANJESHWAR
2	ANNAKAL
3	UPPALA
4	SHIRIYA(U/S)
5	MADHUR
6	PADIATHADUKA
7	PALLANGODE
8	MOONAMKADAVU
9	KAKKADAVU
10	MANGARA

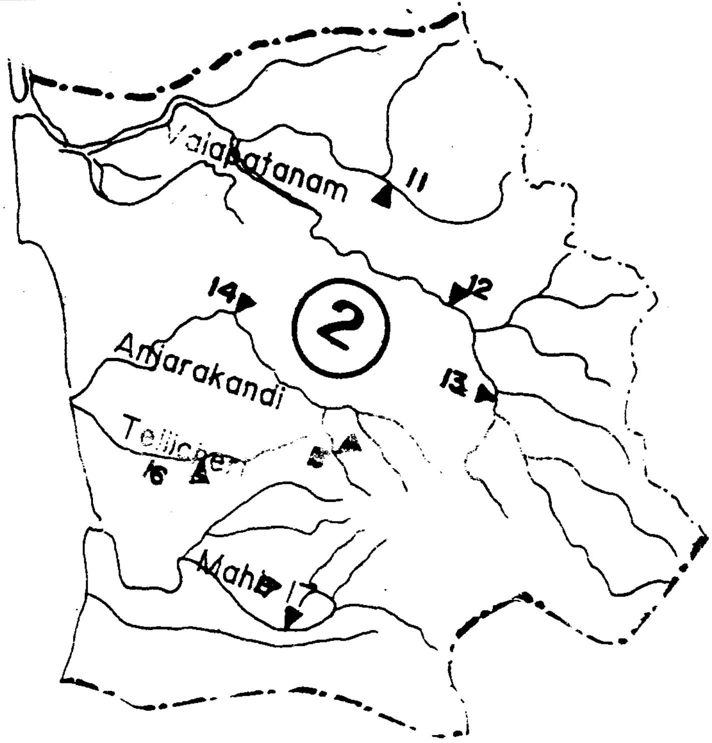
Fig 12 : LOCATION OF DISCHARGE STATIONS – REGION - 1

LEGEND

▲ - DISCHARGE STATION

REGION-2

Station No	Discharge Stations
11	IRUDE
12	KUTTLOOR
13	PALA
14	MERUVAMBAI
15	KANNAVAM
16	PATTIAM
17	VALAYAM



REGION-3

Station No	Discharge Stations
18	BAVELI
19	THIRUNELLY
20	THONDAR
21	MANAMTODY
22	CHOORANI
23	PANAMARAM
24	KAKKAVAYAL
25	MUTHANGE
26	VAZHAVATTA
27	MANJAT

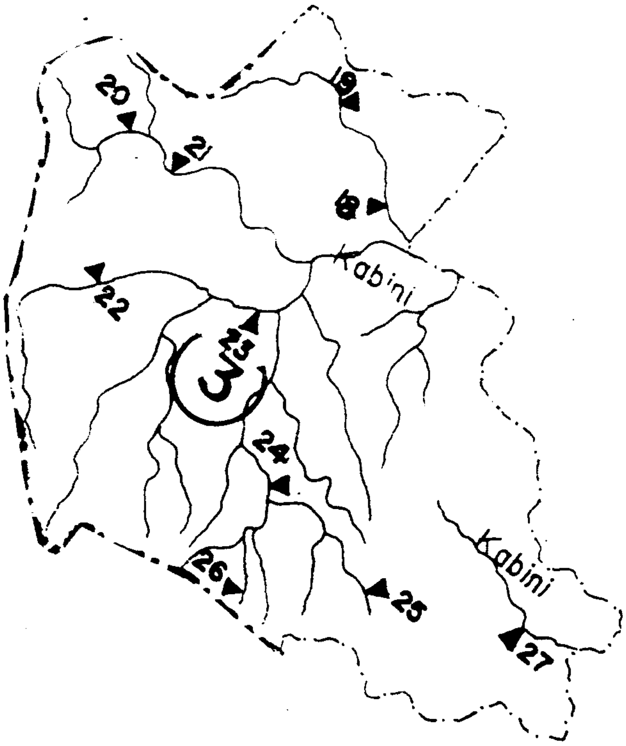
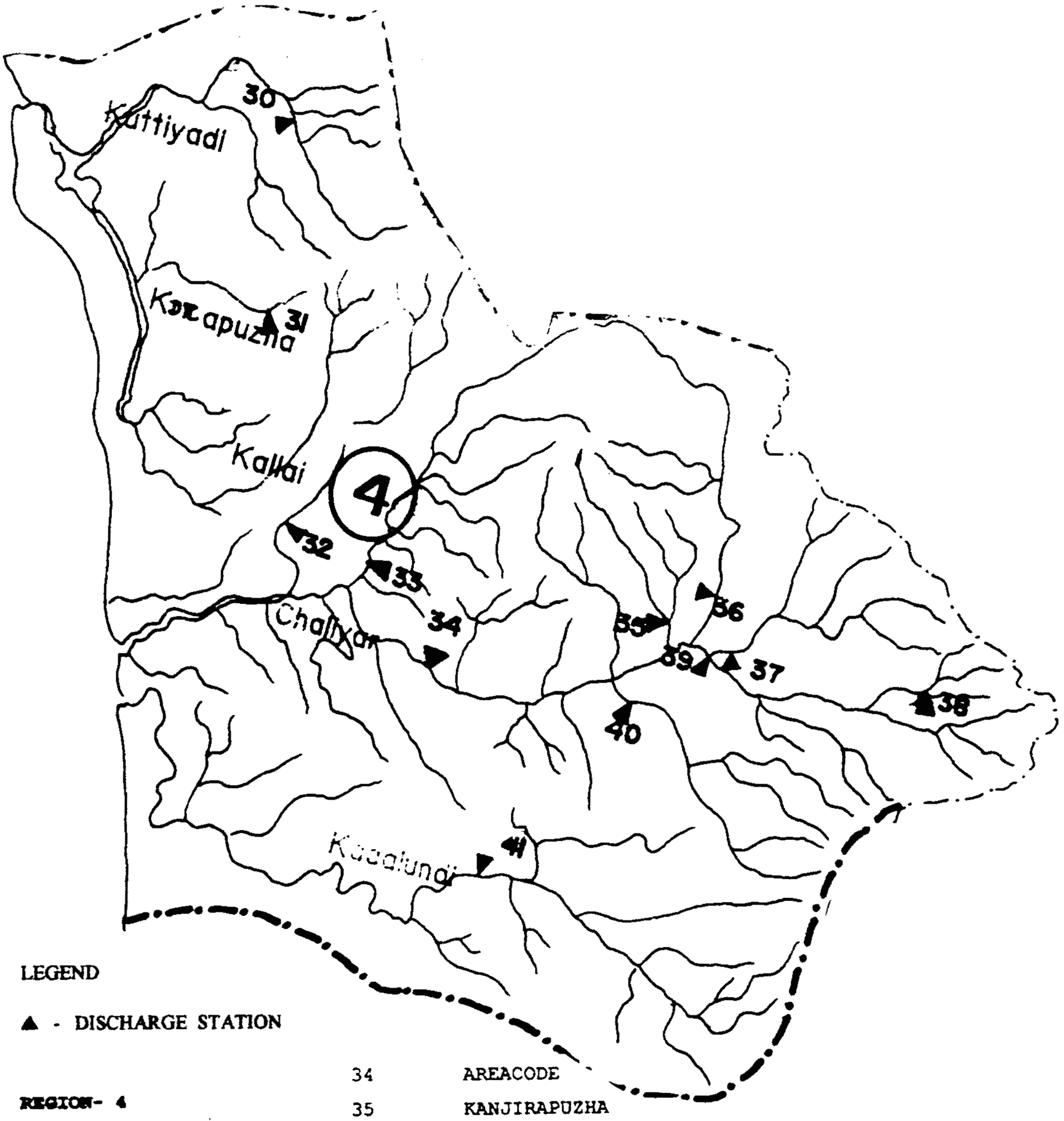


Fig 13 : LOCATION OF DISCHARGE STATIONS – REGIONS – 2 & 3



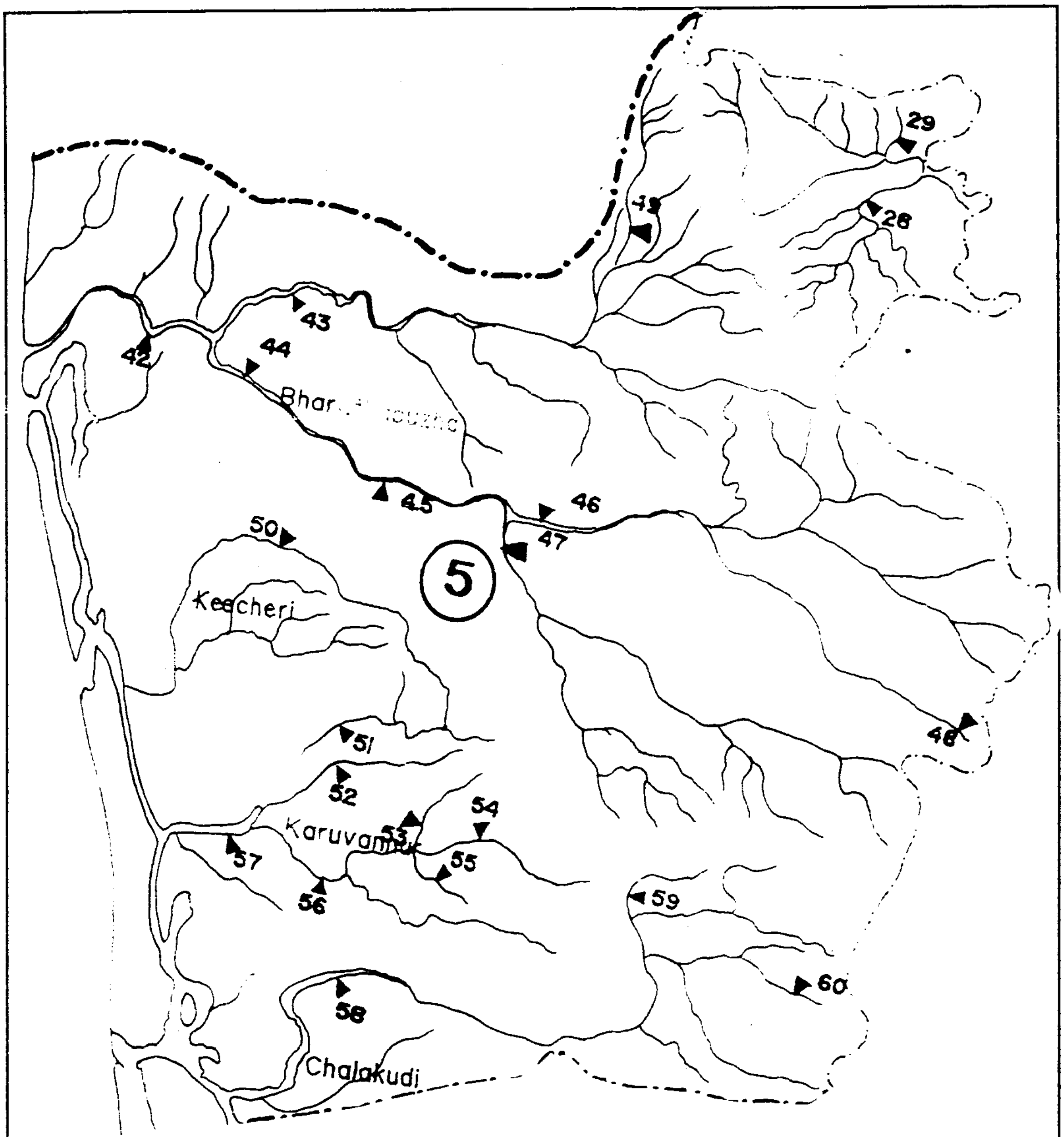
LEGEND

▲ - DISCHARGE STATION

REGION- 4

Station No	**Discharge Stations**
30	KADAMTHARAPUZHA
31	KOLLICKAL
32	KOODATHAI
33	MUKKOM
34	AREACODE
35	KANJIRAPUZHA
36	CHALIYAR
37	PUNNAPUZHA
38	MARUTHAPUZHA
39	KARIMPUZHA
40	KUTHIRAPUZHA
41	ANAKAYAM

Fig 14 : LOCATION OF DISCHARGE STATIONS – REGION – 4



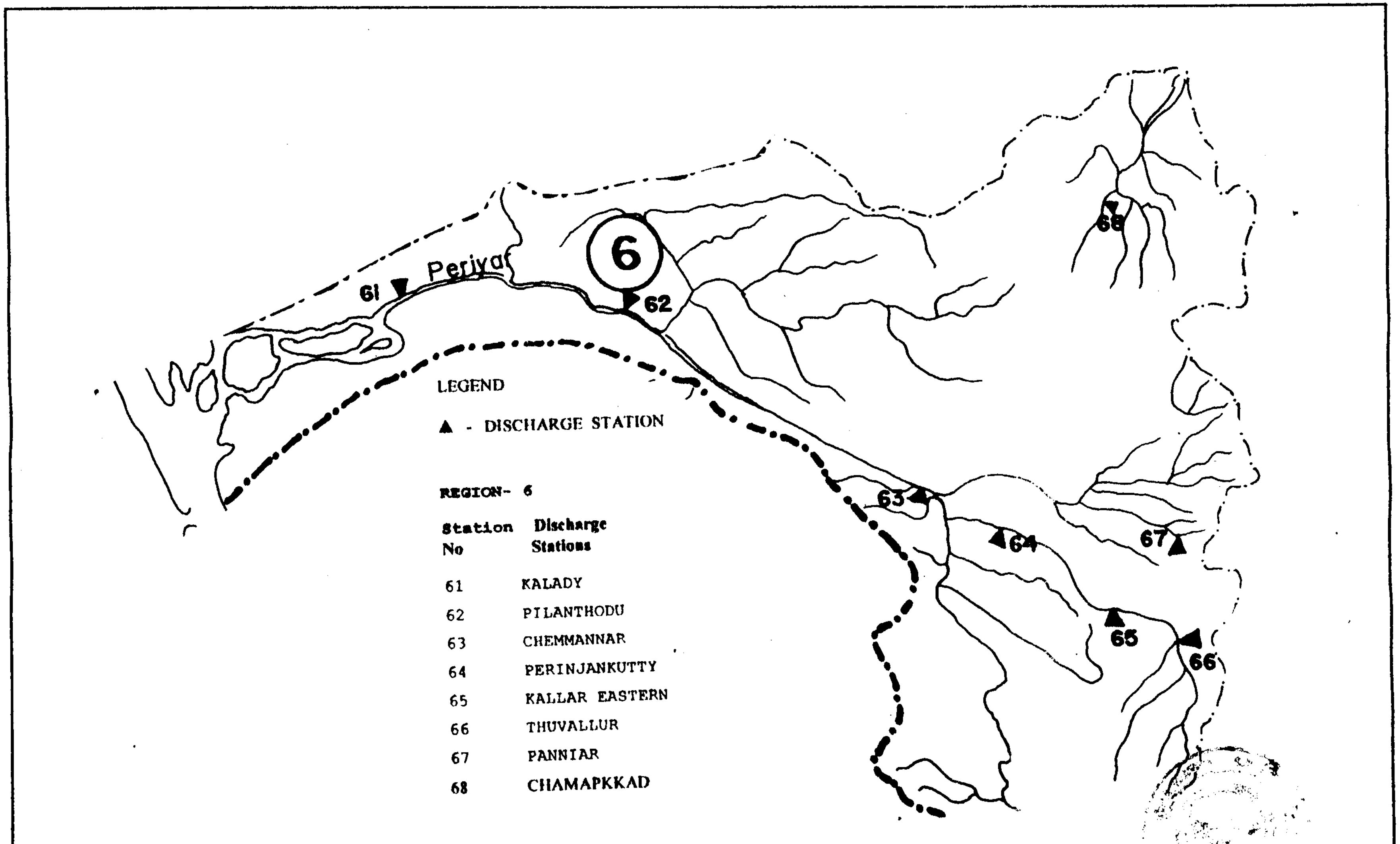
LEGEND

▲ - DISCHARGE STATION

REGION- 5

Station No	**Discharge Stations**
42	THIRUVAGAPUZHA
43	THRITHALA
44	CHERUTHURUTHY
45	PAMPADY
46	CHEERAKUZHI
47	MANAKKADAVU
48	KUTTIPURAM
49	SILENT VALLEY
50	PAZHOOR
51	KUNDUKADAVU
52	MANALI
53	PILATHODU
54	CHIMONI
55	MUPPLY
56	KURUMALI
57	KARUVANNUR
58	AMBALAKADAVU
59	KARAPPARA
60	KURIARKUTTY
28	CHITTOOR
29	CHAVADIYOOR

Fig 15 : LOCATION OF DISCHARGE STATIONS – REGION – 5



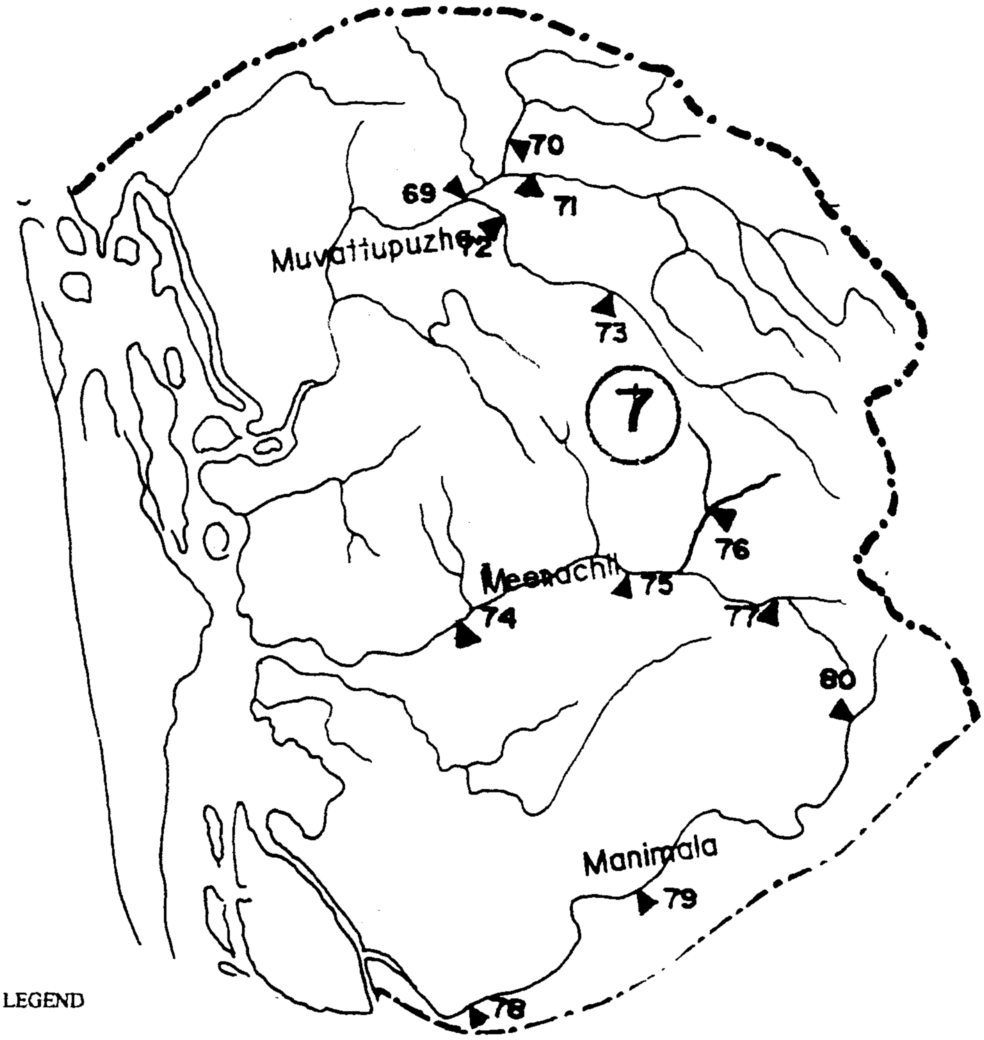
LEGEND

▲ - DISCHARGE STATION

REGION- 6

Station No	Discharge Stations
61	KALADY
62	PILANTHODU
63	CHEMMANNAR
64	PERINJANKUTTY
65	KALLAR EASTERN
66	THUVALLUR
67	PANNIAR
68	CHAMAPKKAD

Fig 16 : LOCATION OF DISCHARGE STATIONS – REGION -6



LEGEND

▲ - DISCHARGE STATION

REGION- 7

Station No	**Discharge Stations**		
69	MUVATTUPUZHA	75	PALAI
70	KAKADASSERRY	76	CHERIPPAD
71	KALAMPUR	77	TEEKOY
72	THODUPUZHA	78	THONDRA
73	MALANKARA	79	MANIMALA
74	PEROOR	80	MUNDAKAYAM

Fig 17 : LOCATION OF DISCHARGE STATIONS – REGION – 7

LEGEND

▲ - DISCHARGE STATION

REGION- 8

Station No	Discharge Stations
81	ERAPUZHA
82	KURUDAMANNIL
83	KOLLAKADAVU
84	PANDALAM
85	KONNY
86	ANAYADI
87	ENATHU
88	PUNALUR
89	THENMALA

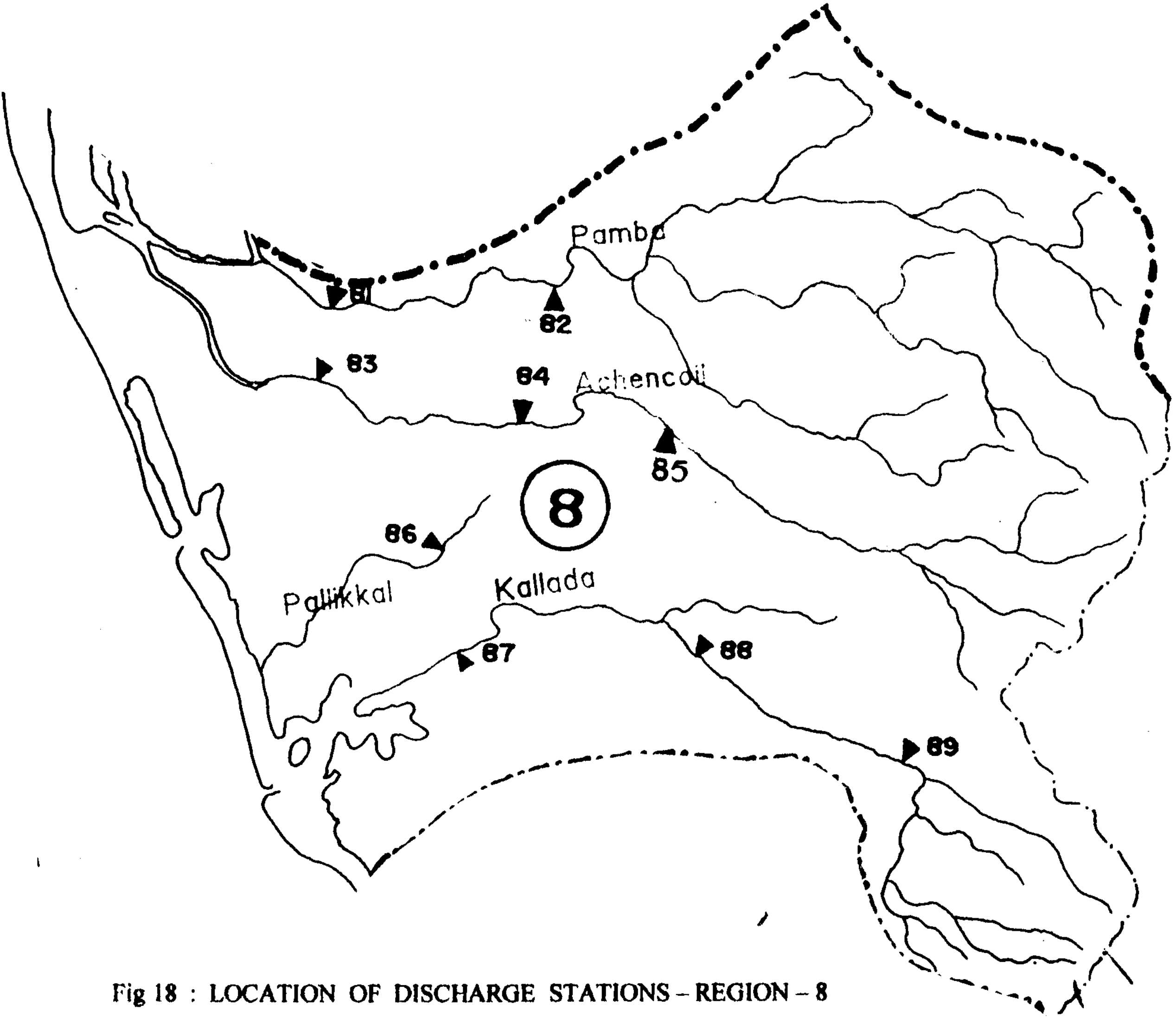
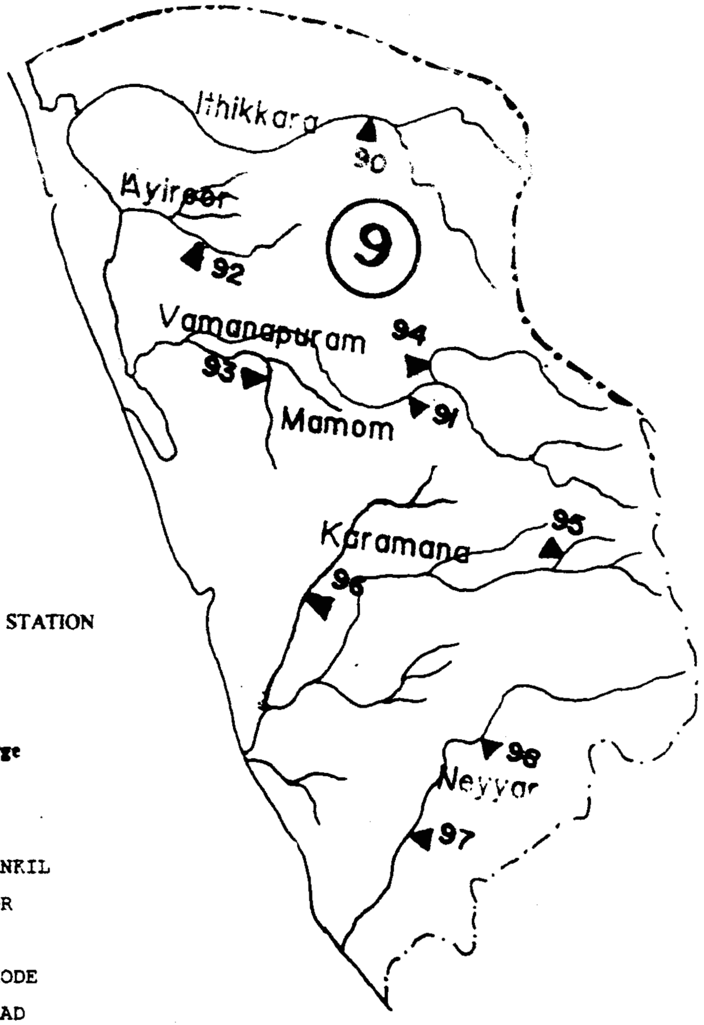


Fig 18 : LOCATION OF DISCHARGE STATIONS – REGION – 8



LEGEND

▲ - DISCHARGE STATION

REGION- 9

Station No	Discharge Stations
90	AYOOR
91	VALAYANKIL
92	AYIROOR
93	MAMEM
94	MYLAMMODE
95	ARAYANAD
96	MARUTHANKUZHY
97	AMARAVILA
98	OTTASEKARA- MANGALAM

Fig 19 : LOCATION OF DISCHARGE STATIONS – REGION – 9

4.2 FLOOD PREDICTION

4.2.1 Analytical Method

In analytical method, Gumbel (EV1), log-normal, Pearson type III and log-Pearson type III distributions were used to fit the standardised data. The parameter estimation, flood estimation and computation of standard error etc. were done as described in the methodology (Chapter 3).

The program developed by Kite (1977) for parameter estimation, fitting of distribution and flood estimation and other related analysis, has been used (NIH, 1986-87). The distributions selected for each region, after applying Chi-square goodness of fit test, are given in Table 8.

In some cases, maximum likelihood estimators did not yield results. Therefore, the results obtained by the method of moments were only adopted.

TABLE 8: DISTRIBUTIONS SELECTED FOR THE NINE REGIONS

Region	Distribution
1	Log-Pearson type III
2	Log-Pearson type III
3	Log-Pearson Type III
4	Gumbel (EVI)
5	Gumbel (EV1)
6	Gumbel (EV1)
7	Gumbel (EV1)
8	Gumbel (EV1)
9	Log-Pearson type III

The return periods, estimated floods and standard errors estimated using the fitted distributions for different regions are given in Tables 9-17.

TABLE 9 : RETURN PERIODS, ESTIMATED FLOODS AND STANDARD ERRORS: REGION – 1

Return period (years)	Estimated flood (cumec)	Standard error
2	0.814430	5.582739E-02
5	1.381742	9.537123E-02
10	1.847740	0.1351875
20	2.366766	0.1982709
50	3.153366	0.3353173
100	3.837126	0.4906096

TABLE 10: RETURN PERIODS, ESTIMATED FLOODS AND STANDARD ERRORS: REGION – 2

Return period (years)	Estimated flood (cumec)	Standard error
2	0.908301	5.346443E-02
5	1.395405	2.745378E-02
10	1.696711	7.644814E-02
20	1.971455	0.1100702
50	2.310113	0.1476041
100	2.553952	0.1731574

TABLE 11: RETURN PERIODS, ESTIMATED FLOODS AND STANDARD ERRORS: REGION - 3

Return period (years)	Estimated flood (cumec)	Standard error
2	0.804709	4.69035E-02
5	1.369134	7.727209E-02
10	1.810011	0.1039721
20	2.280876	0.1441604
50	2.961120	0.2298761
100	3.525490	0.3250712

TABLE 12: RETURN PERIODS, ESTIMATED FLOODS AND STANDARD ERRORS: REGION - 4

Return period (years)	Estimated flood (cumec)	Standard error
2	0.9023507	3.26395E-02
5	1.44878	5.577306E-02
10	1.810564	7.568721E-02
20	2.157596	9.584263E-02
50	2.606793	0.1226234
100	2.943403	0.142962

TABLE 13: RETURN PERIODS, ESTIMATED FLOODS AND STANDARD ERRORS : REGION - 5

Return period (years)	Estimated flood (cumec)	Standard error
2	0.887775	2.920154E-02
5	1.506563	4.967395E-02
10	1.916254	6.731451E-02
20	2.309239	8.517884E-02
50	2.817919	0.1089233
100	3.199102	0.1269593

TABLE 14: RETURN PERIODS, ESTIMATED FLOODS AND STANDARD ERRORS : REGION - 6

Return period (years)	Estimated flood (cumec)	Standard error
2	0.6626403	0.1228249
5	2.227854	0.2115245
10	3.264163	0.2877458
20	4.258214	0.3648156
50	5.544912	0.4677612
100	6.509110	0.5448625

TABLE 15: RETURN PERIODS, ESTIMATED FLOODS AND STANDARD ERRORS : REGION –7

Return period (years)	Estimated flood (cumec)	Standard error
2	0.9256796	2.826283E-02
5	1.365075	4.838115E-02
10	1.655993	6.569284E-02
20	1.935049	8.321033E-02
50	2.296258	0.106483
100	2.566933	0.124156

TABLE 16: RETURN PERIODS, ESTIMATED FLOODS AND STANDARD ERRORS : REGION – 8

Return period (years)	Estimated flood (cumec)	Standard error
2	0.91615	3.995508E-02
5	1.39782	6.895078E-02
10	1.71672	0.0938553
20	2.02262	0.1190308
50	2.41858	0.1524579
100	2.71530	0.1778338

TABLE 17: RETURN PERIODS, ESTIMATED FLOODS AND STANDARD ERRORS : REGION - 9

Return period (years)	Estimated flood (cumec)	Standard error
2	0.994416	7.606906E-02
5	1.781734	0.1203468
10	2.323615	0.1663955
20	2.838447	0.2474647
50	3.485945	0.4132826
100	3.953404	0.5778023

4.2.2 Graphical Method

An attempt was also made to predict the floods using the graphical method. The regional flood frequency curves were developed for nine regions using Gumbel distribution. These curves are given in Figs. 20-28. A Single curve for the entire State has also been developed and is given in Fig. 29.

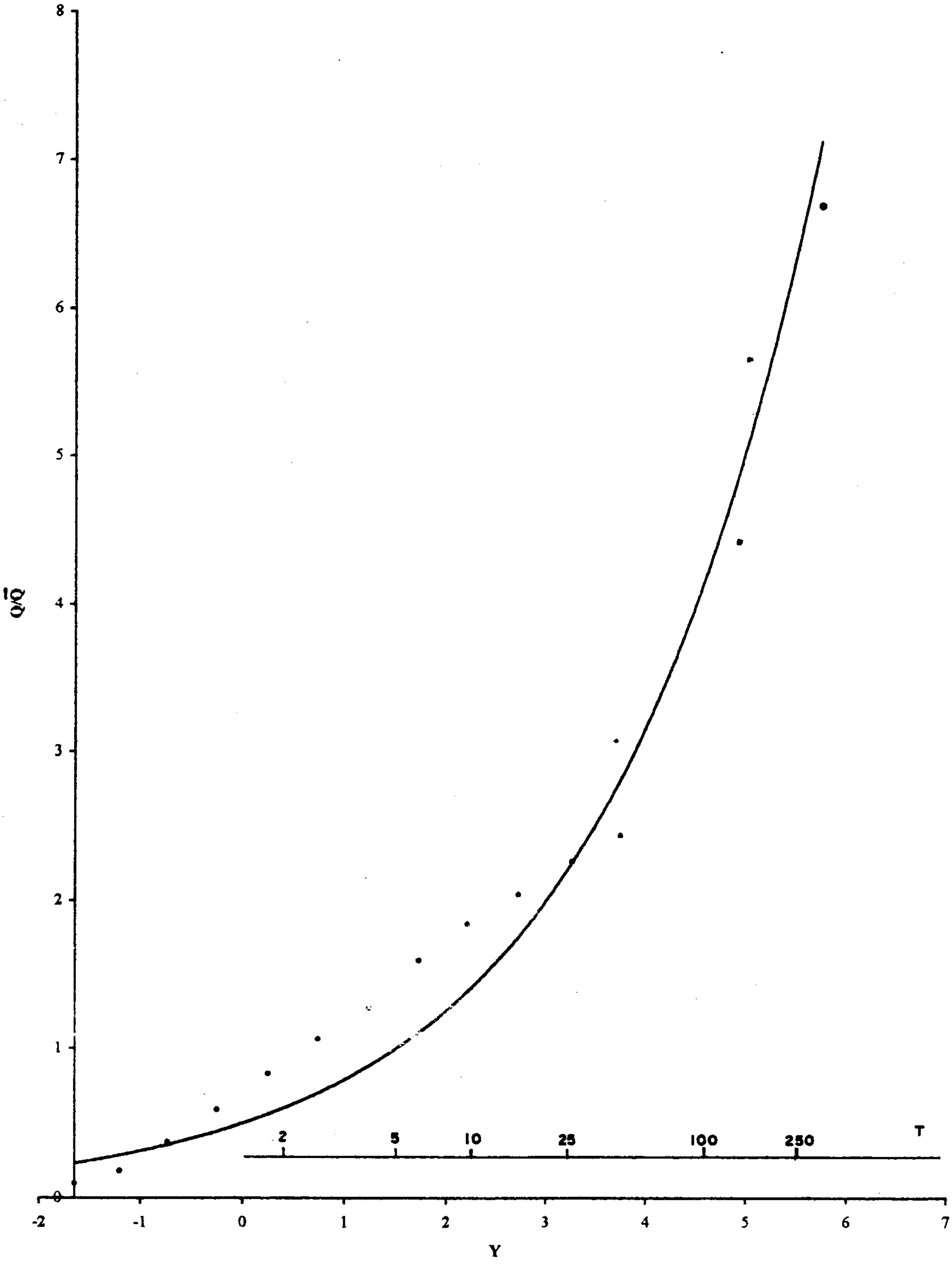


Fig : 20 Regional flood frequency curve - Region 1

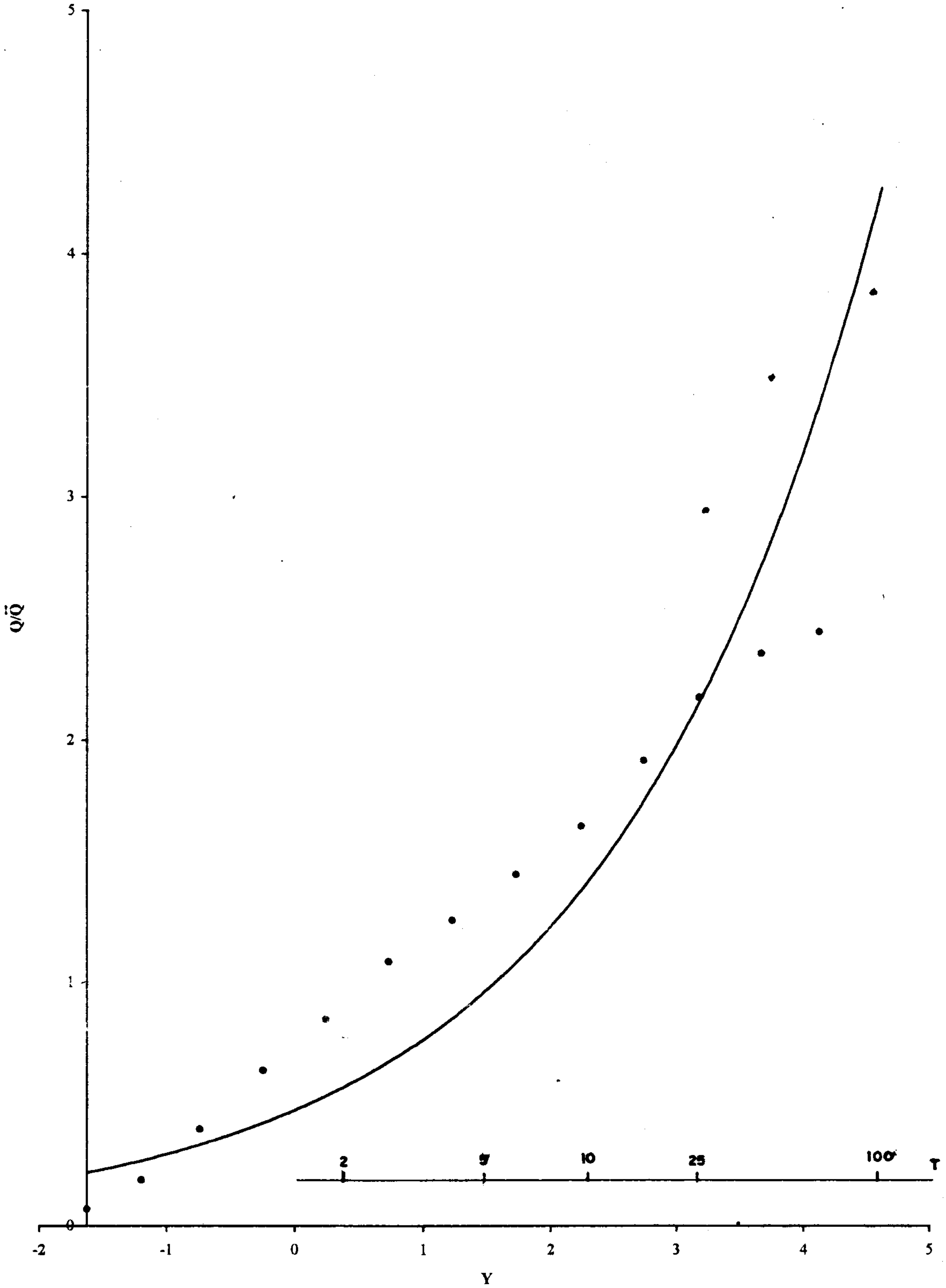


Fig : 21 Regional flood frequency curve - Region 2

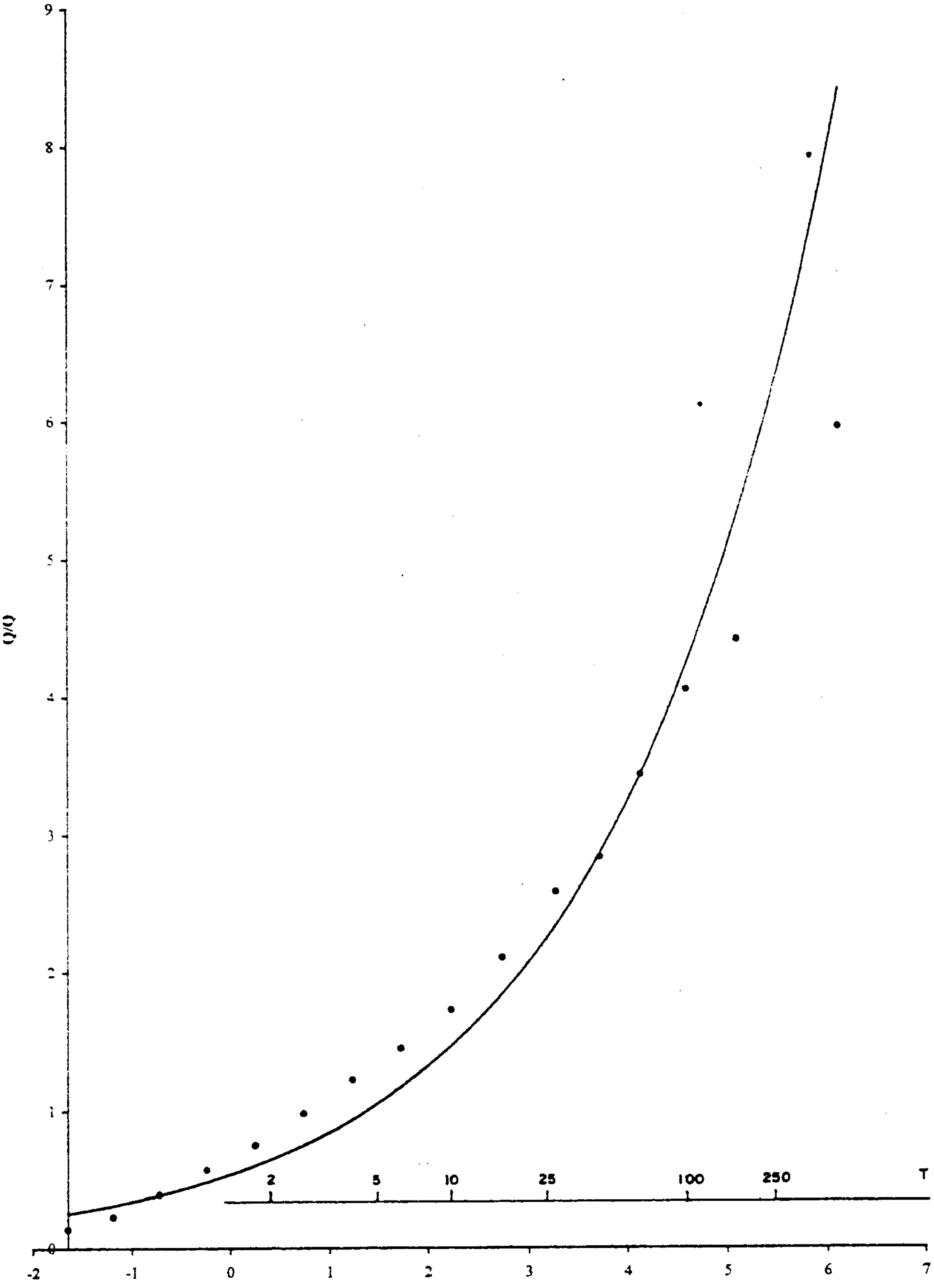


Fig : 22 Regional flood frequency curve - Region 3

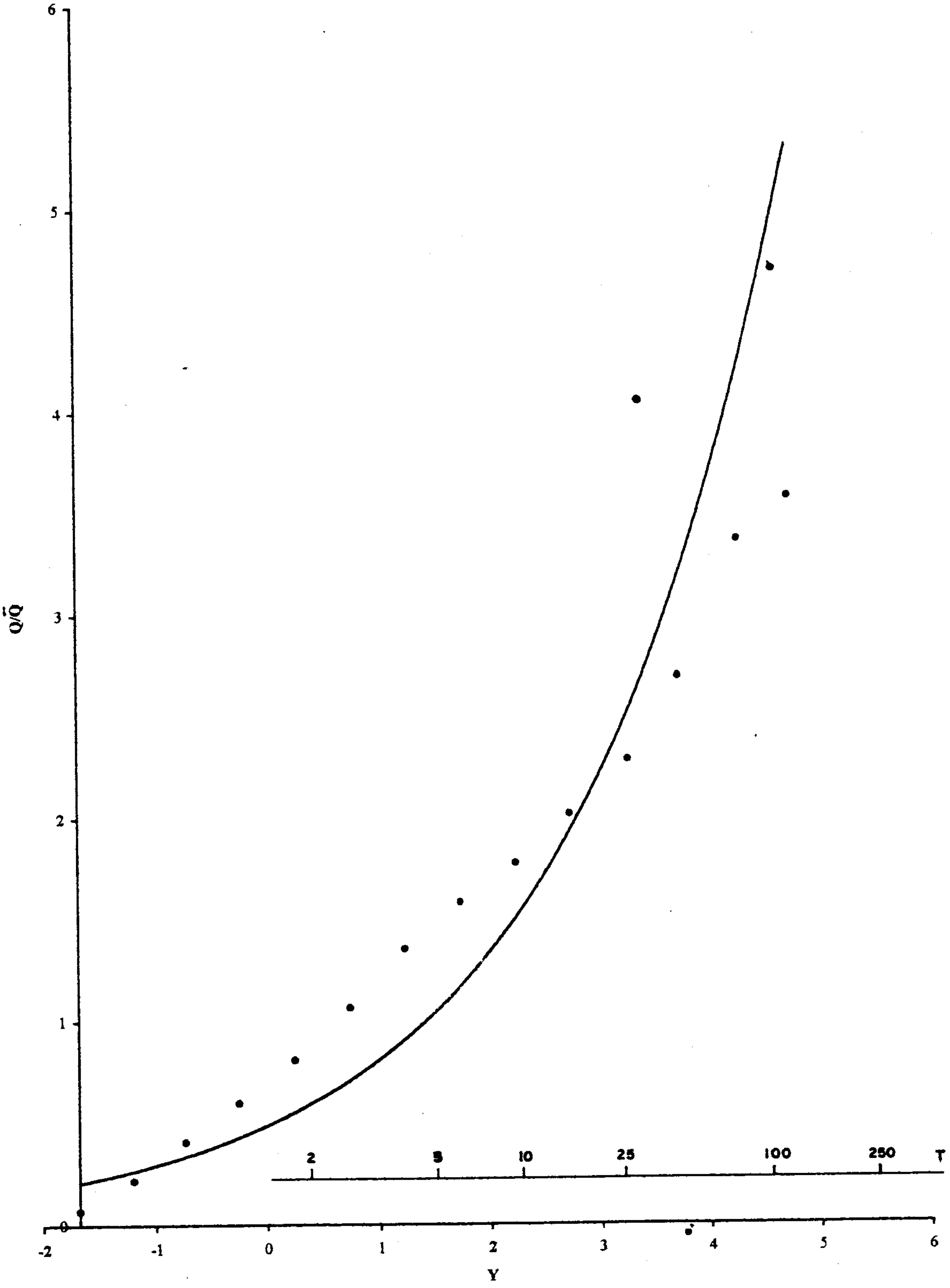


Fig :23 Regional flood frequency curve - Region 4

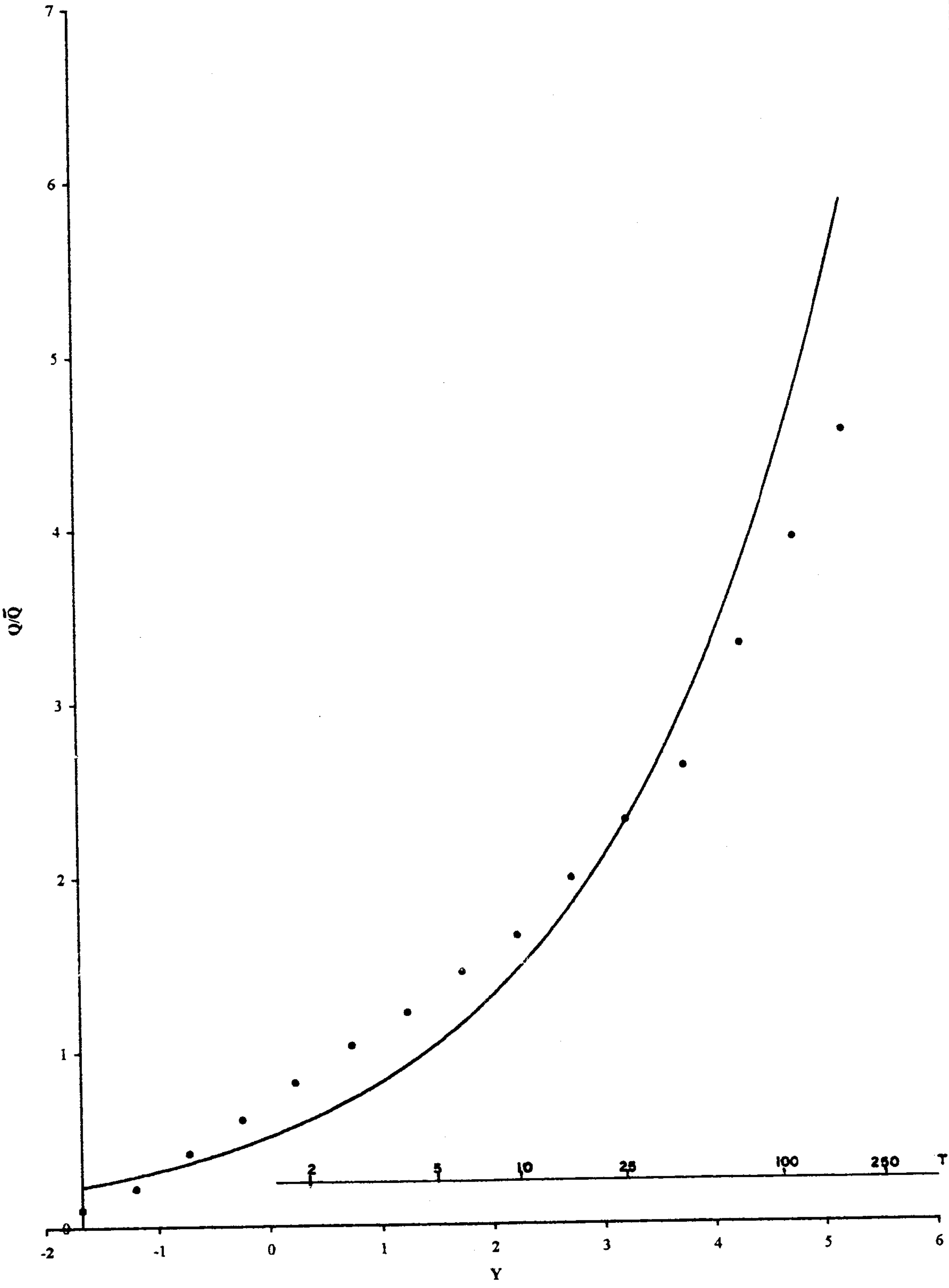


Fig : 24 Regional flood frequency curve - Region 5

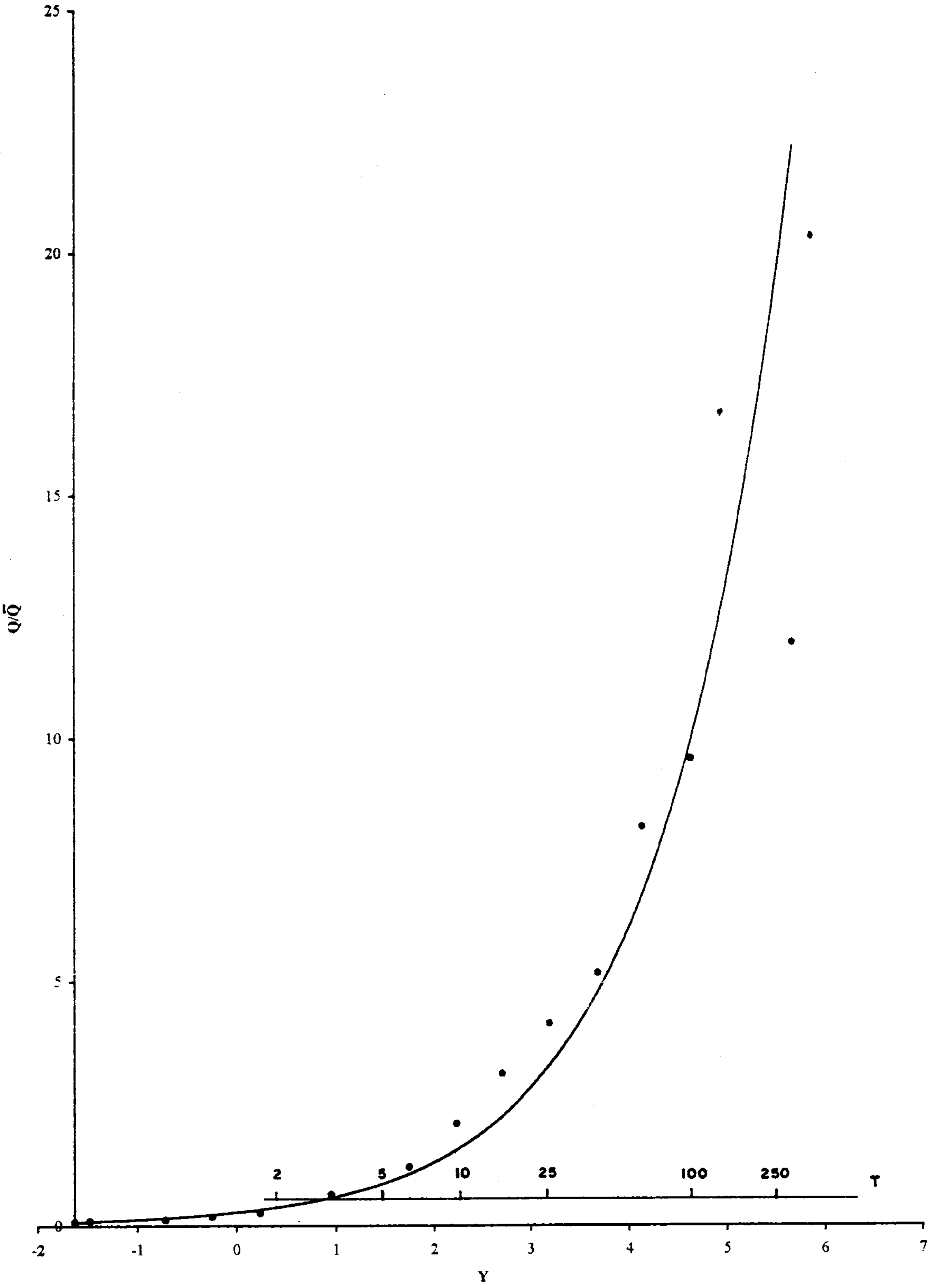


Fig : 25 Regional flood frequency curve - Region 6

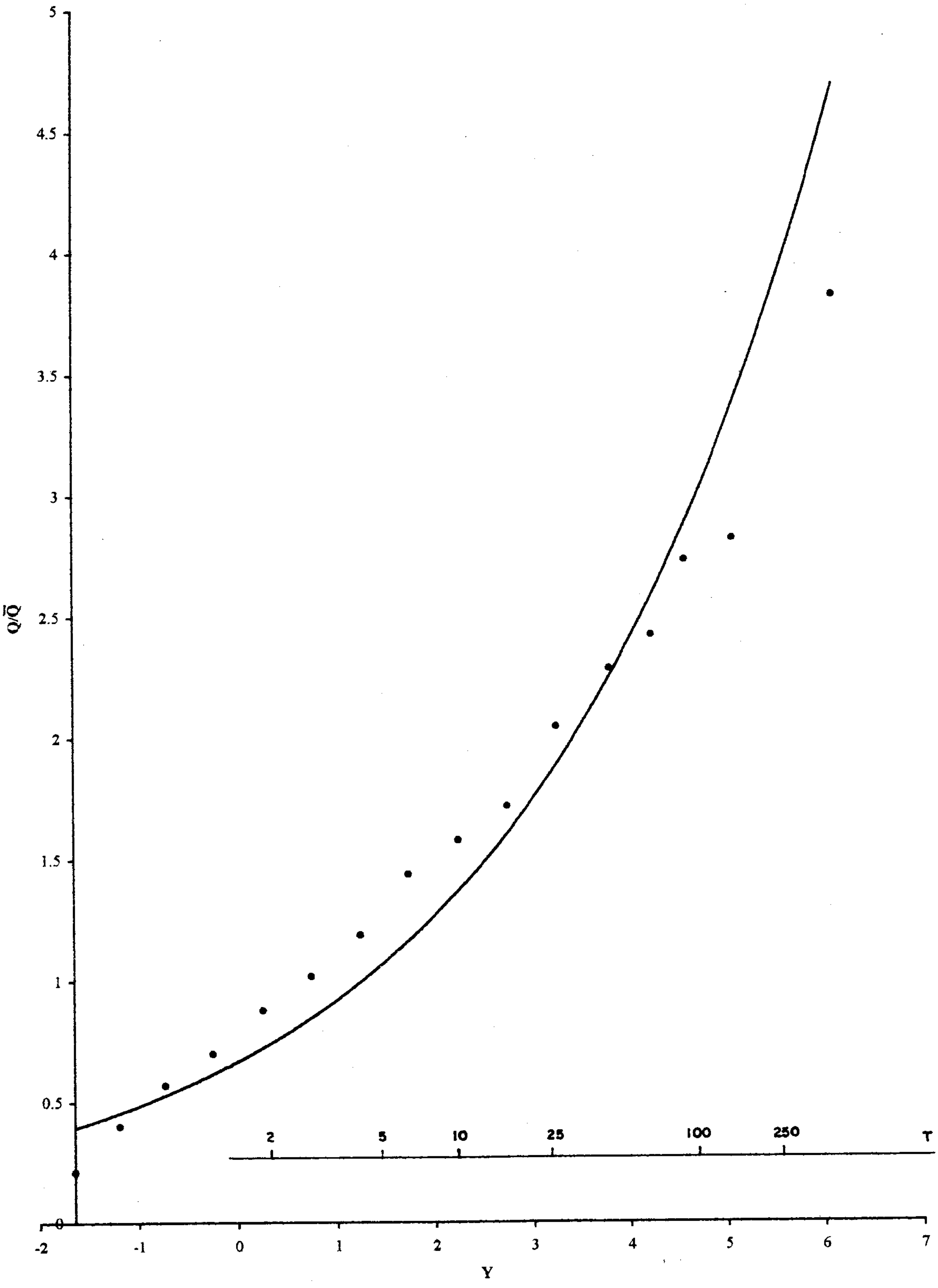


Fig : 26 Regional flood frequency curve - Region 7

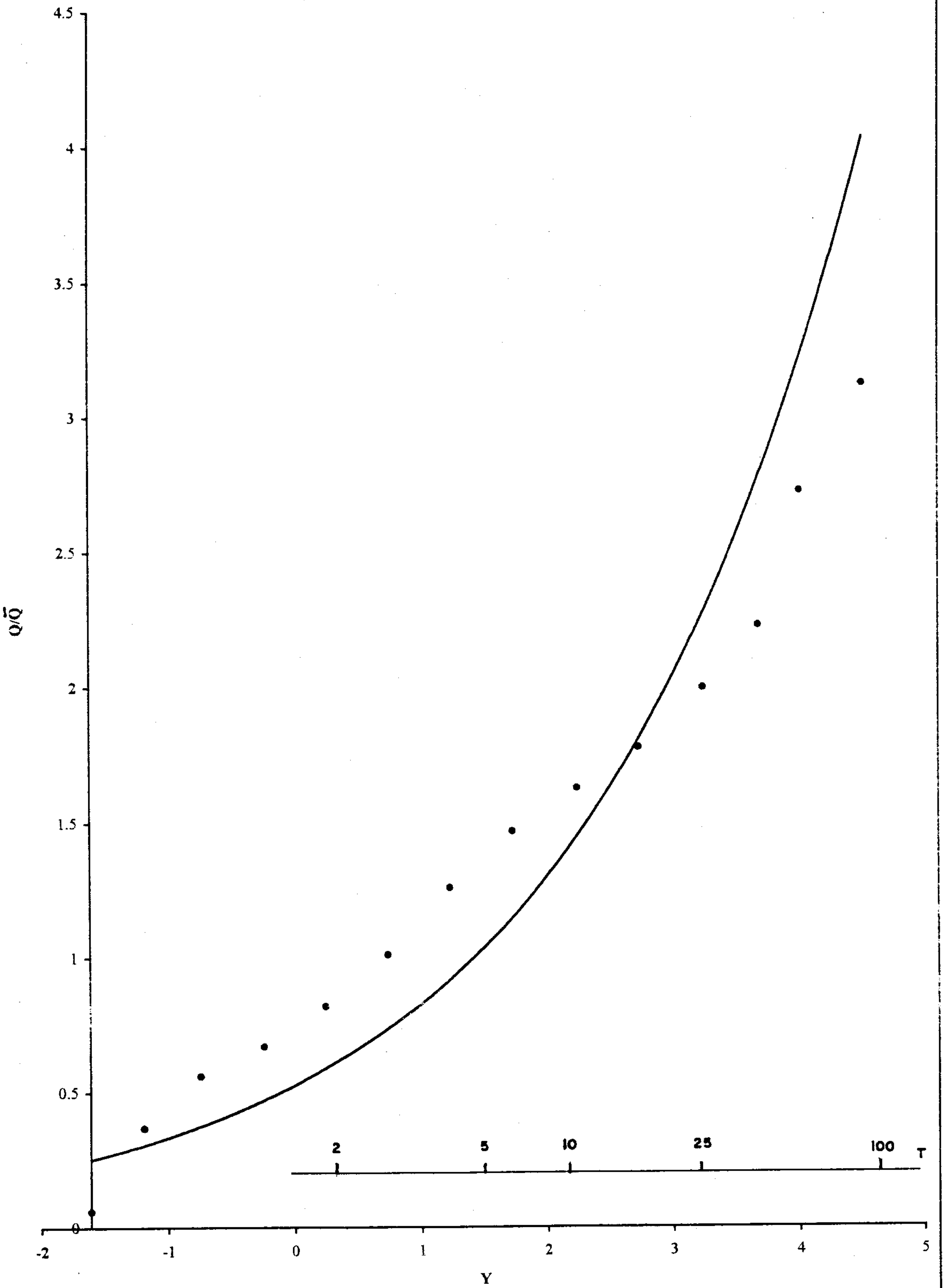


Fig : 27 Regional flood frequency curve - Region 8

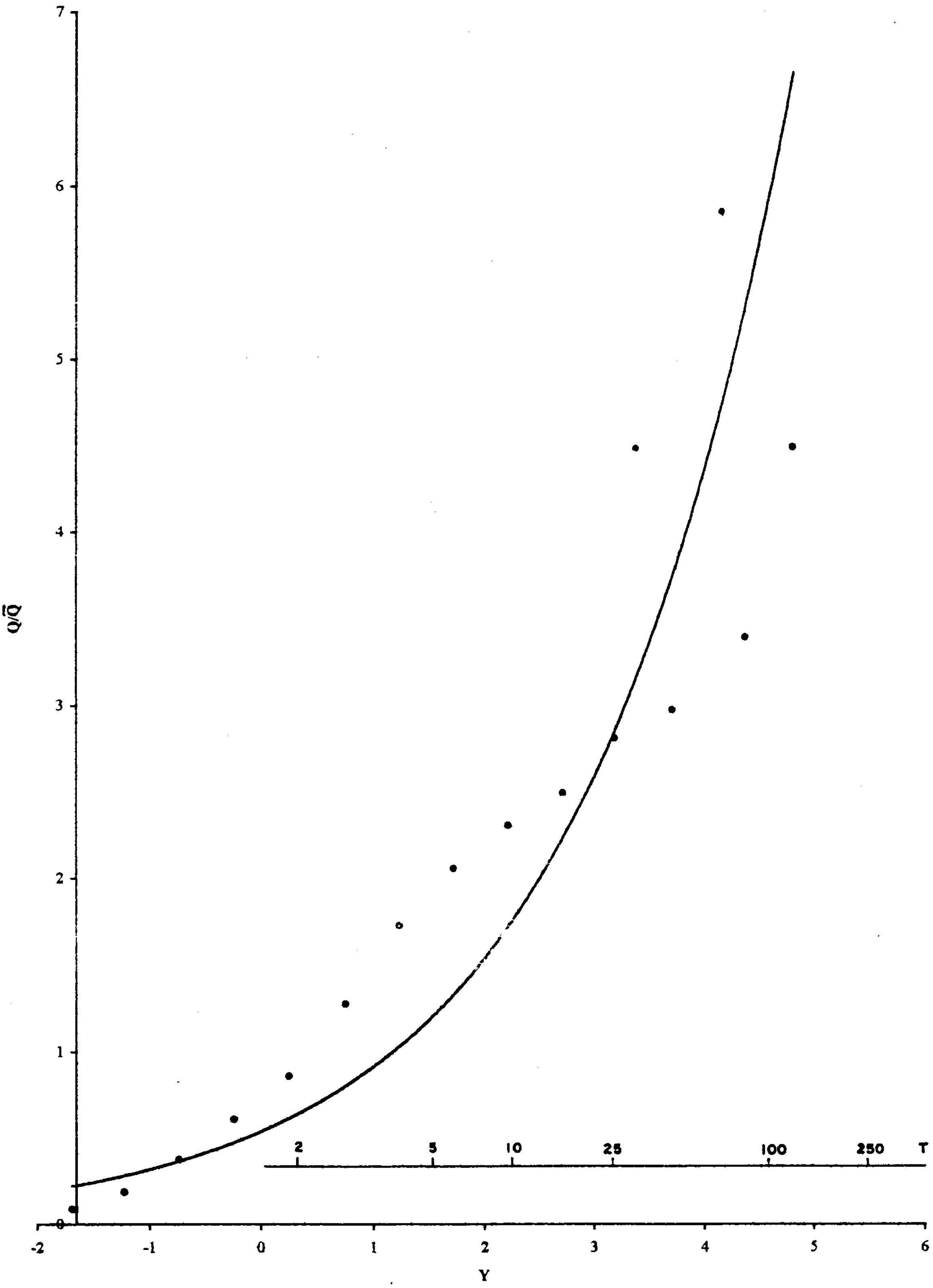


Fig : 28 Regional flood frequency curve - Region 9

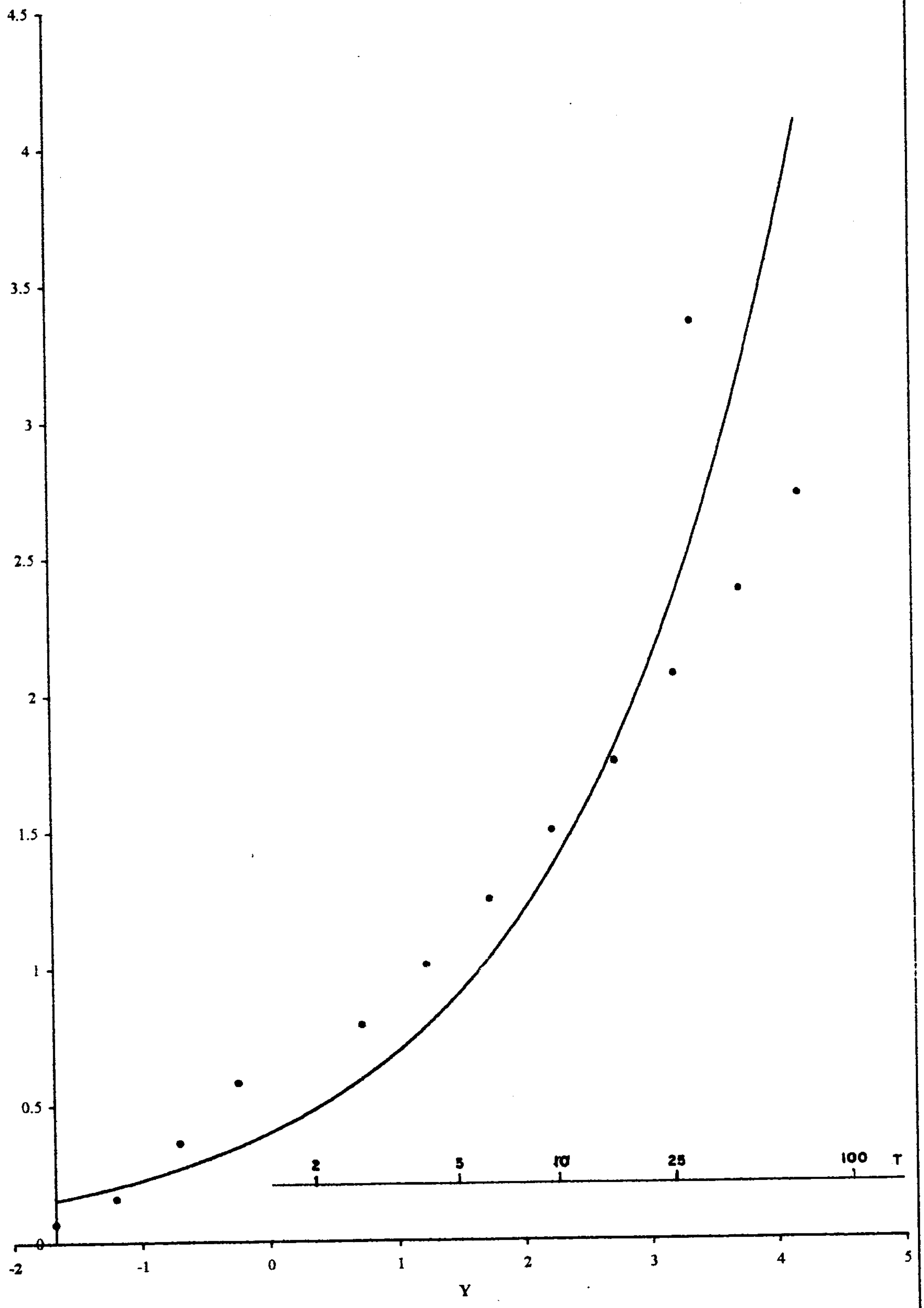


Fig : 29 Regional flood frequency curve - KERALA

4.3 RELATIONSHIP BETWEEN MEAN ANNUAL PEAK FLOW AND CATCHMENT CHARACTERISTICS

Details of peak flow data and catchment characteristics to be considered in this analysis have been described in Chapter 3. The best regression equation has been selected using step-wise regression analysis using SPSS package. The selected regression equation for each region is given in Table 18.

TABLE 18 : REGRESSION EQUATIONS FOR NINE REGIONS

Region	Regression equation	R	R^2
1	$Q = -20.00 + 16.932 * L$	0.9523	0.9068
2	$Q = 87.744 + 1.194 * A$	0.932	0.8686
3	$Q = 140.074 + 1.291 * A - 9.383\ 8 * L$	0.946	0.8949
4	$Q = 94.759 + 0.958 * A$	0.941	0.8854
5	$Q = 61.33 + 0.473 * A$	0.972	0.9447
6	$Q = 164.543 + 0.542 * A$	0.966	0.9331
7	$Q = 158.048 + 0.569 * A$	0.993	0.9860
8	$Q = -175.14 + 0.649 * A$	0.969	0.9389
9	$Q = 29.981 + 5.255 * SL$	0.952	0.906

For entire Kerala, a relationship is formed

$$Q = 212.793 + 0.687 * A - 3.243 * L \qquad [R = 0.9672, \quad R^2 = 0.9354]$$

For getting an idea about the inter-relationship between the catchment characteristics, correlation matrix has been developed for the nine regions and for Kerala as a single region are given in Tables 19 – 28.

TABLE 19: CORRELATION MATRIX FOR DATA OF REGION - 1

	SL	A	SF	L
SL	1.000	0.144	0.010	-0.203
A	0.144	1.000	0.128	-0.869
SF	0.010	0.128	1.000	-0.444
L	-0.203	-0.869	-0.444	1.000

TABLE 20: CORRELATION MATRIX FOR DATA OF REGION - 2

	SL	A	SF	L
SL	1.000	-0.909	-0.206	0.925
A	-0.909	1.000	0.066	-0.979
SF	-0.206	0.066	1.000	0.005
L	0.925	-0.979	0.005	1.000

TABLE 21: CORRELATION MATRIX FOR DATA OF REGION – 3

	SL	L	SF	A
SL	1.000	-0.041	-0.577	0.041
L	-0.041	1.000	-0.310	-0.978
SF	-0.577	-0.310	1.000	0.335
A	-0.130	-0.978	0.335	1.000

TABLE 22: CORRELATION MATRIX FOR DATA OF REGION – 4

	SL	SF	A	L
SL	1.000	-0.451	-0.044	0.259
SF	-0.451	1.000	-0.097	-0.023
A	-0.044	-0.097	1.000	-0.904
L	0.259	-0.023	-0.904	1.000

TABLE 23:CORRELATION MATRIX FOR DATAOF REGION - 5

	SL	L	SF	A
SL	1.000	-0.041	-0.137	0.093
L	-0.041	1.000	-0.195	-0.944
SF	-0.137	-0.195	1.000	0.283
A	0.093	-0.944	0.283	1.000

TABLE 24: CORRELATION MATRIX FOR DATA OF REGION – 6

	SL	SF	L	A
SL	1.000	-0.323	0.131	-0.084
SF	-0.323	1.000	-0.357	0.382
L	0.131	-0.357	1.000	-0.994
A	-0.084	0.382	-0.994	1.000

TABLE 25: CORRELATION MATRIX FOR DATA OF REGION – 7

	SL	SF	A	L
SL	1.000	-0.005	-0.154	0.411
SF	-0.005	1.000	-0.033	0.282
A	-0.154	-0.033	1.000	-0.769
L	0.411	0.282	-0.769	1.000

TABLE 26: CORRELATION MATRIX FOR DATAOF REGION – 8

	SL	A	SF	L
SL	1.000	-0.837	0.803	0.889
A	-0.837	1.000	-0.885	-0.935
SF	0.803	-0.885	1.000	0.790
L	0.889	-0.935	0.790	1.000

TABLE 27: CORRELATION MATRIX FOR DATA OF REGION - 9

	SL	A	SF	L
SL	1.000	-0.044	-0.293	-0.147
A	-0.044	1.000	0.779	-0.832
SF	-0.293	0.779	1.000	-0.646
L	-0.147	-0.832	-0.646	1.000

TABLE 28: CORRELATION MATRIX FOR KERALA

	SL	A	SF	L
SL	1.000	-0.079	-0.342	0.195
A	-0.079	1.000	0.070	-0.933
SF	-0.342	0.070	1.000	-0.119
L	0.195	-0.933	-0.199	1.000

4.4 APPLICATION OF SIMPLE LINEAR MODEL AND LINEAR PERTURBATION MODEL IN SELECTED CATCHMENTS OF KERALA

4.4.1 General

The Simple Linear Model (SLM) and Linear Perturbation Model (LPM), developed in the University of Galway and reported to be suitable for the Malabar Coast (Sreedharan et al, 1995), were applied to two catchments of the Meenachil river basin (Palai and Kidangoor) and two catchments of Achencoil river basin (Konni and Kollakadavu). The main purpose of the study has been to demonstrate the potential of the linear perturbation model in rainfall-discharge modelling, particularly in the monsoon-fed, fast flowing rivers of the south-west coast of India. For purposes of comparison only, the simple linear model has been tested separately on the same data sets for the Meenachil and Achencoil rivers. The application of these models in real-time forecasting mode is not of primary interest, as the emphasis is on the ability of the rainfall-discharge models to provide reasonable forecasts. Specifically, the objective of the present study is to forecast the daily flows at the downstream discharge stations at Palai and Kidangoor in the Meenachil river basin and at Konni and Kollakadavu in the Achenkoil river basin; the sub-basins considered are given in Figs. 30 and 31.

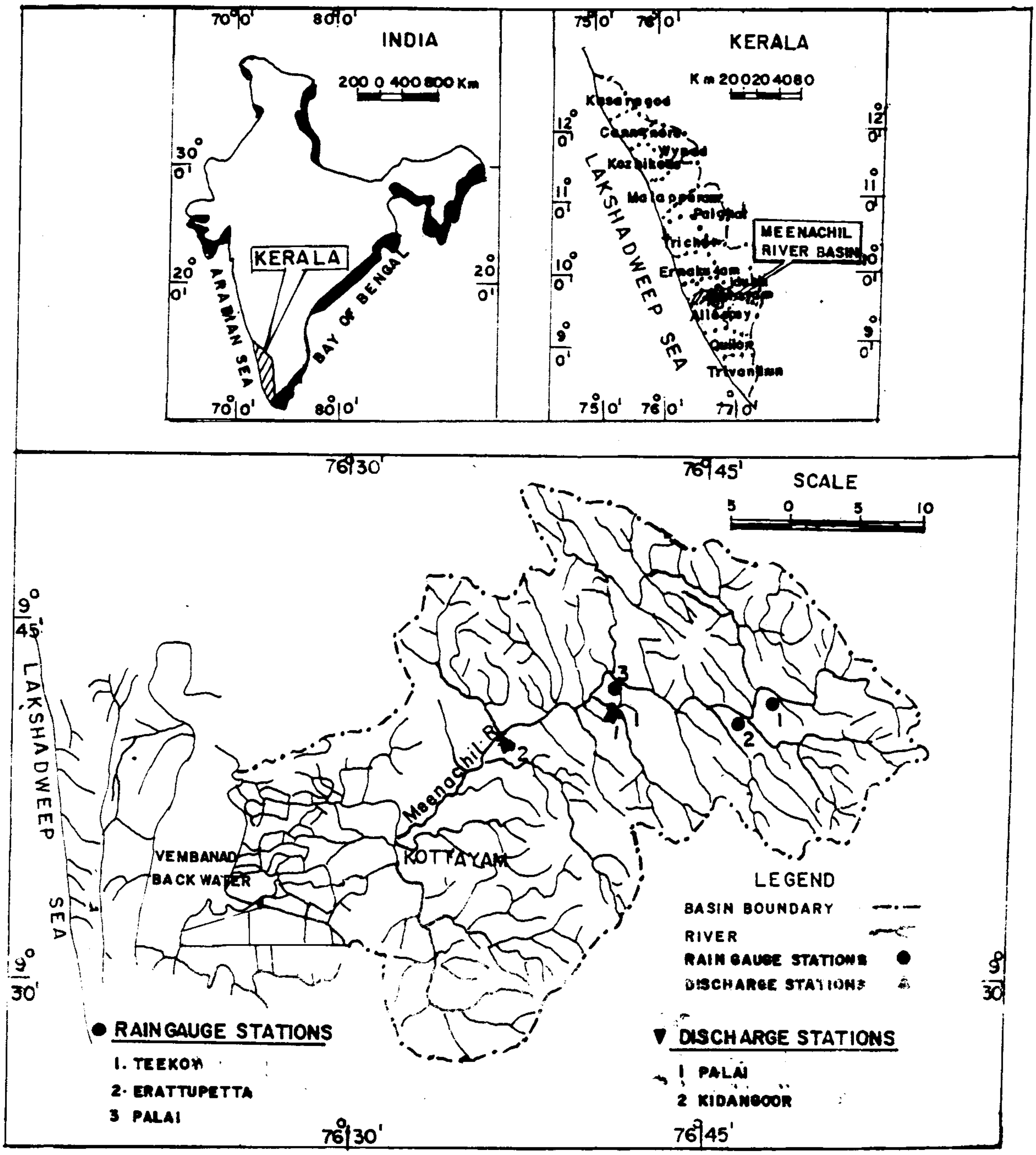


FIG 30 MEENACHIL RIVER BASIN

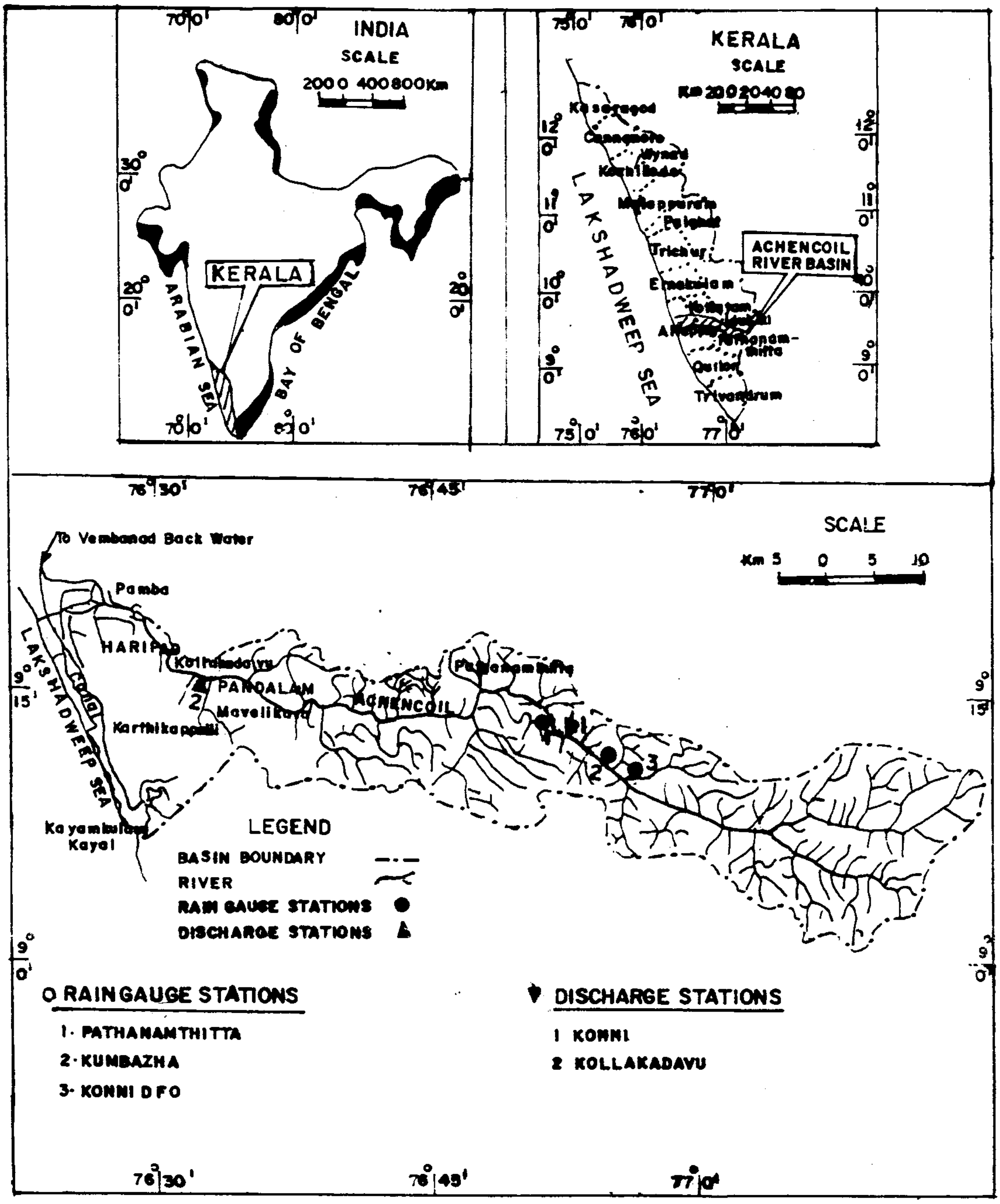


FIG 31 ACHENCOIL RIVER BASIN

4.4.2 Study Area

The Meenachil river has a length of 78 km and a total drainage area of 1272 sq.km. Fig.30. shows the basin map of Meenachil river along with the location of rainfall and discharge stations.

The Achencoil river has a length of 128 km and a total drainage area of 1484 sq.km. Fig.31. shows the basin map of Achencoil river along with the locations of rainfall and discharge stations.

4.4.3 Data Base

The daily rainfall and streamflow data for 10 years (1981-90) for Palai, Kidangoor, Konni and Kollakadvu were collected for the present study from the data records of Kerala PWD. The rainfall data from Erattupetta, Teekoy and Palai were used for calculating mean areal rainfall for the Kidangoor sub-basin and the rainfall data from Palai were used for the Palai sub-basin. The rainfall data from Konni D.F.O were used for the Konni sub-basin in the Achencoil basin and the rainfall data from Pathanamthitta, Kumbazha and Konni D.F.O were used for finding the mean annual rainfall for the Kollakadavu sub-basin. Eight years of data were used for calibration and remaining two years for verification. The drainage area and the length of the main stream of the sub-basins considered are given in Table 29.

TABLE 29: DRAINAGE AREA, MAIN STREAM LENGTH OF THE STUDY AREA

River basin	Sub-basin	Catchment area(sq.km)	Length of mainstream
Meenachil	Palai	0429.79	41.50
	Kidangoor	615 .00	63.25
Achencoil	Konni	0419.00	52.75
	Kollakadavu	0952.71	103.28

4.4.4 Application of SLM

This model presumes a simple linear relationship between the input- the areal mean rainfall on the catchment [x(t)], and the output- the discharge recorded at the gauging station [y(t)]. In discrete form, it is expressed by equation (89) (Chapter 3).

This model was calibrated on daily data, first by ordinary least squares and later under constraints imposed by the gamma function form of the impulse response. The memory length was chosen by trial and error.

The simple linear model under the constraint of a gamma function impulse response was also applied to the four stations. The search was continued to find the values of parameters n, nK and G_g which minimised F. To obtain reasonable starting values of the parameters of the model, the information already obtained in the unconstrained least squares optimisation was used. The parameter n and K were varied successively and jointly to obtain as close a reproduction of the observed pulse response as possible. Using these n and nK as initial estimates, and retaining the gain factor as indicated in the unconstrained analysis, a search was conducted in the n, nK plane to minimise the sums of squares of differences between the computed and observed discharges. After obtaining optimum values for n and nK, these were retained and Gg was optimised. The Rosenbrock (1960) search method, explained for the parameter, are given in Table 30 and the results of the simple linear model obtained with non-parametric form and under the constraints of the gamma function impulse response are given in Tables 31- 34.

TABLE 30: ESTIMATED PARAMETERS OF THE SLM UNDER THE CONSTRAINT OF THE GAMMA FUNCTION IMPULSE RESPONSE

Sub-basin	Memory length (days)	Estimated gamma function model parameters		
		n	nK(days)	Gain factor(G_g)
Palai	07	1.42	3.39	0.2866
Kidangoor	15	0.60	7.03	0.3914
Konni	09	1.29	2.83	0.2713
Kollakadavu	20	1.74	4.35	0.6824

TABLE 31: SUMMARY OF THE RESULTS OBTAINED WITH THE SLM – PALAI SUB-BASIN

Model type	Memory length (days)	Initial variance per unit time		Residual variance per unit time		Model efficiency R^2(%)	
		Calibration	Verification	Calibration	Verifiication	Calibration	Verification
Unconstrained	07	153.53	182.60	58.02	66.21	43.20	34.24
Gamma function	07	153.53	182.60	63.50	76.34	37.22	33.12

Table 32: SUMMARY OF THE RESULTS OBTAINED WITH THE SLM – KIDANGOOR SUB-BASIN

Model type	Memory length (days)	Initial variance per unit time		Residual variance per unit time		Model efficiency, R^2(%)	
		Calibration	Verifcation	Calibration	Verifiication	Calibration	Verification
Unconstrained	15	124.60	132.80	42.18	51.84	62.13	58.17
Gamma function	15	124.60	132.80	48.23	54.28	58.45	55.34

TABLE 33: SUMMARY OF THE RESULTS OBTAINED WITH THE SLM – KONNI SUB-BASIN

Model type	Memory length (days)	Initial variance per unit time		Residual variance per unit time		Model efficiency, R^2(%)	
		Calibration	verification	Calibration	Verification	Calibration	Verification
Unconstrained	09	160.13	198.23	80.20	92.34	38.37	26.88
Gamma function	09	160.13	198.23	84.36	98.25	35.30	25.32

TABLE 34: SUMMARY OF THE RESULTS OBTAINED WITH THE SLM – KOLLAKADAVU SUB-BASIN

Model type	Memory length (days)	Initial variance per unit time		Residual variance per unit time		Model efficiency, R^2(%)	
		Calibration	Verification	Calibration	Verification	Calibration	Verification
Unconstrained	20	116.28	122.34	33.02	44.34	66.32	60.82
Gamma function	20	116.28	122.34	36.34	46.84	63.02	54.37

In the sub-basins of Palai in the Meenachil basin and Konni in the Achencoil basin, the model efficiencies are very poor. In the other two sub-basins, satisfactory results have been obtained. The results indicate that only in exceptional cases the simple linear model serves as a viable forecasting tool and linear model alone is not a viable method of predicting runoff. Figs. 32 - 35 show the standardised pulse response of the simple linear model derived by ordinary least squares. Figs. 36-39 show the comparison of the non-parametric linear model derived by the method of ordinary least squares with those obtained under the constraint of the gamma function impulse response estimated by Rosenbrock's direct search algorithm. From the results of the linear model under the constraints of the gamma function impulse response, they are not significantly poorer than those obtained without constraints. This implies that the shape of the pulse response is of little significance or that the gamma function shape is sufficiently general to represent actually occurring shapes.

4.4.5 Application of LPM

This model assumes that x, the perturbations or departures from the smoothed seasonal input values (i_d) are linearly related to y, the corresponding perturbation or departures from the seasonal output values(q_d). It is expressed by equation (98) (Chapter 3).

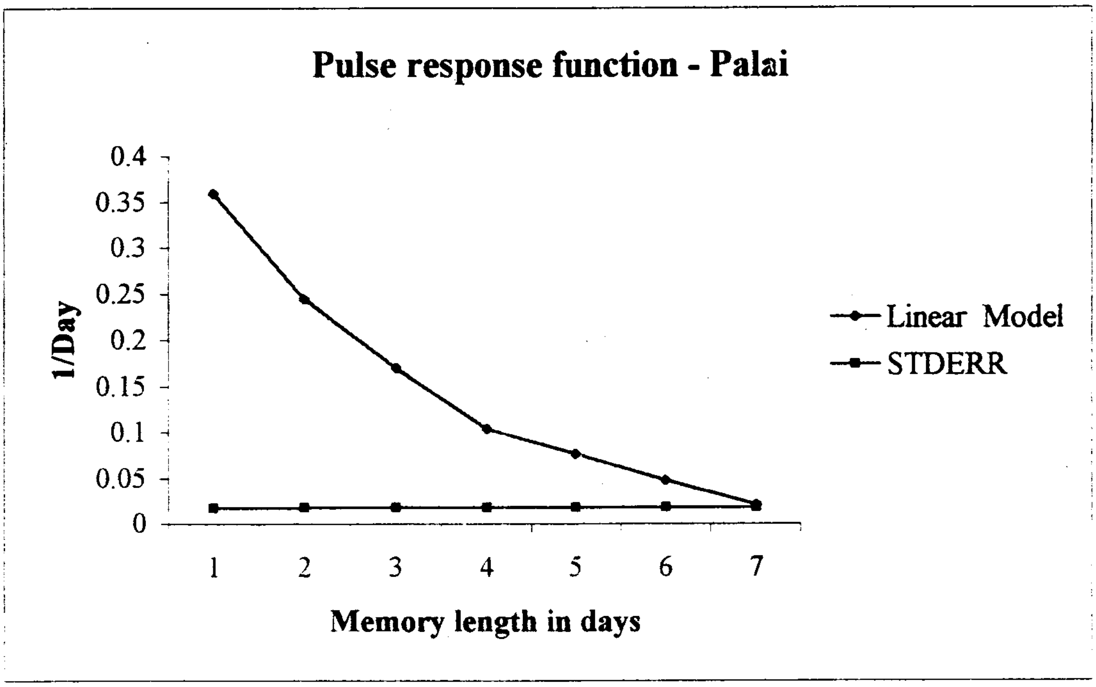


Fig . 32 Pulse responses of the simple linear model using ordinary least squares for Palai

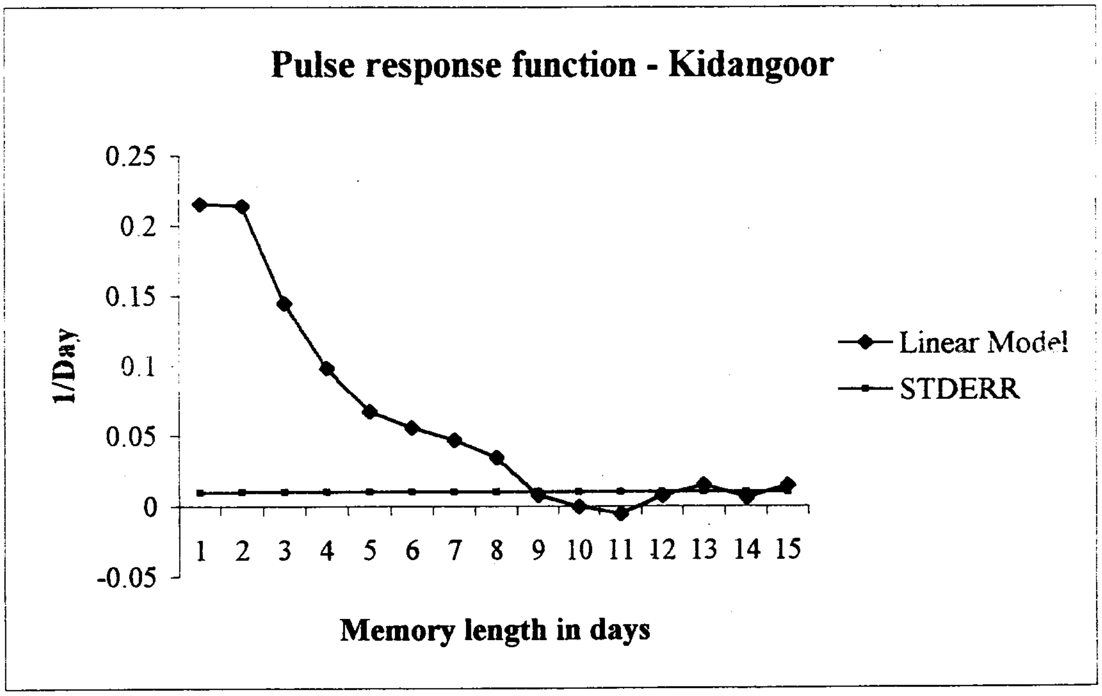


Fig . 33 Pulse responses of the simple linear model using ordinary least squares for Kidangoor

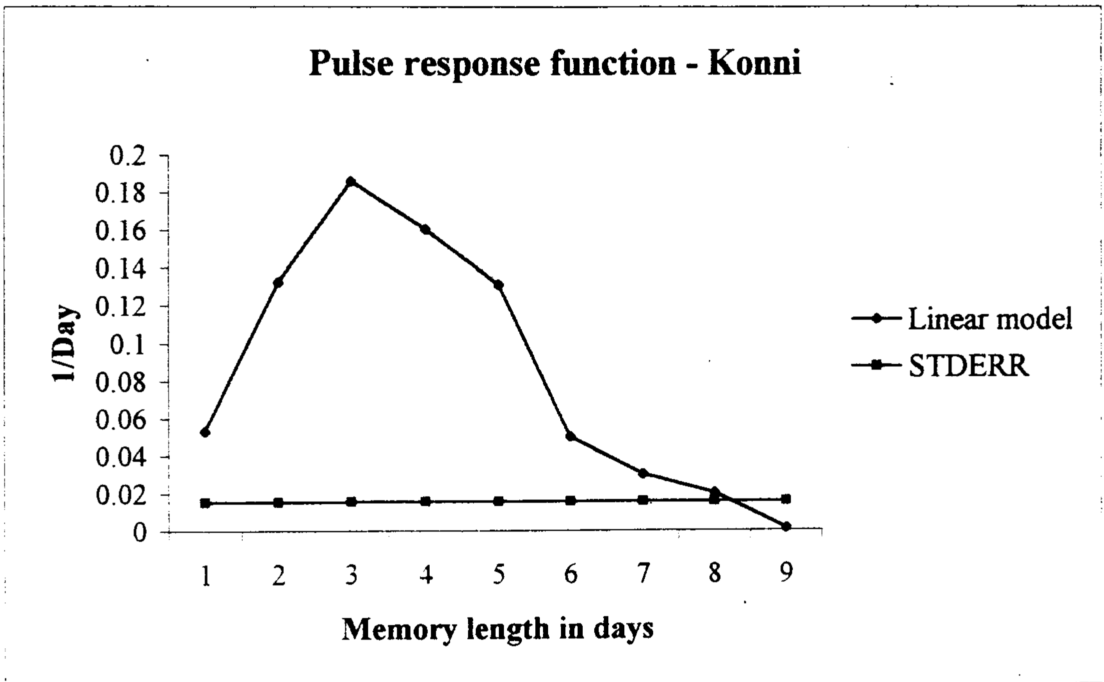


Fig . **34** Pulse responses of the simple linear model using ordinary least squares for Konni

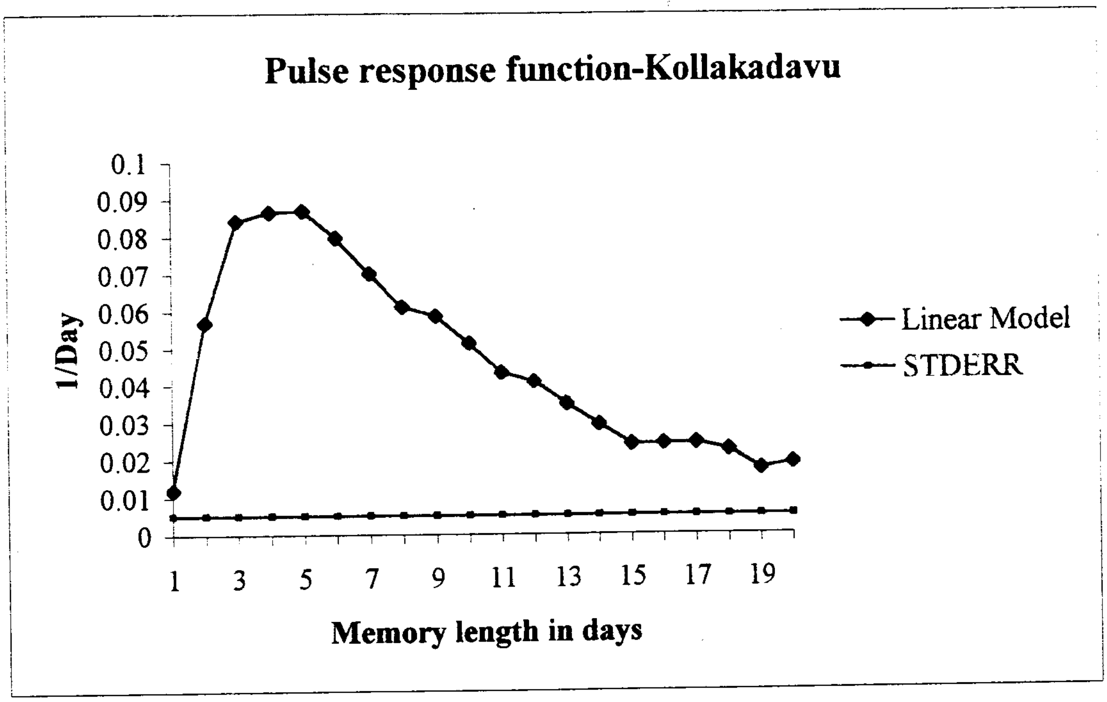


Fig. **35** Pulse responses of the simple linear model using ordinary least squares for Kollakadavu

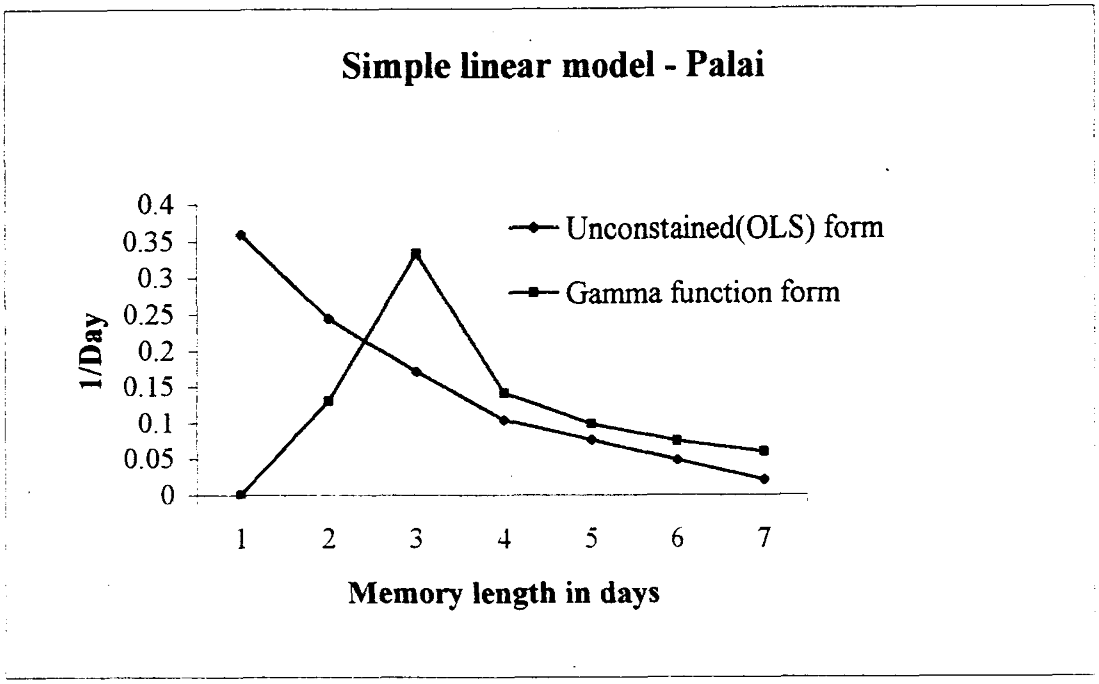


Fig . 36 Comparison of the pulse responses of the non-parametric simple linear model derived by the method of ordinary least squares with those obtained under the constraint of the gamma function impulse response, estimated by Rosenbrock's direct search algorithm

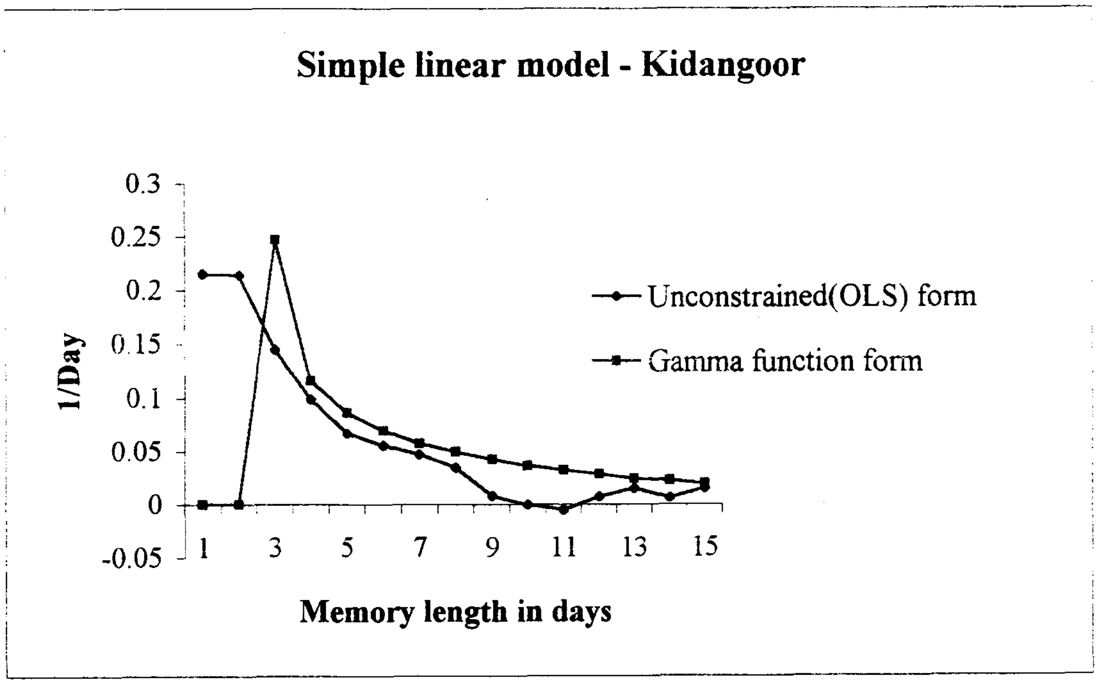


Fig . 37 Comparison of the pulse responses of the non-parametric simple linear model derived by the method of ordinary least squares with those obtained under the constraint of the gamma function impulse response, estimated by Rosenbrock's direct search algorithm

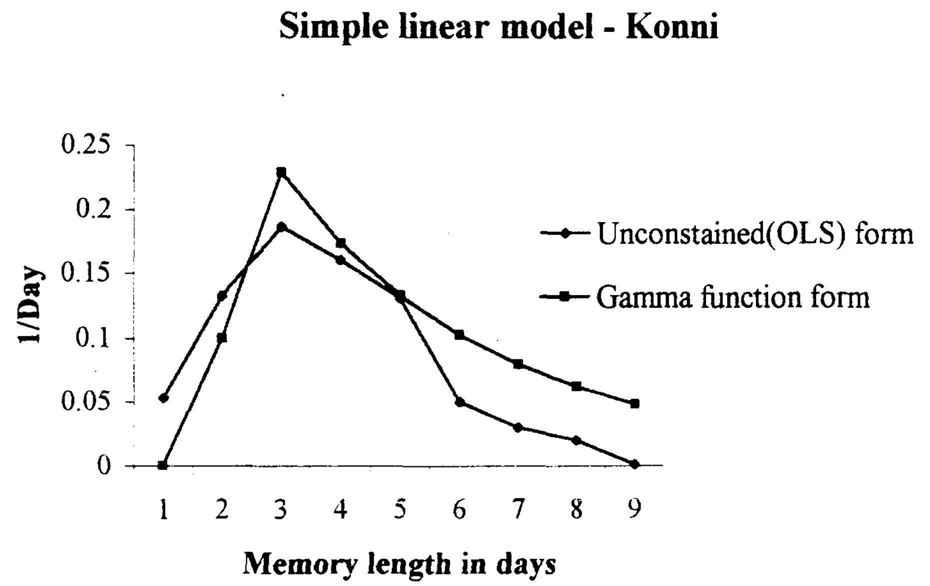


Fig . 38 Comparison of the pulse responses of the non-parametric simple linear model derived by the method of ordinary least squares with those obtained under the constraint of the gamma function impulse response, estimated by Rosenbrock's direct search algorithm

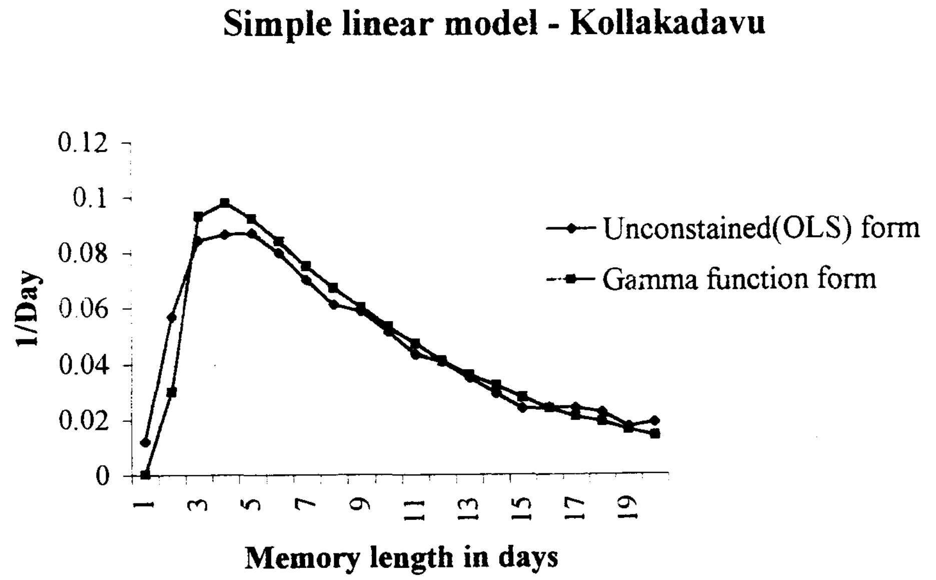


Fig . 39 Comparison of the pulse responses of the non-parametric simple linear model derived by the method of ordinary least squares with those obtained under the constraint of the gamma function impulse response, estimated by Rosenbrock's direct search algorithm

This model was applied in the following manner to all the sub-basins under investigation:

(i) Seasonal mean rainfalls and seasonal mean discharges were calculated for the periods of calibration (smoothing done by the method of unconstrained Fourier analysis using the four harmonics);

(ii) The smoothed seasonal mean values were subtracted from the observed rainfall and discharge series for the periods of calibration to yield the time series of perturbation, x and y;

(iii) The pulse response function was estimated, initially by the method of ordinary least squares, and then by a search technique under the constraints of the gamma function impulse response;

(iv) The pulse responses were convoluted with the rainfall perturbation to obtain the estimated outflow perturbation series;

(v) The estimated discharge series was calculated by adding the seasonal mean discharge to the estimated outflow perturbation series; and

(vi) The difference between observed and computed discharges were squared and summed and the usual measure of efficiency (R^2) calculated.

This model was applied in the non-parametric form and under the constraints of the gamma function form for all sub-basins. The estimated parameters are given in the parametric form in Table 35.

TABLE 35: ESTIMATED PARAMETERS OF THE LINEAR PERTURBATION MODEL UNDER THE CONSTRAINTS OF THE GAMMA FUNCTION IMPULSE RESPONSE

Sub-basin	Memory length(days)	Estimated gamma function model parameters		
		n	nK	Gainfactor (G_g)
Palai	07	1.42	03 .39	0 .3466
Kidangoor	15	0.49	11. 59	0. 4929
Konni	09	1.29	02 .83	0 .3213
Kollakadavu	20	1.74	04. 35	0 .8214

The results of the LPM in non-parametric form and under the constraints of the gamma function form are given in Tables 36- 39.

TABLE 36: SUMMARY OF THE RESULTS OBTAINED WITH LPM – PALAI SUB-BASIN

Model type	Memory (days)	Initial variance length per unit time		Residual variance per unit time		Model efficiency, R^2(%)	
		Calib-ration	Verifi-cation	Calib-ration	Verifi-cation	Calib-ration	Verifi-cation
Unconstrained	07	153.53	182.60	54.02	59.13	48.20	43.23
Gamma function	07	153.53	182.60	57.03	66.21	41.22	38.12

TABLE 37: SUMMARY OF RESULTS OBTAINED WITH LPM – KIDANAGOOR SUB-BASIN

Model type	Memory length (days)	Initial variance per unit time		Residual variance per unit time		Model efficiency R^2(%)	
		Calib-ration	Verifi-cation	Calib-ration	Verifi-cation	Calib-ration	Verifi-cation
Unconstrained	15	124.60	132.80	28.45	39.82	80.60	76.12
Gamma function	15	124.60	132.80	30.84	43.28	78.21	73.24

TABLE 38: SUMMARY OF THE RESULTS OBTAINED WITH LPM – KONNI SUB-BASIN

Model type	Memory length (days)	Initial variance per unit time		Residual variance per unit time		Model efficiency, R^2(%)	
		Calib-ration	Verifi-cation	Calib-ration	Verifi-cation	Calib-ration	Verifi-cation
Unconstrained	09	160.13	198.23	72.54	84.23	43.56	32.12
Gamma function	09	160.13	198.23	76.23	92.32	38.30	29.36

TABLE 39: SUMMARY OF THE RESULTS OBTAINED WITH LPM – KOLLAKADAVU SUB-BASIN

Model type	Memory length (days)	Initial variance per unit time		Residual variance per unit time		Model efficiency, R^2(%)	
		Calibration	Verification	Calibration	Verification	Calibration	Verification
Unconstrained	20	116.28	122.34	26.02	38.32	81.69	78.43
Gamma function	20	116.28	122.34	28.84	42.54	77.28	73.42

Figs. 40 - 43 give a plot of standardised pulse response of the linear perturbation model by ordinary least squares. For the sub-basins of Kollakadavu and Kidangoor, the model efficiencies are 81.69 and 80.60 percent respectively for the calibration period and 78.43 and 76.12 percent respectively for the verification period for non-parametric form. For the sub-basins of Konni and Palai, the efficiencies are generally poor. Figs. 44 - 47 show the comparison of the non-parametric LPM derived by the method of ordinary least squares and those obtained under the constraint of the gamma function. From the results, the LPM under the constraints of the gamma function impulse response is not significantly different from those obtained without constraints, thus further indicating that the gamma function shape is adequately flexible.

When compared with the results of the SLM, the LPM is significantly better in the sub-basins of Kollakadavu and Kidangoor. In these sub-basins, the seasonal component of the linear perturbation model accounts for a large percentage of the initial variance. For the remaining sub-basins, there is virtually no difference between the results of the SLM and those of the LPM; results from both are poor.

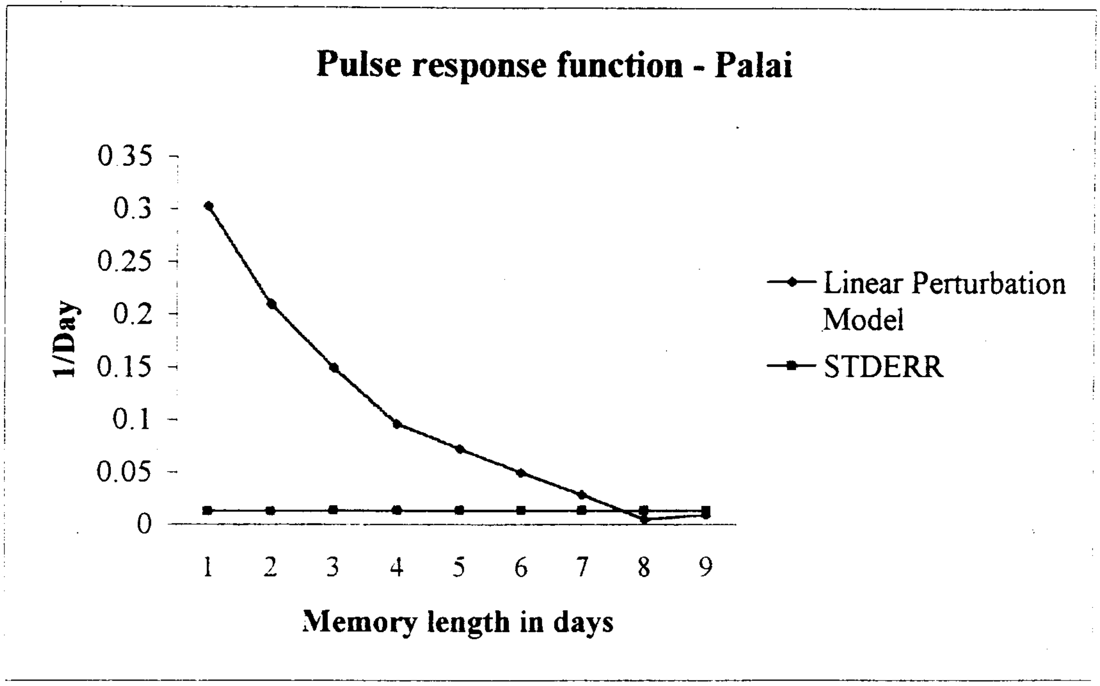


Fig. 40 Pulse responses of the linear perturbation model derived by ordinary least. squares for Palai

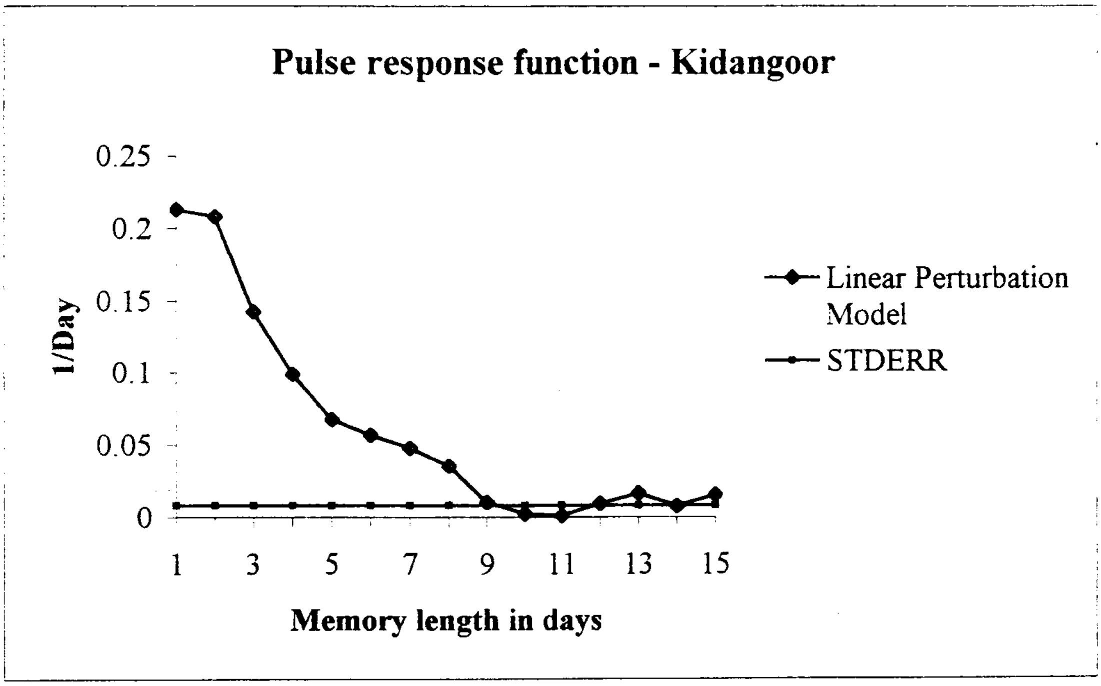


Fig. 41 Pulse responses of the linear perturbation model derived by ordinary least. squares for Kidangoor

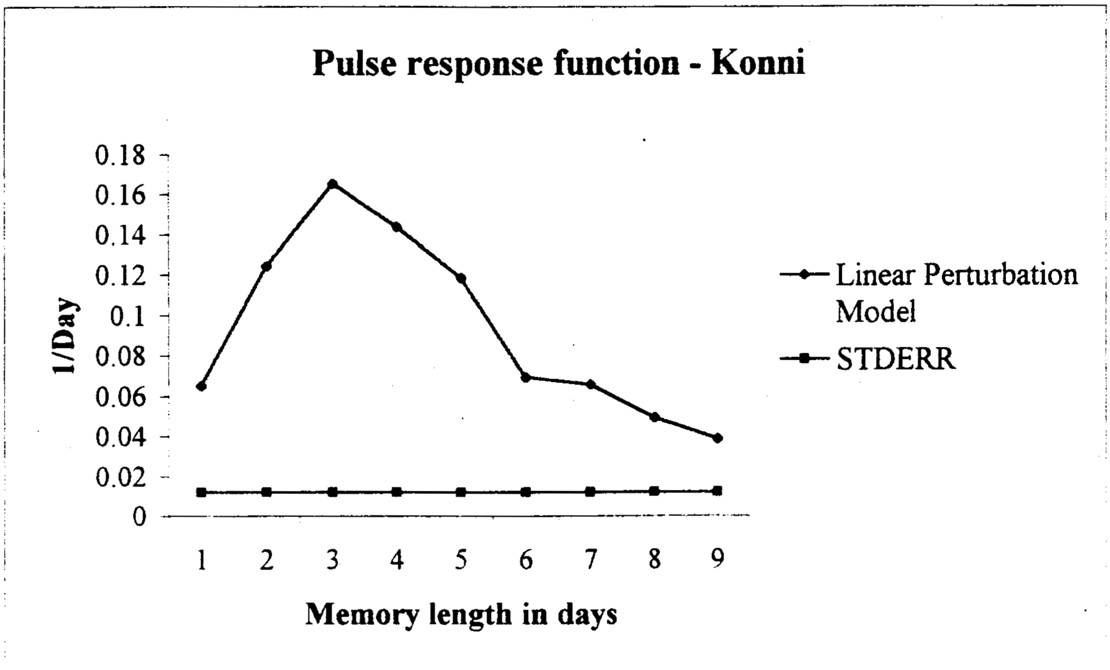


Fig . **42** Pulse responses of the linear perturbation model derived by ordinary least. squares for Konni

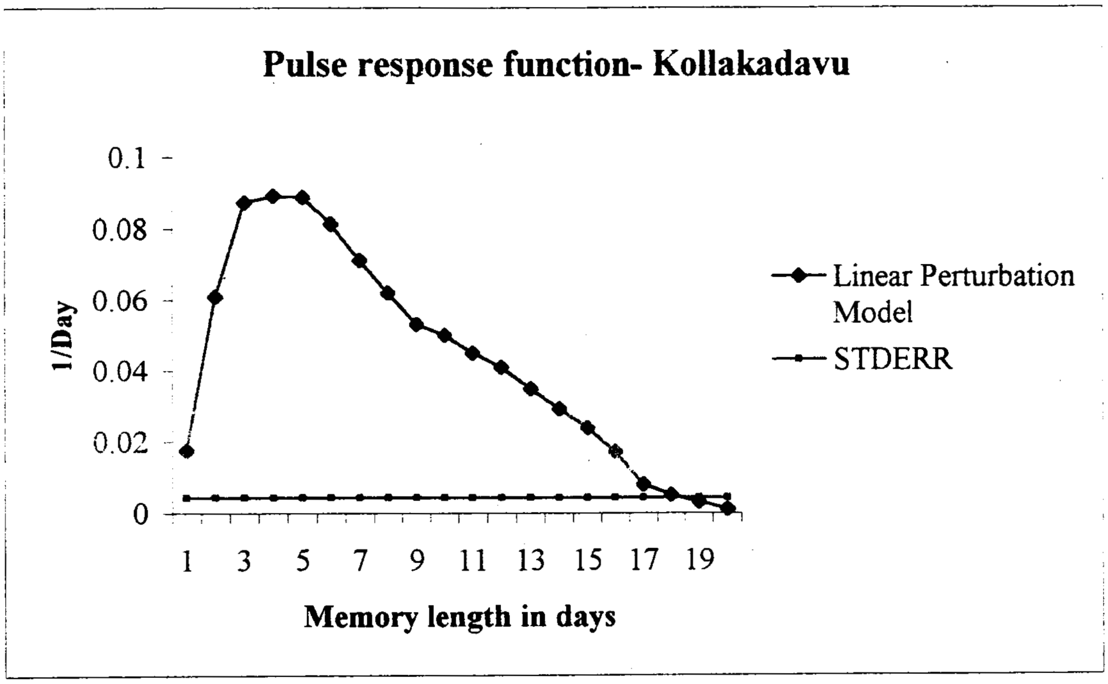


Fig. **43** Pulse responses of the linear perturbation model derived by ordinary least. squares for Kollakadavu

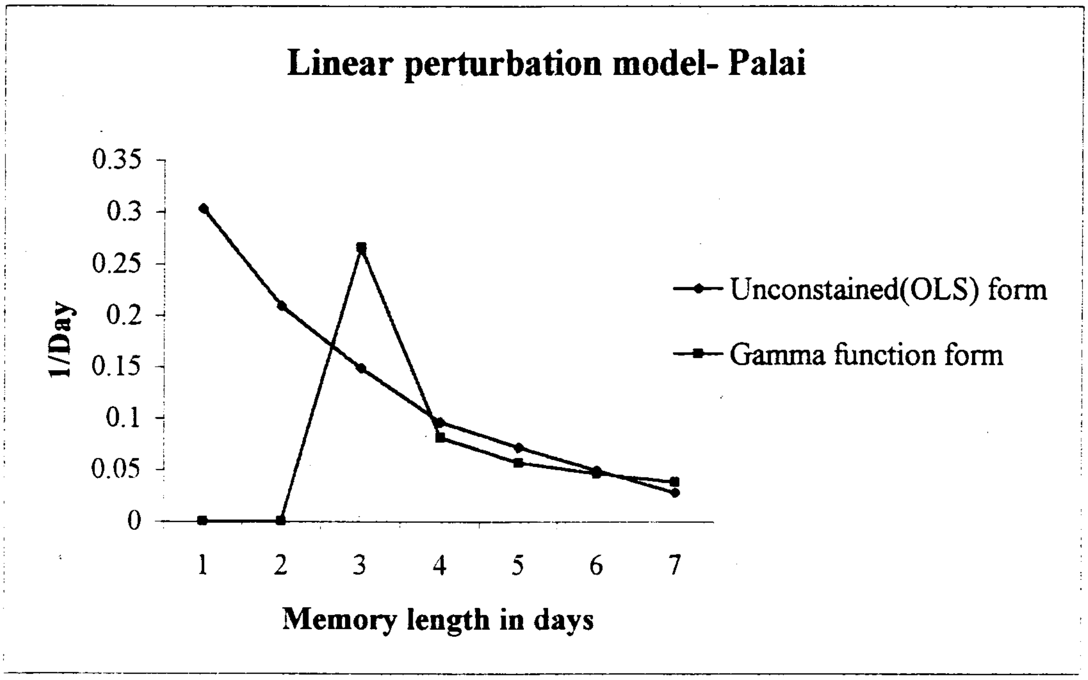


Fig . 44 Comparison of the pulse responses of the non-parametric linear perturbation model derived by the method of ordinary least squares with those obtained under the constraint of the gamma function impulse response, estimated by Rosenbrock's direct search algorithm

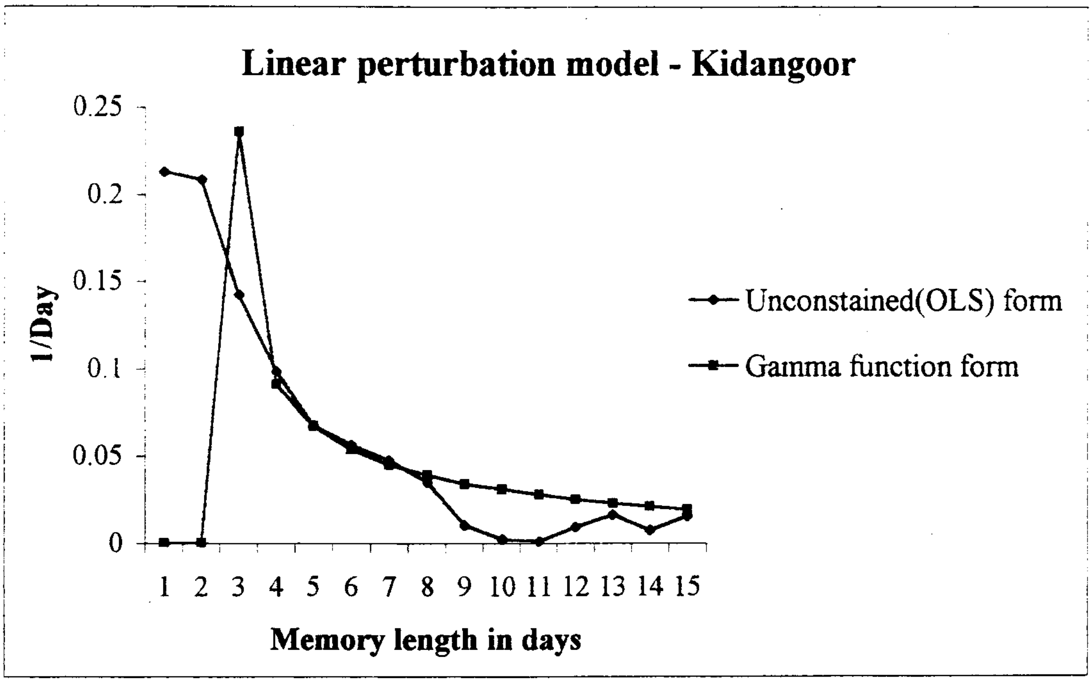


Fig . 45 Comparison of the pulse responses of the non-parametric linear perturbation model derived by the method of ordinary least squares with those obtained under the constraint of the gamma function impulse response, estimated by Rosenbrock's direct search algorithm

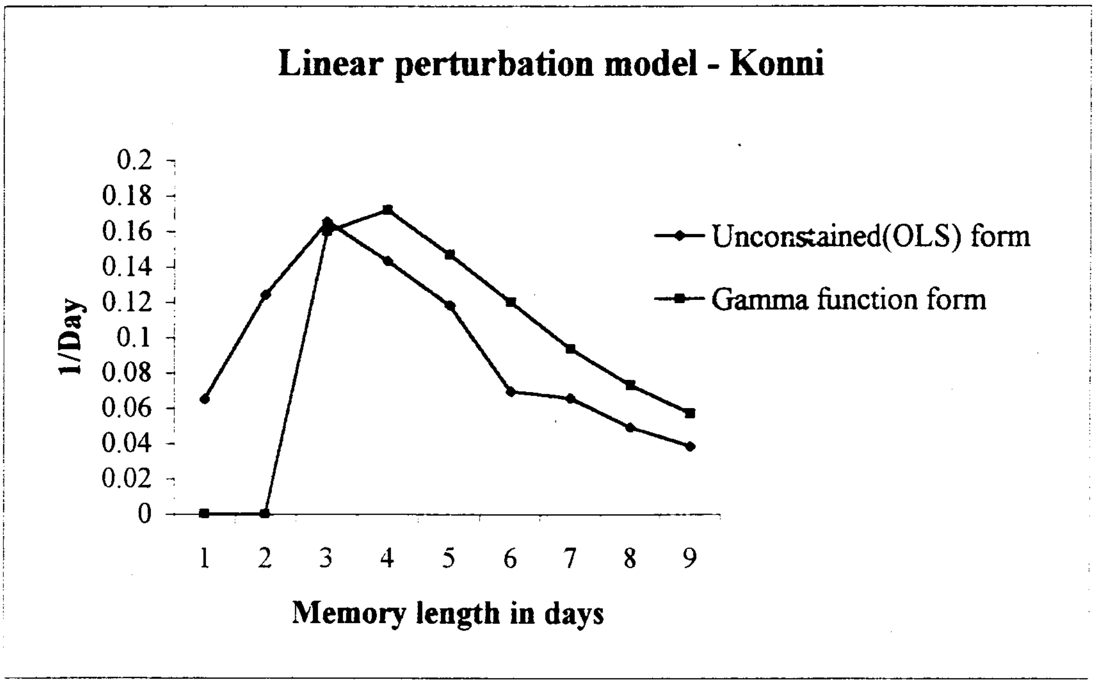


Fig. 46 Comparison of the pulse response s of the non-parametric linear perturbation model derived by the method of ordinary least squares with those obtained under the constraint of the gamma function impulse response. estimated by Rosenbrock's direct search algorithm

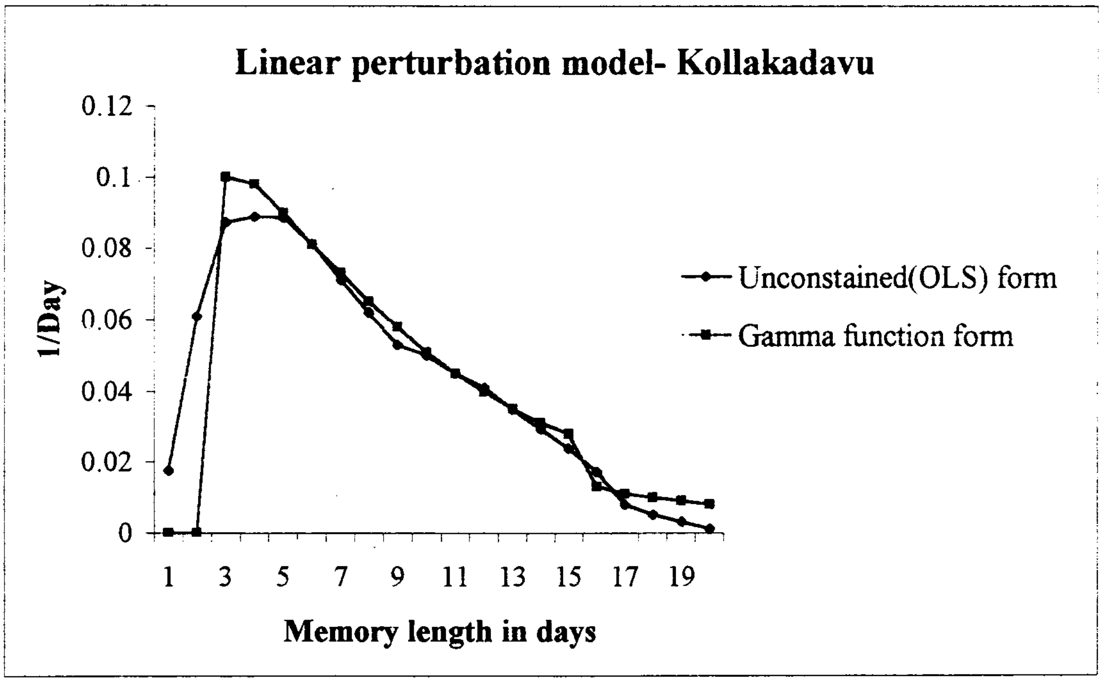


Fig. 47 Comparison of the pulse responses of the non-parametric linear perturbation model derived by the method of ordinary least squares with those obtained under the constraint of the gamma function impulse response, estimated by Rosenbrock's direct search algorithm

Figs. 48 – 51 show pulse responses estimated by the method of least squares, for the unconstrained LPM, superimposed on those of the unconstrained SLM, for the sub-basins of Kidangoor and Kollakadav. The pulse responses for the LPM are more acceptable in shape, indicating that the problem of instability of estimation is less in this case.

4.5 RESULTS AND DISCUSSIONS

The regional flood frequency analysis was attempted in the present study with an aim to predict the flood estimation for different return periods, mainly for locations where actual gauge data are not available.

Both analytical and graphical approaches recommend for flood frequency studies have been used in the present investigation. In the analytical approach, four of the widely used distributions for flood studies, namely Gumbel, log-normal, Pearson type III and log-Pearson type III were used. Since Gumbel distribution has been often applied in graphical approach for flood prediction, this distribution is adopted in the graphical approach followed in the present study.

The regional approach is found to be appropriate for areas with less depth of streamflow data. The flood regions were identified considering certain features of the regions, which are important from the point of view of peak flows (described in Methodology). Nine such homogeneous regions were identified and separate curves evolved for each of them. Keeping in view of certain practical advantages, a single curve was developed for the entire State.

In order to estimate the mean annual peak flow in a particular site, an attempt was made to correlate the mean of the annual past peak flows with catchment characteristics, which may have predominant impact on the peak flows. Such mean annual peak value obtained can be made use for predicting the flood estimates for different return periods. Whenever actual gauge data are not available, the mean

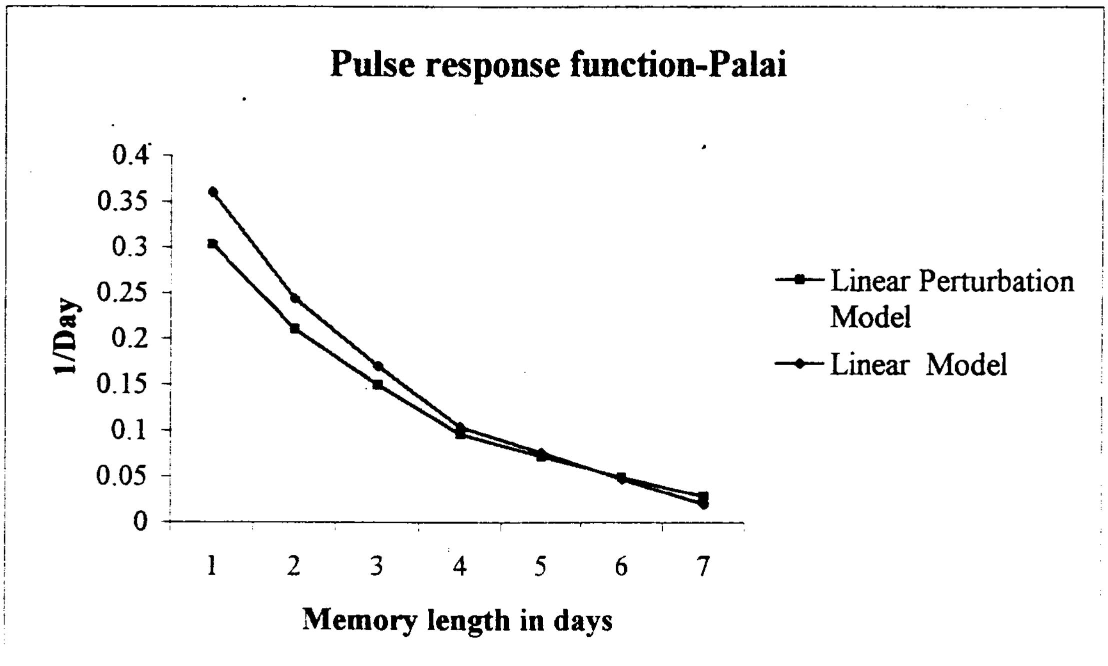


Fig . 48 Pulse responses derived by the ordinary least squares for the linear perturbation model and those obtained for the simple linear model - Palai

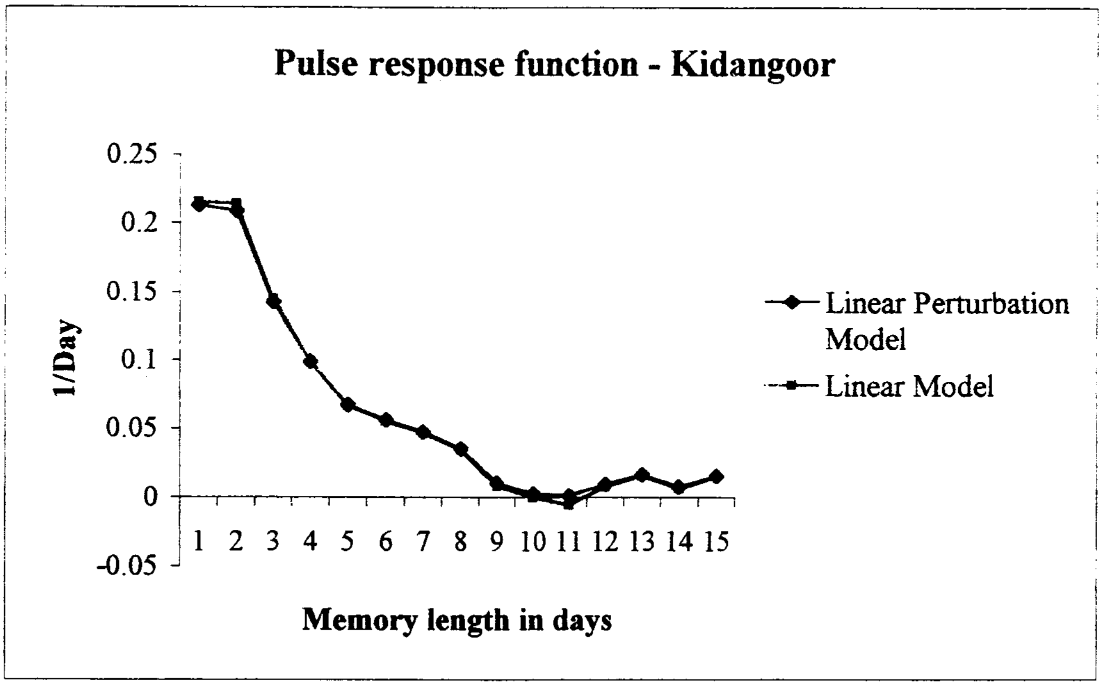


Fig. 49 Pulse responses derived by the ordinary least squares for the linear perturbation model and those obtained for the simple linear model - Kidangoor

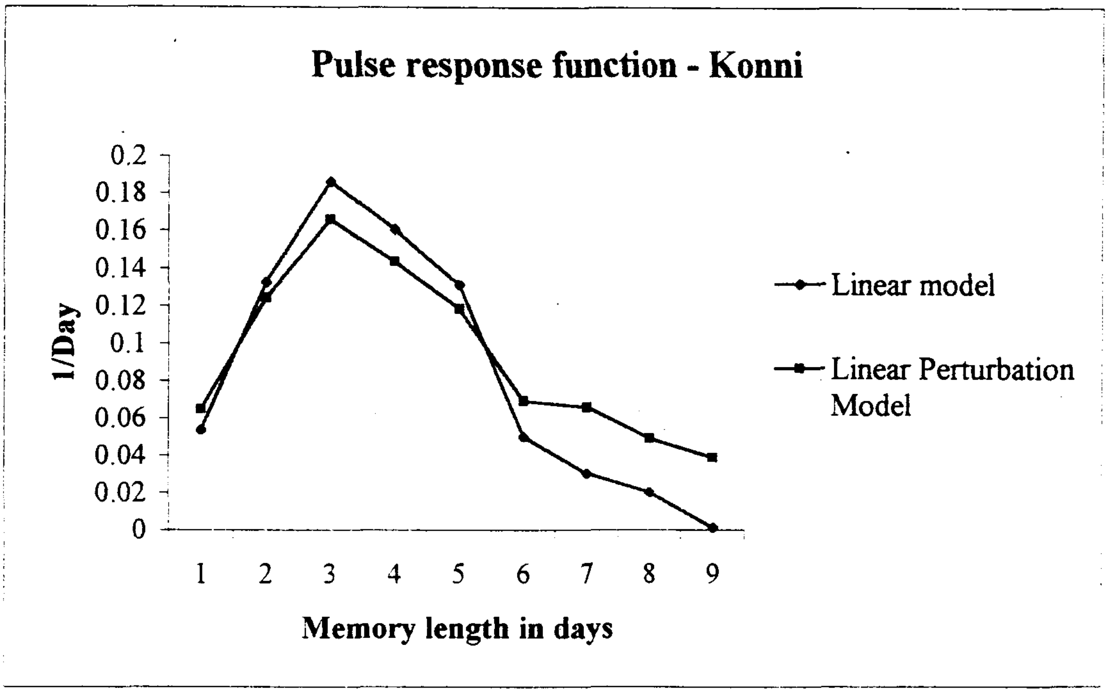


Fig . 50 Pulse responses derived by the ordinary least squares for the linear perturbation model and those obtained for the simple linear model - Konni

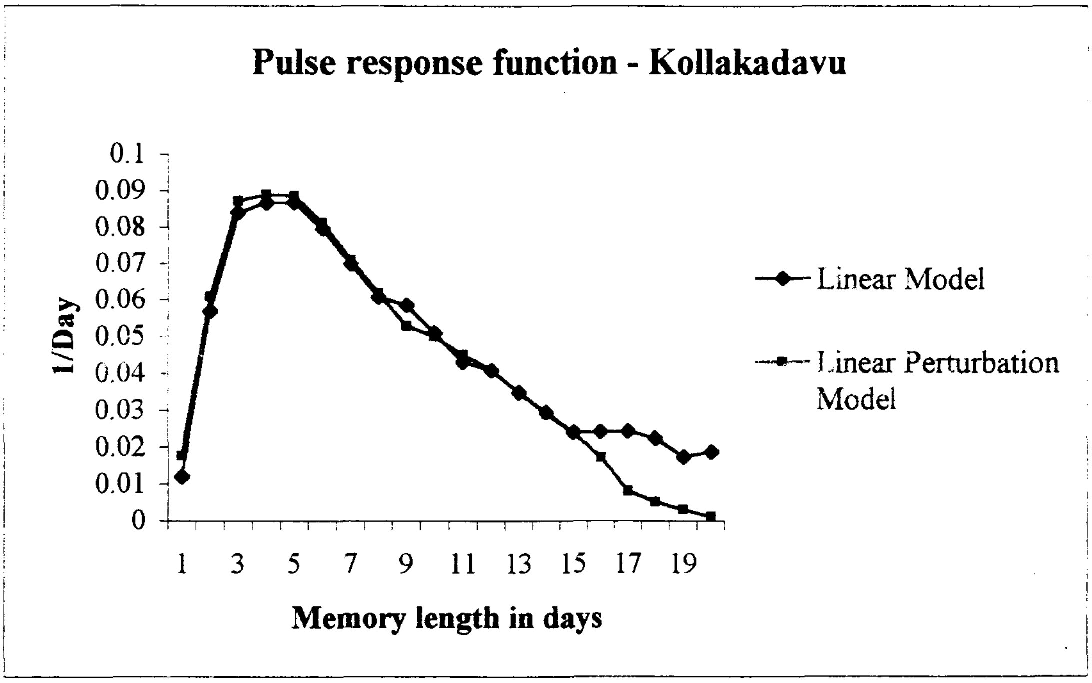


Fig . 51 Pulse responses derived by the ordinary least squares for the linear perturbation model and those obtained for the simple linear model - Kollakadvu annual peak flows can be estimated if the geomorphological parameters of the basin are known. The details are often available from the toposheets (Maps) produced by the Survey of India.

The simple linear model and linear perturbation model were attempted in four sub-basins: Palai and Kidangoor of Meenachil basin and Konni and Kollakadavu of Achencoil basins. From the results, it can be inferred that the simple linear model alone is not a viable model to predict runoff from the rainfall data. The linear perturbation model is significantly better on the catchments of Kidangoor and Kollakadavu. For the remaining catchments, there is no significant difference between the results of the simple linear model and linear perturbation model; both yielded poor results. Comparing the linear perturbation model and simple linear model (Figs. 48-51),the pulse responses of the linear perturbation model are more acceptable in shape, indicating that the problem of instability of estimation is less severe in this model.

The linear perturbation model under the constraints of gamma function impulse response shape was applied to all catchments under investigation. It is not significantly different from those obtained without constraints, indicating that the gamma function shape is adequately flexible.

The simple linear model is not a viable model to predict the runoff from the rainfall data in these basins. From the results, it is seen that linear perturbation model is consistently better in terms of efficiency (R^2).

In the case of Kollakadavu, shapes of the curves of the two models are the same, but efficiencies are different. The results of the present study show the potential of the Linear Perturbation Model as a tool for runoff prediction on seasonal catchments.

SUMMARY AND CONCLUSIONS

Soney John "Statistical analysis of peak flows with special reference to Kerala river basins" Thesis. Department of Statistics, University of Calicut, 1999

5. SUMMARY AND CONCLUSIONS

As a part of the present research programme, all available data on streamflow were collected, compiled, processed and made use of for the regional flood frequency study. The GTS maps of the Survey of India were used in estimating the geomorphic parameters, to be correlated with mean annual peak flow for predictive purposes. The rainfall and runoff data of Palai and Kidangoor sub-basins of the Meenachil river basin and Konni and Kollakadvu sub-basins of Achencoil river basin were utilised for the rainfall-runoff studies reported in this thesis. The Data Bank, computational and cartographic facilities of the Centre for Water Resources Development and Management, Kozhikode (CWRDM), were extensively used for the study.

The major features of this study are summarised below:

1. Regional flood frequency study was carried out for different flood frequency regions of Kerala State using an average of twenty station-year data from 98 stations.

2. In order to estimate the return periods of floods of different magnitudes, four of the distributions frequently used for such studies, namely Gumbel, log normal, Pearson type III and log-Pearson type III, were made use of. Depending on the goodness of fit test, a particular distribution was identified for each of the nine flood frequency regions.

3. In order to serve the practical purposes of the engineers and the professionals working in the field, the graphical approach was followed in estimating the return periods using Gumbel distribution. Such curves were developed not only for each flood frequency region but also for the entire State, assuming that it is homogeneous from the point of view of flood characteristics.

4. In order to aid those involved in design and field investigations, an attempt was made to develop equations to predict mean annual peak flows using

geomorphic parameters. Such equations are developed for each of the nine flood frequency regions and for the State as a whole.

5. An attempt was made to test and validate two of the rainfall-runoff models developed at the University College, Galway and reported to be suitable for the Western Ghat region by the CWRDM. Between the simple linear model and the linear perturbation model, the linear perturbation model was found to be useful for predicting the runoff in the two larger sub-basins of the Meenachil and Achencoil river basins of the Western Ghat region. Both the models were not suitable for two other small sub-basins studied in the Meenachil and the Achencoil river basins.

The major **limitations** of the study have been with regard to the depth and quality of data. However, even with the available data, this study is expected to be useful both from the academic and from the practical points of view. It would also serve as a guideline for similar future studies in the river basins of the Western Ghat region.

The practical utility of the study is highlighted below:

1. The results of the regional flood frequency study will be of use to those involved in design of buildings, roads, bridges, hydraulic structures and even urban planning and agricultural production.

2 It is difficult if not impossible to install stream gauging stations wherever development activities are envisaged. Therefore, the only alternative left out is to predict the mean annual peak flow with the information readily available. The equations developed relating the mean annual peak flows with the geomorphic parameters can be readily made use of to predict the peak flows, if the GTS maps of the Survey of India are available. Those involved in such exercises can directly make use of the equations developed relating the mean annual peak flows to geomorphic parameters.

3 Since the measurement of streamflow is both expensive and time consuming, the necessity has been recognised to relate runoff with rainfall, especially since rainfall data is more often available for the region. The study has brought to light that the linear perturbation model can provide good results for the larger sub-basins of the Western Ghat region. Such a model will help in predicting the flows in larger sub-basins which do not have stream gauging stations, but do have reliable rainfall stations.

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APPENDIX – 1

Latitude and Longitude of discharge stations

ST NO (1)	NAME OF STATION (2)	LATITUDE Deg.	LATITUDE Min. (3)	LONGITUDE Deg.	LONGITUDE Min. (4)
01	MANJESHWAR	12	43	74	54
02	ANAKKAL	12	42	75	00
03	UPPALA	12	41	74	55
04	SHIRIYA	12	37	75	01
05	MADHUR	12	33	75	01
06	PADIATHADUKA	12	30	75	05
07	PALLANGODE	12	27	75	11
08	MOONAMKADAVU	12	20	75	11
09	KAKKADAVU	12	16	75	15
10	MANGARA	12	09	75	27
11	IRUDE	12	02	75	35
12	KUILOOR	11	59	75	39
13	PALA	11	56	75	44
14	MERUVAMBAI	11	54	75	32
15	KANNAVAM	11	51	75	39
16	PATTIAM	11	47	75	34
17	VALAYAM	11	43	75	40
18	BAVELI	11	52	76	05
19	THIRUNELLY	11	54	76	00
20	THONDAR	11	45	75	56
21	MANAMTHODY	11	46	76	01
22	CHOORANI	11	38	75	57
23	PANAMARAM	11	44	76	04
24	KAKKAVAYAL	11	37	76	10
25	MUTHANGA	11	35	76	15
26	VAZHAVATTA	11	35	76	11
27	MANJAT	11	41	76	22
28	CHITTOOR	11	02	76	39
29	CHAVADIYUR	11	11	76	41
30	KADATHARAPUZHA	11	37	75	49

(1)	(2)	(3)		(4)	
31	KOLLIKKAL	11	27	75	50
32	KOODATHAI	11	18	75	54
33	MUKKOM	11	17	75	59
34	AREACODE	11	14	76	04
35	KANJIRAPUZHA	11	19	76	13
36	CHALIYAR	11	20	76	15
37	PUNNAPUZHA	11	18	76	18
38	MARUTHAPUZHA	11	21	76	17
39	KARIMPUZHA	11	14	76	13
40	KUTHIRAPUZHA	11	17	76	17
41	ANAKAYAM	11	06	76	08
42	THIRUVAGAPUZHA	10	54	76	09
43	THRITHALA	10	49	76	07
44	CHERUTHUTUTHY	10	45	76	16
45	PAMPADY	10	44	76	26
46	CHEERAKUZHY	10	43	76	26
47	MANAKADAVU	10	34	76	53
48	KUTTIPURAM	10	49	76	01
49	SILENT VALLEY	11	05	76	26
50	PAZHOOR	10	41	76	11
51	KUNDUKADAVU	10	32	76	18
52	MANALI	10	25	76	21
53	PILANTHODE	10	24	76	19
54	CHIMONI	10	23	76	29
55	MUPPLY	10	22	76	25
56	KURUMALI	10	24	76	10
57	KARUVANNUR	10	24	76	11
58	AMBALAKDAVU	10	17	76	20
59	KARAPPARA	10	26	76	36
60	KURIARKUTTY	10	25	76	45
61	KALADY	10	01	77	05
62	PILANTHODU	10	07	76	25
63	CHEMMANNAR	09	57	77	03
64	PERINJANKUTTY	09	59	77	12
65	KALLAR EASTERN	10	10	77	08

(1)	(2)	(3)		(4)	
66	THUVALUR	09	50	77	11
67	PANNIAR	10	07	76	23
68	CHAMPAKKAD	10	15	77	09
69	MUVATTUPUZHA	10	00	76	34
70	KAKKADASERRY	10	02	76	37
71	KALMPUR	10	00	76	38
72	THODUPUZHA	09	58	76	36
73	MALANKARA	09	50	76	46
74	PEROOR	09	37	76	35
75	PALAI	09	43	76	40
76	CHERIPPAD	09	42	76	46
77	TEEKOY	09	38	76	49
78	THONDRA	09	22	76	36
79	MANIMALA	09	27	76	44
80	MUNDAKAYAM	09	42	76	58
81	ERAPUZHA	09	15	76	35
82	KURUDAMANNIL	09	20	76	46
83	KOLLAKADAVU	09	16	76	35
84	PANDALAM	09	14	76	44
85	KONNY BRIDGE	09	12	76	53
86	ANAYADI	09	06	76	39
87	ENATHU	09	02	76	42
88	PUNALUR	09	03	76	55
89	THENMALA	08	57	77	05
90	AYOOR	08	53	76	55
91	VALAYANKIL	08	42	77	01
92	AYIROOR	08	43	76	46
93	MAMAM	08	40	76	50
94	MYLAMMOODE	08	44	77	00
95	ARYANAD	08	37	77	09
96	MARUTHANKUZHY	08	30	76	57
97	AMARAVILA	08	23	77	08
98	OTTASEKARAMANGALAM	08	27	77	09