FLOOD CHARACTERISTICS AND POTENTIAL RESERVOIR CAPACITY OF RIVER OSUN AT APOJE GAUGING STATION

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B. Eng (AGE/95/5757)

A THESIS IN

THE DEPARTMENT OF AGRICULTURAL ENGINEERING SUBMITTED TO

THE SCHOOL OF POSTGRADUATE STUDIES,

IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR THE AWARD OF

MASTER OF ENGINEERING (M. Eng.), DEGREE IN

AGRICULTURAL ENGINEERING (SOIL AND WATER ENGINEERING OPTION) OF

THE FEDERAL UNIVERSITY OF TECHNOLOGY, AKURE, ONDO STATE

CERTIFICATION

This work has not been presented elsewhere for the award of a degree, or any other purpose.

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ACKNOWLEDGEMENT

My appreciation goes to Christ, the High Priest for granting me the grace to undergo this postgraduate study. I appreciate His encouragement, comfort and strength during the dark hours of discouragement, frustration and loneliness. Also, I thank Him for the Light of His words, which remove all doubts and uncertainty presented by the world's systems.

I acknowledge the effort of my dear supervisor Dr. M. O Alatise for his invaluable contributions towards my academic performances and acquisition of knowledge. I appreciate the friendly disposition and comfort enjoyed in his home and office and in the period of associating with him. I am convinced that he has imparted moral and spiritual values in my life that will help me in handling futures academic challenges and undertakings. Your endurance, perseverance and tolerance are clear evidences of your maturity and wide exposures in the world of academics. Soon, I will rejoice with you as you reach the peak of your career in Jesus Name. Once again, I appreciate you it is well with your soul, spirit and body Amen. I appreciate the effort of Prof. A.S. Ogunlowo and Prof. A.A Olufayo for their contributions towards my academic career in life.

I cannot but appreciate the effort of my dear parent: Pa Daniel Adegboyega Adeboye and late Mrs. Marian Olajiire Adeboye who the saw importance in my formal education and embarked on it without reservation. How I wish my mother is alive today to hear the news of this success! Death has done the worst but Christ has done His best. Rest in peace till we meet in the Church of the Firstborn and sing the song of joy, "Hallelujah for the Lord God Omnipotent Reigns." The effort of my father towards the successful completion of this programme is well appreciated. Thank you for giving me the legacy of ages. May you reap the fruits of your labour Sir Amen.

I acknowledge the cooperation of Mr. Akinola of the Ogun-Osun River Basin and Rural Development Authourity, Abeokuta for his readiness to release the data used in this study. It is well with you Amen.

My thanks also go to my dear friends, Sola Adesanya, Ghenga Odeniyi, Joseph Orsar for their assistance during the process of compiling these reports. I appreciate the efforts of Tunde, Engr. Akinro and Wale Olarewaju and a host of others for their assistance in many ways. I appreciate the company and encouragement of sister Abiodun Abosede and my younger sisters, Iyabode and Fisayo in the course of this programme.

Finally, I give glory to *God* who strengthens me in all things.

DEDICATION.

This effort in ensuring a proper water management is dedicated to Jesus Christ in whom ALL things consist.

ABSTRACT.

Hydrological data namely: the total annual discharges and the gauge heights spanning 18 years (1982-1999) were obtained from Ogun-Osun River Basin and Rural Development Authourity, Abeokuta. The hydrograph and rating curve was drawn for the station. The flood frequency analysis was done using the "Annual maximum series" (AMS). Three statistical methods namely; the Normal, Log Normal and Log Pearson Type (111) distributions were fitted to the annual maximum discharges and flood frequencies were determined using seven plotting positions. The method of moment (MOM) was used in determining the mean, standard deviation and the coefficient of variation of the annual maximum discharges. On normal probability paper, the Observed and the predicted maximum annual discharges using the three distributions were plotted against their probabilities of non-exceedence. This procedure was repeated for the Weibull's, Blom's, Cunnane's, California, Gringorton's and the Chegodayev's plotting positions. At 95% confidence interval, the Upper and the lower frequencies of the maximum annual discharges were. The estimated upper and lower flood limits were determined

The time series of the annual hydrograph showed that the maximum annual discharges vary from169 m^3/s to 400m^3/s within duration of 18 years. The predicted annual maximum discharges under the Log Pearson Type (111) distribution using the Weibull's plotting position varies from 433, 440 and 444 m^3/sw at return periods of 25, 50 and 100 years with coefficient of correlation of 0.99. A skew coefficient of –0.5 was estimated for the maximum annual discharges of River Osun at Apoje sub-basin.

For each year, the monthly cumulative volumes were plotted against their correspondent months. The reservoir and the spillway capacities in the months of each year were estimated by calculating the maximum departure of the mass curve from the

year were estimated by calculating the maximum departure of the mass curve from the assumed uniform rates of withdrawal per month before and after spillage. The maximum reservoir capacity in each year was determined by adding the reservoir and the spillway capacities together.

The annual cumulative volumes vary from 68,000 million m^3 to 305,550 million m^3. The monthly demand varies from 5,030million m^3 to 25,400 million m^3 of water. The mean and coefficient of variation of the cumulative annual volume at Apoje sub-basin were $125.08 \times 10^9 m^3$ and 0.42 respectively. The estimated maximum reservoir capacity for Apoje sub-basin was 22,000 million m^3.

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SYMBOLS	UNITS	DESCRIPTIONS.
C_s	-	Coefficient of Skewness.
$C.V$	-	Coefficient of Variation.
D	-	Number of Days of Measurement
$K^{U}_{T,\alpha}$	-	Upper Confidence Factor
$K^{U}_{T,\alpha}$	-	Lower Confidence Factor
K_T	-	Frequency Factor.
k		Numerical Constant
$L_{T,\alpha}$	m^3s^{-1}	Lower Flood Limit at return period T &level of Sig n
		Number of Observations/Samples
P		Probability of Exceedence
Q	$m,\ m^3s^{-1}$	Discharges.
Q_T	m^3s^{-1}	Discharges at Return Period T
Q_a	m^3s^{-1}	Total Annual Discharges
$\overline{Q}$	m^3s^{-1}	Mean of the Annual maximal
Q_{max}	m^3s^{-1}	Syst. annual Maximal/ Maxi. Annual Discharges
S_v		Standard Deviation of the Annual Volume
S	-	Head loss per Unit Length of channel.
S_Q	m^3s^{-1}	Standard Deviation of the Annual Maximum Discharges.
S_y	m^3s^{-1}	Stand. Deviation of the Annual Maximum Discharges
$U_{T,\alpha}$	m^3s^{-1}	Upper Flood Limit at return period T& level of Sig. α
w	-	Intermediate Variable
y	m	Log Transformed Annual Maximum Discharges.
$\overline{y}$	ms^{-1}	Mean of the Log Transformed Annual Maximal
y_T	m^3s^{-1}	Variate of the Annual Maxima. at Return Ped T(Years)

α	-	Level of Significance
V_a	m^3	Monthly Total Volume
V_c	m^3	Annual Cumulative Volume
$\overline{V_c}$	m^3	Mean of the Annual Cumulative Volume

CHAPTER ONE

1.1 INTRODUCTION.

Water is the most abundant substance on earth. It is the principal constituent of the entire living thing and a major force that is constantly shaping the surface of the earth. Without water, the links within the ecosystem will be broken and this will eventually put an end to all forms of living thing. Water has notable contributions to progress of civilization. History has it that civilization of Egypt is traceable to the development of River Nile for irrigation and navigation (Ray *et al*, 1998). Even the ancient civilizations in Africa were dependent on the regular flow of rivers such as River Congo, Zambezi, Niger, Victoria, and Malawi (Marks, 2003). This is true for all other places of the world.

Through the study of engineering hydrology, the occurrence, circulation and distribution of water are monitored and assessed. Various physical and weather parameters such as rainfall, humidity, river stage, stream flows, temperature, evaporation, and sediment accumulation are quantified. The infiltration and subsequent percolation of water through soil layers to constitute groundwater are also of importance in hydrology (Wilson, 1993).

The various ways in which water finds applications are very evident. These are the domestic uses: ranging from direct consumption to other household activities, agricultural uses namely: crop production, fish farming, land and wild life conservation and industrial uses, which include hydroelectric power generation. The water requirement in all these areas will continue to increase due to increase in human populations (Neil, 1995).

Despite the relative abundance of water, the complaints everywhere remain the same and that is "shortage of supply of water both in quantity and quality". Generally, more than 1.4 Billion people in the world do not have access to portable water while in Africa, more than half of the populations do not have assess to safe drinking water let alone having enough for crop productions and other uses. There is a slogan of water, water everywhere, but there is little or none to drink (Zebediah, 1999).

Human activities directly or indirectly interfere with hydrologic cycle, thereby upsetting the natural balance and biodiversity. These include: deforestation, poor urban planning, urbanization, and illegal dumping of refuse on natural and artificial channels (Morgan, 1996). Some of these activities are beneficial to man, but in solving one problem another one may be created. Deforestation exposes land surfaces to the battering actions of heavy tropical rainfall. The topsoil is eroded and accumulated sediments block the waterways. Sediments and refuse reduce carrying capacities of channels. In most cases, these lead to flooding and their attendant consequences. In Nigeria, Ogunpa river flood in 1980 and the flood occurrences in Lekki, Kaduna, and Kano are glaring examples (Ayoade, 1992).

In Nigeria, more than 80 percent of the population is involved in agricultural activities such as cash crops production, and aquaculture. The extent of production in all these areas cannot justify the abundant land and water resources for, which the nation is endowed with. This is due primarily to poor planning, management and implementation of programmes relating to water resources development (Ayoade, 1992). As reported by (Alatise, 1992), River Basin and Rural Development Authorities were created by decree 25 of 1976 in order to check the poor trend in water resources utilization. The emphasis was laid on water resources development for agriculture and other productive sectors of the economy and this can only be achieved by proper assessment of the surface and ground water in order to meet the demands of the different sectors.

Apoje sub-basin, on River, Osun is located in the southwestern part of Nigeria. Irrigation projects in this sub-basin are non-existent, prospects in fish farming are yet to be tapped, and water supply to communities in this area is still inadequate. The farm settlement scheme established by the government of the old western region in Nigeria had been scrapped. The land and water resources in this area remain unused over the years. Therefore, the focus of this study is to make a scientific assessment and evaluation of the untapped water resources at Apoje sub-basin and to draw the attention of the government to these invaluable resources.

Normal, Lognormal and Log Pearson Type (111) distributions and seven plotting positions will be used in estimating the flood magnitude and frequency. Comparison will be made between the predicted and the observed annual maximum discharges in order to determine their adequacy of fit.

1.2 OBJECTIVES OF THE STUDY.

The objectives of this study are to:

(i) determine the flood characteristics of River Osun at Apoje sub- basin based on the available hydrological information;

(ii) estimate the flood magnitude and frequency of River Osun at Apoje station using different statistical methods;

(iii) determine the potential reservoir capacity of River Osun at Apoje gauging station.

1.3 JUSTIFICATION OF THE STUDY

The frequent occurrences of flood and their consequences have been a major problem in Nigeria. In most cases, large expanse of farmland, property and lives are destroyed. Recent flood disaster in Ibadan and Taraba states are clear evidences.

Low productivities in agricultural and industrial sectors can be attributed to poor utilization of abundant water resources in Nigeria.

Therefore, this project would provide information on magnitudes and frequency of occurrence at Apoje sub–basin as well as the abundant water resources that abound in the area.

1.4 SCOPE OF THE STUDY

This study highlighted the frequency and magnitude of flood at Apoje sub-basin of Osun River basin using 7 different plotting positions that were analysed by 3 statistical methods. It also give the reservoir capacity of Osun basin using Rippl's mass curve technique.

CHAPTER TWO

2.1 LITERATURE REVIEW

2.2 WATER RESOURCES PLANNING.

Water resources planning are of importance in economic and social development of any country. The quality, quantify and accessibility determine the levels of public health, food productions, energy and other important aspects of life (Neil, 1995). In order to meet the demands for the desired quantity and quality of water at a particular location, the engineers together with economists, political scientists, lawyers, planners and conservationists come together and prepare framework for implementation.

The incentives to plan for increase control of any water resources often follow a major disaster such as flood, drought, poor water quality conditions and water borne disease epidemic. Planning boards, advisory groups and public hearings may go a long way in sustaining the momentum needed to carry plans through implementation (French, 1986).

2.3 FLOODS.

Floods occurs when the carrying capacity of water channels in a drainage basin are exceeded and result in a rise and flowing of water on normally dry land in an over whelming volume (Ward, 1990). These lead to the large input of quick flow to the stream channel which in most cases is beyond the carrying capacity of channel. The severity of the flooding in an area will reflect the severity of the quick flow-forming processes. Hydrologically, floods can be categorised or classified into the following:

Inland River Flood: This occurs when heavy rainfall causes river and water channels to overflow thereby allowing free flow of water to their immediate vicinity.

This is rampant in urban areas where population is high. Instances are Mississippi river floods in 1973 and Ogunpa river floods in Ibadan in 1980. In-land river flood may isolate rural communities for long periods of time.

Mountain/Coaster River Floods: Due to the topographical features of hilly areas, flood posses high energy of flow and its effects can be more devastating than the inland river floods.

Flash Floods: This results from relatively short intense burst of rainfall commonly from thunderstorms. In urban centres where drainages are poor or non-existent, it poses greatest threat to lives and property.

Dam Break Floods: Failure of Dams and water retaining structures may lead to widespread of water on initially dry land.

Volcanic Floods: These are floods that occur as a result of volcanic processes. This is peculiar to areas that are prone to volcanic activities.

2.4 FLOOD DEFINITIONS.

For the purpose of statistical analysis, floods can be categorised into *ordinary floods, which* is the flood to be equalled in magnitude once or more time in the life of a project.

Annual flood is the highest in a water year, which may be equalled or exceeded once each year on the average.

Maximum known flood is the flood, which has occurred within the memory of the inhabitants of a region.

Maximum observed flood is the highest of the recorded floods of a section of a stream during a specified period, which may be a week, a year or an entire period of record.

Peak flood is the maximum instantaneous rate of flow during a flood.

Design flood is the flood adapted for design purposes and these may be the maximum probable flood or standard project flood or a flood corresponding to some desired frequency of occurrence depending on the standard security that will be provided against possible failure of structures.

Foundation design flood is the flood of higher magnitude that 50 years design flood which can occur in a life time of a bridge.

Standard project flood is the flood resulting from the most severe combinations of meteorological and hydrologic condition considered for a region excluding rare combinations.

2.5 CAUSES OF FLOODS.

Flood, is a natural disaster, which can be predicted with high degree of certainty. Its occurrences can be facilitated by some factors, which are natural and artificial. Natural factors are physical, vegetative and climatological characteristics of basins. A basin's physical characteristics include shape, size, elevation, soil and vegetation cover, while climatic features are rainfall intensity, duration, areal distribution. Geomorphology of a drainage basin also influences water accumulation, infiltration and overland flow. Artificial factors that contribute to flood occurrence are indiscriminate dumping of refuse on water ways and failure of water retaining structures such as dams, culverts (Eberhard, 1983).

2.6 FLOOD CONTROL MEASURES.

Flood control measures are strategies of preventing or managing the effects of excess water on land. The goal of flood mitigation therefore is to direct floodwaters away from areas where the risk to life and property would be very high, such as urban region, and towards areas where less damage would result (Brett and

Nikolas, 1999). Generally, flood control measures can be categorised into two, namely the structural and non-structural measures.

***2.6.1 Structural Measures*:** These include the construction of flood control dams, reservoir for water storage, chanelization of water courses and flood proofing of areas that are prone to floods (Neil, 1985). Structural measures are taken for the purpose of reducing the volume of flow of water and their impacts at the downstream side of a river. Using a direct sensitivity method (DSM), the flow models can be used in a series of hypothetical simulations incorporating various scenarios of active flood mitigation. With each simulation, the sensitivity of the flood wave magnitude to control measures can be calculated and the best location and instant for action can be determined. In less severe areas, the actual withdrawal may be achieved by either deliberately breaching a levee during a flood or by identifying strategic locations in advance of flooding events and installing appropriate structures that would permit controlled lateral outflow.

***2.6.2 Non-Structural Measures*:** These measures include formulation of policies and regulations, which directly influence land allocation, land-use and planning. Development and redevelopment policies on design and location of services and utilities in cities and sub-urban areas should be encouraged. Flood warning, permanent evacuation in severe areas and relief packages for the affected people will also reduce casualties.

2.7 FLOOD INFORMATION.

Information about stream flow is very important in flood magnitudes determination, flood forecast and quantitative design of embankment and water retaining structures. But in most cases, due to technical difficulties and lack of trained personnel, stream flow records are very scarce. In the process of using the available information for future predictions, extrapolations are made based on certain assumptions and these may increase the level of risk in project planning and implementation. For the purpose of analysis, the flood information are categorised into the historic and systematic information.

Historic Information is the data obtained prior to systematic collection of information (David *et al*, 1987). They are obtained with varying degree of accuracy and reliability especially from an ungauged station or site. Historic stream flow records are characterised by errors, uneven space of time of measurement and loss of certain peak flow which play important role in flood frequency and magnitudes determination (Minghui and Jery, 1989).

Systematic Information contains the continuous annual flood peak series. Systematic records are obtainable in gauged stations where daily stream flow records are measured. Estimation of flood magnitudes using the systematic information is more reliable and possesses high degree of consistency than those from the limited – historic data.

2.8 FLOOD FREQUENCY ANALYSES.

Flood frequency analysis is a set of statistical tools used to assess the significance of a flood event relative to stream flow conditions in a watershed. It is important for developing hydrologic models that predict the flood response to a rainfall input and also used for assessing return period of a flood event. Frequency of hydrologic event is the probability that some value of a discrete variable will be equalled or exceeded. According to Viessman and Lewis, (1996), two methods are used in hydrologic frequency analysis and are, the ***plotting technique*** and the use of ***frequency factors***.

2.8.1 Graphical Flood Frequency Analyses.

Graphical techniques involve the use of plotting positions in determining probabilities of exceedence. A variety of formulae have been proposed for estimating the exeedence probabilities of flood discharges in a record of annual flood (see Table 2.8.1). Although, the formulae may differ, all have the common ideas that the plotting positions can be used in estimating the magnitudes of hydrologic events and their probabilities of occurrence, detecting outliners, fitting distributions to data and evaluating adequacy of fit (Robert, 1987; Van-Thanh-Van, *et al*, 1989). In the recent times, analytical procedures for estimating distribution parameters such as probability-weighed moments and maximum likelihood have been considered in theory, more efficient than the graphical fitting method.

Generally, the probabilities that an event will be equalled or exceeded can be expressed as

$$P(Q \geq Q_T) = \frac{m+b}{n-a} \qquad (2..1)$$

Where, m = rank, n = number of events or observations, a and b are constants.

Table 2.1 Plotting Positions and the numerical values of the constants a and b.

Formulae	a and b $P_m = \left(\frac{m+b}{n-a}\right)$.
Hazen	a = 0, b= -0.5
California	a = 0, b= 0
Foster	a = 0, b= -1/2
Weibull	a = -1.0, b =0
Beard	a = -0.38, b = -0.3
Benard and Bos-levenbach	a = -0.2, b= -0.3
Chegodajev	a = -0.4, b = -0.3
Blom	a = -1/4, b = -3/8
Turkey	a = -1/3, b = -1/3
Gringorton	a = -0.12, b = -0.44
Cunnane	a = -0.2, b = - 0.4
Adamowski	a = -0.5, b = -0.25

2.8.2 Flood Analyses Using Frequency Factors.

For hydrologic frequency analysis, a dimensionless constant called frequency factor is used in flood magnitudes determinations. The preferred method of expressing flood frequency as recommended by the U.S Water Resources Council, (1981) is the annual exeedence probability. The probabilities that the ranked systematic annual maximal (Sumioka *et al,* 1998) will be equalled or exceeded in any specified year were determined by the following plotting positions (Van–Thanh *et al*, 1989) and expressed by the equations in Table 2.2

Table 2.2: Plotting Positions used in determining the Flood Frequencies.

Plotting Positions	Formulae
Hazen,	$P(Q \geq Q_T) = \frac{m-0.5}{n}$
Weibull	$P(Q \geq Q_T) = \frac{m}{n+1}$
Blom,	$P(Q \geq Q_T) = \frac{m-0.375}{n+0.25}$
Cunnane	$P(Q \geq Q_T) = \frac{m-0.4}{n+0.2}$
California	$P(Q \geq Q_T) = \frac{m}{n}$
Gringorton	$P(Q \geq Q_T) = \frac{m-0.44}{n+0.12}$
Chegodayeu,	$P(Q \geq Q_T) = \frac{m-0.3}{n+0.4}$

where P = probability of exceedence, Q = anticipated discharges (m^3/s), Q_T = discharges at estimated return periods to be equalled or exceeded, n = number of years of observations. m = Rank. The return periods were determined using the equation

$$T_r = \frac{1}{p} \tag{2.2}$$

where Tr is the return period, p is as defined previously.

2.9 PROBABILITY DISTRIBUTIIONS FOR HYDROLOGIC VARIABLES.

2.9.1 Normal Distribution: The probability distribution in this distribution according to (Chow et al, 1988) is expressed as

$$f(z) = \frac{1}{\sqrt{2\pi}} e^{-z^2/2} \quad (2.3)$$

where z is the standard normal variable. The statistical parameters namely, the mean and standard deviation of the annual maximum discharges were calculated using the method of moment and they are expressed as

$$\overline{Q} = \frac{\sum_{i=1}^{19} Q_{max}}{N} \quad (2.4)$$

$$S_Q = \sqrt{\frac{\sum_{i=1}^{19} (Q - \overline{Q})^2}{N-1}} \quad (2.5)$$

where , $\overline{Q}$ = Mean of the annual maximum discharges (m^3/s), Q_{max} = Systematic annual maximum discharges (m^3/s), N = Number of observations, S_Q = Standard deviation of the annual maximum discharges (m^3/s). The frequency factors corresponding to the return periods were calculated using the expression:

$$w = \sqrt{\ln\left[\frac{1}{p^2}\right]}, \qquad 0 < p \leq 0.5 \quad (2.6)$$

$$z = w - \frac{2.515517 + 0.802853w + 0.010328w^2}{1 + 1.432788w + 0.18926w^2 + 0.001308w^3} \quad (2.7)$$

where, w = intermediate variable, p = probability of exceedance, z = standard normal variable or the frequency factor. The estimated flood for the various return periods were calculated using the equation

$$\overline{Q} = \quad Q_T = \overline{Q} + z \cdot S_Q \qquad (2.8)$$

where, $\overline{Q}$, Q_T, and z are as previously defined.

2.9.2 Lognormal Distribution: The probability density function of a sample is expressed as

$$f(x) = \frac{1}{x\sigma\sqrt{2\pi}} \exp\left(-\frac{y-\mu_y}{2\sigma_y^2}\right) \qquad (2.9.)$$

The Log Normal distribution involves the logarithmic transformation of the annual maxima (to any base). Using the method of moment (MOM), the mean and standard deviation of the logarithm (base 10) of the annual maxima were determined by the expressions.

$$\overline{y} = \frac{\sum_{i=1}^{19} \log Q_{max}}{N} \qquad (2.10)$$

$$S_y = \sqrt{\left[\frac{\sum_{i=1}^{19} (y-\overline{y})^2}{(N-1)}\right]} \qquad (2.11)$$

where $\overline{y}$ = mean of y (m^3/s), N = number of observation, y = logarithm of annual maximal (to base 10), S_y = Standard deviation of y (m^3/s). Also, the flood estimates at different return periods were calculated by the equations.

$$y_T = \overline{y} + z \cdot S_y \qquad (2.12)$$

$$Q_T = 10^{\left[\overline{y} + z \times S_y\right]} \qquad (2.13)$$

Where, y_T = variate of annual maximal at return periods Tr(years), S_y $\overline{y}$ z Q_T are as previously defined. Log-Normal distribution is a special case of the Log Pearson

Type (III) distribution where the coefficient of skew ness is assumed to be zero. This indicates that the peak flows are normally distributed about their mean.

2.9.3 Log Pearson Type (111) Distribution: The distribution has been adopted as the standard for flood frequency analysis (Sumioka *et al*, 1998) and its probability density function can be expressed as

$$f(x) = \frac{\lambda^{\beta}(y-\varepsilon)^{\beta-1}\varepsilon^{-\lambda(y-\varepsilon)}}{x\Gamma(\beta)} \tag{2.14}$$

where, $y = \log x$, $\lambda = \frac{S_y}{\sqrt{\beta}}$, $\beta = \left[\frac{2}{C_s(y)}\right]^2$, $\varepsilon = \bar{y} = S_y\sqrt{\beta}$, (2.15)

The Log Pearson Type (III) distribution is referred to as the three parameter fit. The coefficient of skew ness of the logarithm (base 10) of the annual maximum discharges is obtained in addition to the mean and standard deviation calculated using the equations **2.10** and **2.11** respectively. The coefficient of skew ness is expressed as

$$C_s = \frac{n\cdot\left(\sum_{i=1}^{19}(y-\bar{y})^2\right)}{(N-1)(N-2)(N-3)} \tag{2.16}$$

Where $C_s =$ Coefficient of skew ness, S_y, $\bar{y}$, y and N are as preciously defined. The frequency factors K_T as given by (Kite, 1977) and is expressed by,

$$K_T = z + (z^2-1)k + \tfrac{1}{3}(z^3-6z)k^2 - (z^2-1)k^3 + zk^5 + \tfrac{1}{3}k^5 \tag{2.17}$$

$$k = \frac{C_s}{6}$$

where, $K_T =$ is the frequency factor, z, C_s are defined previously.

The flood estimates at values K_T is given by

$$Q_T = 10^{\left[\overline{y}+k_T \cdot S_y\right]} \quad (2.18)$$

where $Q_T, K_T, S, \overline{y}\ K_T$ are as previously defined.

2.10 RELIABILITY TEST.

A reliability test at 95% confidence limit and level of significance

$$\alpha = (\frac{1-0.95}{2}) = 0.025 \quad (2.19)$$

Are normally used in estimating the upper and lower flood limits. Mathematically, the upper and lower flood limits at recurrence interval T (years), according to (Chow, *et al*1988) are expressed as

$$U_{T,\alpha} = \overline{y} + S_y \cdot K^U_{T,\alpha} \quad (2.20)$$

$$L_{T,\alpha} = \overline{y} + S \cdot K^L_{T,\alpha} \quad (2.21)$$

respectively where $K^U_{T,\alpha}$ and $K^L_{T,\alpha}$ are upper and lower confidence factors at return periods T and level of significance α are defined by the equations:

$$K^U_{T,\alpha} = \frac{K_T + \sqrt{K_T^2 - ab}}{a} \quad (2.22)$$

and

$$K^L_{T,\alpha} = \frac{K_T + \sqrt{K_T^2 - ab}}{a} \quad (2.23)$$

The constants a and b in the equations above are defined as

$$a = 1 - \frac{z_\alpha^2}{2(N-1)} \quad (2.24)$$

$$b = K_T - \frac{z_\alpha^2}{N} \quad (2.25)$$

z_α is the standard normal variable with the probability of exceedence α = 0.025. The floods at upper and lower limits are estimated by the expressions:

$$Q^U_{T,\alpha} = 10^{\left[\bar{y}+S_y \cdot K^U_{T,\alpha}\right]} \tag{2.26}$$

$$Q^L_{T,\alpha} = 10^{\left[\bar{y}+S_y \cdot K^L_{T,\alpha}\right]} \tag{2.27}$$

respectively where, $Q^U_{T,\alpha}$ = the Upper flood limit at return period T and level of significance α, $Q^L_{T,\alpha}$ = the Lower flood limit at return period T and level of significance α, $\bar{y}$, S_y $K^U_{T,\alpha}$ $K^L_{T,\alpha}$ are as previously defined.

2.11 EFFECTS OF CATCHMENT CHARACTERISTICS ON RUNOFF.

Removal of forested area with its thick layer of mulch and its concentration to crop land or pasture causes a disturbance and the changes in infiltration rate of the soil. This results in increased runoff volume and changes in timing of flow. Likewise, urbanisation of land usually results in highly accelerated removal of storm water with the corresponding increase in the volume and peak of the runoff in a watershed (Chow, *et al*, 1988). The principal effects of land use have been categorised into changes in peak flow characteristics, changes in total runoff and quality of water, and changes in hydrologic amenities.

2.12 DAMS AND THEIR CAPACITIES DETERMINATION.

Dams are hydraulic structures, which are constructed across a river for the purpose of storing water in a reservoir and to build-up heads thereby increasing the potential of river water (Dandekar and Sharma, 1979). Civilisations in various parts of the world have been identified with the construction of major storage reservoirs to satisfy irrigation demands and other vita uses. Dams are built for the purposes of floodwater control especially in areas that are prone to river flooding. The dam built at saddel-el-kafara, Egypt around 2,600BC is generally accepted as the oldest known dam of real significance. Numerous other dams were constructed in modern Iraq, Iran and Saudi Arabia, Yeman around 750 BC (Novak *et al,* 1990). In the modern era, the usefulness of dams goes beyond irrigation and water storage only. Dams are built for power generation and examples include Konya dam in Indian 1954 and Kainji hydroelectric dam in Nigeria (Dandekar and Sharma, 1979). Dams are classified on factors and criteria. They can be broadly classified into large dams and small dams. *Large dams* are dams whose storage capacity and height are greater than 62 Million cubic metres and about 18.29metre respectively based on the recommendations of the Task Force on Spillway Design Floods. In such dams, major failure and their considerable loss of lives cannot be tolerated (Viessman, 1996). The spillway design is based on the probable maximum flood (PMF). The intermediate and minor dams are designed based on the requirements of standard projects and frequency analysis respectively. Small dams are those whose capacity is less than 31 Million cubic metre. Dams are also grouped based on the materials of construction such as *earth dams, rock fill dams, and concrete dams.* Based on hydraulic design, there are overflow and non-overflow dams.

2.13 PLANNING CONSIDERATIONS OF PROJECT INVOLVING DAM AND RESERVOIR.

According to Roberson *et al,*(1988), planning means determining in an orderly manner the best way to accomplish a particular objective by evaluating various alternatives. Whether it is for municipal water supply, irrigation or hydroelectric power generation, the following items are considered in dam and reservoir designs. Hydrological data of the stream where the dam is to be built are analysed to determine flood and drought frequencies and magnitudes, reservoir and spillway capacities. Geophysical investigation of the sites is carried out to determine the stability of the geologic formations. Also a complete assessment of the area to be inundated by the reservoir must be made and these include topographic map, land ownership, land classification, location of roads and other public utilities.

2.14 RESERVOIRS.

A reservoir is a man-made lake or structure that is used to store water. Generally, outflow can be monitored but in some cases inflow cannot be controlled. Common instances are those sited on hills for municipal water supply and those created by damming on stream. The reservoir Capacity is the volume of water that can be stored by a reservoir. In the case of man-made tanks, the capacity is the inside volume below the maximum water level. For a reservoir behind a dam, the capacity is the volume of water in the reservoir below the normal maximum pool level and this can be determined using the topographic map of the region.

CHAPTER THREE

3.1 RESEARCH METHODOLOGY.

3.2 BASIN LOCATION AND PHYSIOGRAPHY.

The Osun and Ona River Basins are located in an area whose boundaries are approximately latitudes $8^0 20^1$N and $6^0 30^1$N and longitudes $5^0 10^1$N and $3^0 25^1$N. The combined area of the two basins as revealed by the feasibility report of Ogun-Osun River Basin and Rural Development Authority is approximately 16,700Km^2. The greater parts of Osun and Ona River Basins are located in Oyo and Ogun States with less than 7% of the entire areas located in Kwara and Lagos States of Nigeria. The major drainage system in the basin is that of River Osun which rises in Oke-Mesi ridge about 5km North of Effon-Alaye on the border between the former Oyo and Ondo States of Nigeria, and flows through the Itawure gap of latitude $7^0 53^1 30^{11}$ before winding its way westwards through Oshogbo and Ede and Southwards to enter Lagos Lagoon about 8 km East of Epe. The main tributary is River Oba that rises about 15 km North of Ogbomoso and Erinle that rises South of Offa (see Fig 3.1).

3.3 CLIMATE.

Generally, the Osun basin's climate is influenced by the movement of the inter-tropical convergence Zone (ITCZ), a quasi-stationary boundary zone which separates the sub-tropical continental air mass over the Sahara and the equatorial maritime air mass over the Atlantic Ocean. The former air mass is characterised by the dry North- Easterly winds called the Barmattan found in the rain – bearing South-Westerly winds from the Gulf of Guinea.

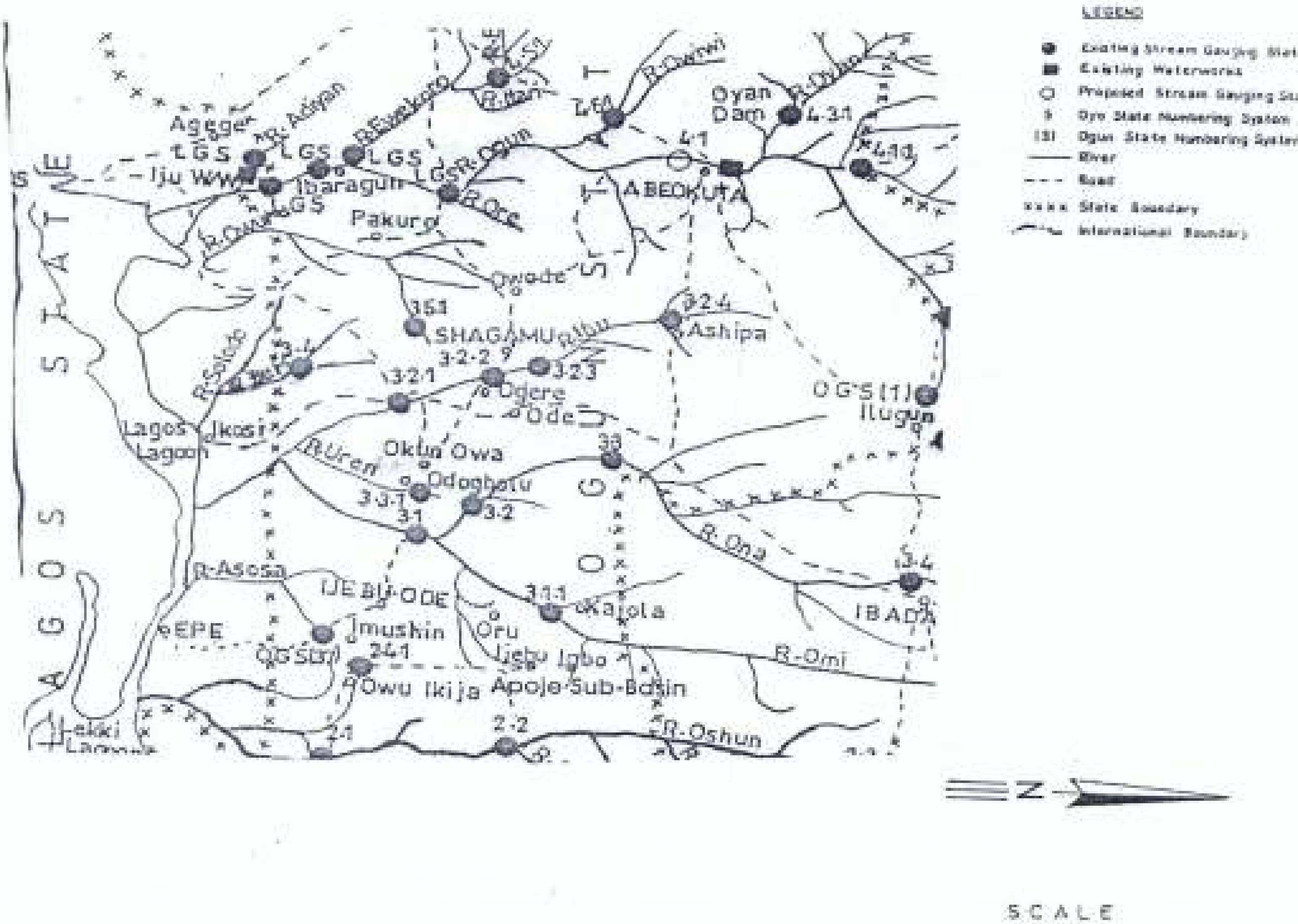

Fig.3.1 Hydrological Networks of Ogun- Osun River Basins.

Source: *Feasibility Study of Osun and Ona River Basins, Published by Ogun-Osun River Basin and Rural Development Authourity, Abeokuta.*

The ITCZ moves Northwards beyond the basin at the peak of the rainy season in June, July and Southwards to the coast, in the middle of dry season in December and January. Rather abrupt is the change from rainy season to the dry season while the entrance of the rains after the dry season is gradual (Alatise, 1992).

3.3.1 Rainfall.

Seasonal distribution is the main feature of rainfall pattern. Rainy season emerges earlier in the south in March and continues till end of October or early November, with at least seven months of rainfall. In the North of Ogbomoso the rain starts in early May or late April and ends in middle of October. Dry days are regular and sufficiently regular in late July and early August to constitute "little dry season" whose monthly precipitation depth is below 120mm. In the wet season, the mean rainfall ranges between 1,020 and 1,520 mm in the South of the basin, but in the North, it is less than 1,020 mm. In the North and South, the mean dry season rainfall varies from 127 to 178 m*m* and 178 to 254 mm respectively.

3.3.2 Temperature.

The record of temperature in the basin is that the hottest months are February and March during which temperature are high over the entire area. For the month of February, the mean daily maximum temperature is 31.4^0C in the North. The minimum recorded temperature during Hammatan in the North is 47^0C. During the rainy season in the month of July, the lowest mean minimum temperature of about 22.8^0C was recorded.

3.3.3 Humidity.

In the basin, at 12 G.M.T the lowest mean monthly humidity is 62% in the South but 50% in the North. The mean annual humidity varies from 75% in the South to 55% in the North.

3.4 SOIL AND LAND RESOURCES.

Due to the erosion and sediment transport, the soils in the basin have developed into alluvial parent materials. The basement complex in the upper part of the basin gives rise to a wide variety of soils, coarse in texture and of low fertility. For productive intensive crop production in the basin, irrigation and heavy fertilizer application will be highly required.

The soils in the basin are classified into two groups on the basis of location and elevation. These are:

(i) The upland soils which are more developed and ranges from heavy and hydromorphic to coarse and well-drained;

(ii) The lowland soil, which are hydromorphic and affected by a high water table and seasonal flooding.

3.5 HYDROLOGY OF RIVER OSUN.

Water resources in Osun basin include the surface water and groundwater.

Surface water plays the prominent role in the basin. Sometimes limited stream flow records create problems in water resources assessment. Generation of long record is done by streamflow synthesis, of rainfall records. In Osun River basins, the two major potential sources of underground water are the coastal plains sand (incorporating the upper part of Ilaro & Abeokuta formations: These formations have the following common features:

(i) the origin of the deposition;

(ii) the mode of deposition;

(iii) Alternation of sands, lenticular horixons and clayed units with relatively thin sand horizons. (Alatise, 1992).

(iii) Alternation of sands, lenticular horixons and clayed units with relatively thin sand horizons. (Alatise, 1992).

The movement of the groundwater is from the North to South towards the sea, and the aquifer units to the east and west are assumed to coincide with the boundaries of surface catchments. The coaster plain sands are overlain by alluvium in the valleys and along rivers are unnearfined and drains into the Lagoon in Lekki.

The formation in Abeokuta comprises of the phreatic zone which is replenished by rainfall infiltration through the unsaturated formation and confined zone which dips to the South of the formation outcrops with area considerably in excess of the unconfined phreatic zone. Due to the relative abundance of surface water in the basin, groundwater seems not to be of any significance for crop cultivation and consumption.

3.6 HYDROLOGICAL NETWORKS OF OSUN RIVER BASIN.

There are sixteen (16) stream gauging stations and seven (7) waterworks within the boundaries of Osun river basin. Three of the waterworks, Ijebu-Igbo, Asejire and Ijebu-Ode have stream-gauging stations in their vicinity to measure flows. For effective management, the gauging stations and water works are controlled by and operated by agencies in their respective states namely: Water Corporation of Oyo State (WCOS) and Ogun State water Corporation (OSWC). The numbering system used in these stations is those introduced by the former Western Corporation and (OSWC). These are shown in Fig. 3.1

3.7 AVAILABLE INFORMATION.

The discharges and gauge heights of River Osun at Apoje gange station spanning 19 years (1982-1999) were obtained from Ogun – Osun River Basin and Rural Development Authority, Abeokuta, Ogun State. The peak discharges, that is the annual maximum discharges of each year and the corresponding peak gauge heights were selected. The selected annual maximum discharges were ranked in descending order of magnitude to form an annual maximum series (AMS) based on the recommendation of the U.S. Water Resource council Bulletin 17B of 1981 (see Appendix A).

3.8 FLOOD CHARACTERISTICS OF RIVER OSUN.

The selected and ranked annual maximum discharges were plotted against their corresponding years to generate an annual hydrograph for River Osun at Apoje gauging station. Likewise, the gauge heights were plotted against their corresponding annual maximum discharges in order to generate a rating curve for the station.

3.9 FLOOD FREQUENCY ANALYSES.

The probabilities that the ranked annual maximum discharges will be equaled or exceeded in any year were determined using the Hazen's, Weibulls, Blom's, Cunnane's, California, Gringorton's and Chegodayev's plotting positions as shown in **Table 2.2.** Under the Hazen's plotting position, the probabilities of non–exeedence of the annual maximum discharges were determined by deducting the exceedence probabilities from unity (1). The return periods were determined using equation **2.2.**

3.10 PROBABILITY DISTRIBUTIONS.

Three statistical methods namely: Normal, Log Normal and Log Pearson Type (III) distributions were fitted to the ranked annual maximum discharges of River Osun. A skew coefficient of zero was assumed for both the Normal and Log Normal distributions. Under the Normal Distribution, the mean and standard deviation of the ranked annual maximum discharges were determined using equations **2.4, and 2.5** respectively. The intermediate and frequency factors were determined using the equations **2.6** and **2.7** The Normal distribution was fitted on each observed annual maximum discharge by using equation **2.8.** Under the Log Normal distribution, the logarithms (based 10) of the ranked observed maximum annual discharges were taken. The mean, standard deviation and frequency factors were determined using the equations **2.10, 2.11 and 2.7** respectively. The Log Normal distribution was fitted on each observed annual maximum discharge using equation **2.13**. Likewise, under the Log Pearson Type (III) distribution, the mean, standard deviation and the skew coefficient of the log transformed annual maximum discharges were determined using the equations **2.10, 2.11 and 2.16** respectively. The intermediate and frequency factors were determined using the equations **2.6** and **2.7** respectively. The calculated skew coefficient was used in determining the frequency factors using the equation **2.17**. The Log Pearson Type (III) distribution was fitted on each observed annual maximum discharges using the equation **2.18**. On normal probability papers, the predicted annual maximum discharges obtained using Normal distribution and the observed annual maximum discharges were plotted against their probabilities of non-exceedence using the Hazen's plotting position. Likewise, this was done for Log Normal and Log Pearson Type (III) Distributions using the Hazen's plotting position. The same procedures were carried out using the Weibull's, Blom's, Cunnane's, California, Gringortion's and the Chegodajev's plotting positions.

3.11 RELIABILITY.

In order to determine the upper and the lower flood limits of River Osun at Apoje sub-basin, a reliability test was carried out at 95% confidence limit on the observed annual maximum discharges. The probabilities of exceedence were determined using Welbull's plotting position. The upper and the lower confidence factors were determined using the equations **2.22** and **2.23** respectively. At return period Tr (years), the upper and lower flood limits were estimated using equations **2.26** and **2.27** respectively. The upper and lower flood limits were plotted against their corresponding non-exceedence probabilities.

3.12 RESERVOIR CAPACITY OF RIVER OSUN AT APOJE GAUGING STATION.

The daily discharges of each month (from January to December) were added together to estimate the monthly total discharge. The estimated monthly total discharges were converted to volume of flow using the expression

$$V_m = \left[Q_a \times D \times 24 \times 3600\right]\ m^3 \qquad (3.1)$$

where V_m = monthly Total volume Q_m monthly Total discharges (m^3/s), D = Number of days of measurement. The cumulative volume of water for each year was determined using the expression

$$V_c = \sum_{i=1}^{12} V_a), \quad (m^3) \qquad (3.2)$$

The mean, standard deviation and coefficient of variation of the total annual volume were determined using the expressions:

$$\overline{V_a} = \frac{1}{N}\sum_{i=1}^{12} V_a \quad (m^3) \qquad (3.3)$$

$$S_v = \sqrt{\frac{1}{N-1}\cdot\sum_{i=1}^{12}\left(V_a - \overline{V_a}\right)^2} \quad (m^3) \qquad (3.4)$$

and $$C.V = \frac{S_v}{V_a} \qquad (3.5)$$

where $\overline{V_a}$ = Mean annual volume (m^3), V_m = Monthly Total volume, S_a = Standard deviation of the annual total volume, $C.V$ = is the coefficient of variation of the inflow.

Due to the spatial distribution of total annual inflows at Apoje sub-basin, it is of importance to quantitatively determine the monthly and the annual inflow in each year in order to determine the sizes of spillway and reservoir for effective management of the water resources. Mass curve was generated for each year by plotting the cumulative inflow against the months. A uniform rate of water withdrawal was assumed for the annual cumulative inflow and determined by dividing the cumulative inflow by thirty (12). The hypothetical reservoir and spillway capacities were determined by calculating the departure of the mass curves from the uniform rates of withdrawal at the periods of low and excess inflow respectively. The maximum reservoir capacity for each year was determined by adding highest "the reservoir and spillway capacities" together.

CHAPTER FOUR

4.1 RESULTS AND DISCUSSION.

4.2 FLOOD CHARACTERISTICS OF RIVER OSUN AT APOJE SUB-BASIN.

The data on peak instantaneous discharges and their corresponding gauge heights at Apoje gauging station for the period of 18 years observation are detailed in Appendix A. The peak discharges were plotted against their corresponding water years in order to generate a time series of the annual hydrograph as shown in Fig. 4.1. From this figure, it can be seen that the highest discharge of 400m^3/s was observed in 1987 but later declined to 169m^3/s in the year 1999. The random nature of the peak discharges over the years shows that they are stochastic hence the probability distribution for the purpose of prediction of future events was appropriate. According to Kottegoda, (1980), emphasis in time series analyses should be laid on the mechanisms that generate the data but not necessarily on the future sequence of events over a period of time. In the time series of the annual hydrograph, the rainfall distribution, which is a function of the catchment's climatic characteristics within the Osun basin, is responsible for the difference in magnitude and length of the observations of the peak discharges. The diversion of water to the control structures at Asejire and Ede is also responsible for the variation of the peak discharges of Apoje, which is the last gauging station along River Osun (see Fig 3.1).

Fig 4.2 shows the monthly hydrograph of River Osun at Apoje gauging station for the 1987 and 1998 water years. The peak discharge in the stated years were selected and plotted against their corresponding months. In 1987, the peak discharge in January, was 82m^3/s which later declined to 72m^3/s in February

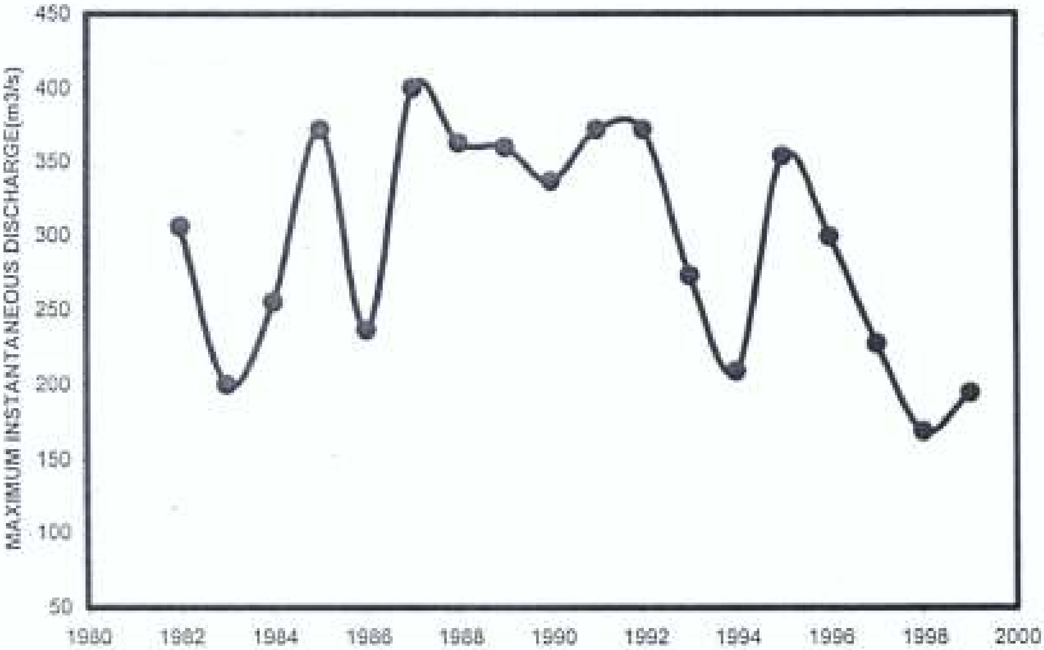

FIG. 4.1 ANNUAL HYDROGRAPH OF RIVER OSUN AT APOJE GAUGING STATION(1982-1999)

and this value remained constant till April. The peak discharge in May was 83.5m^3/s, which increased to 168.5m^3/s in July. The highest "peak discharge" of 400 m^3/s was observed in September. This can be attributed to the fact that heavy rainfall normally begins in September. The peak discharge fell from 366m^3/s in October to 112 m^3/s in December.

In 1998, the peak discharge in January was 97m^3/s, which later reduced to 84m^3/s. The peak discharge remained at 72m^3/s between March and May but later increased to 102m^3/s in June. In October, the peak discharge was 169m^3/s, which later declined to 102m^3/s in December.

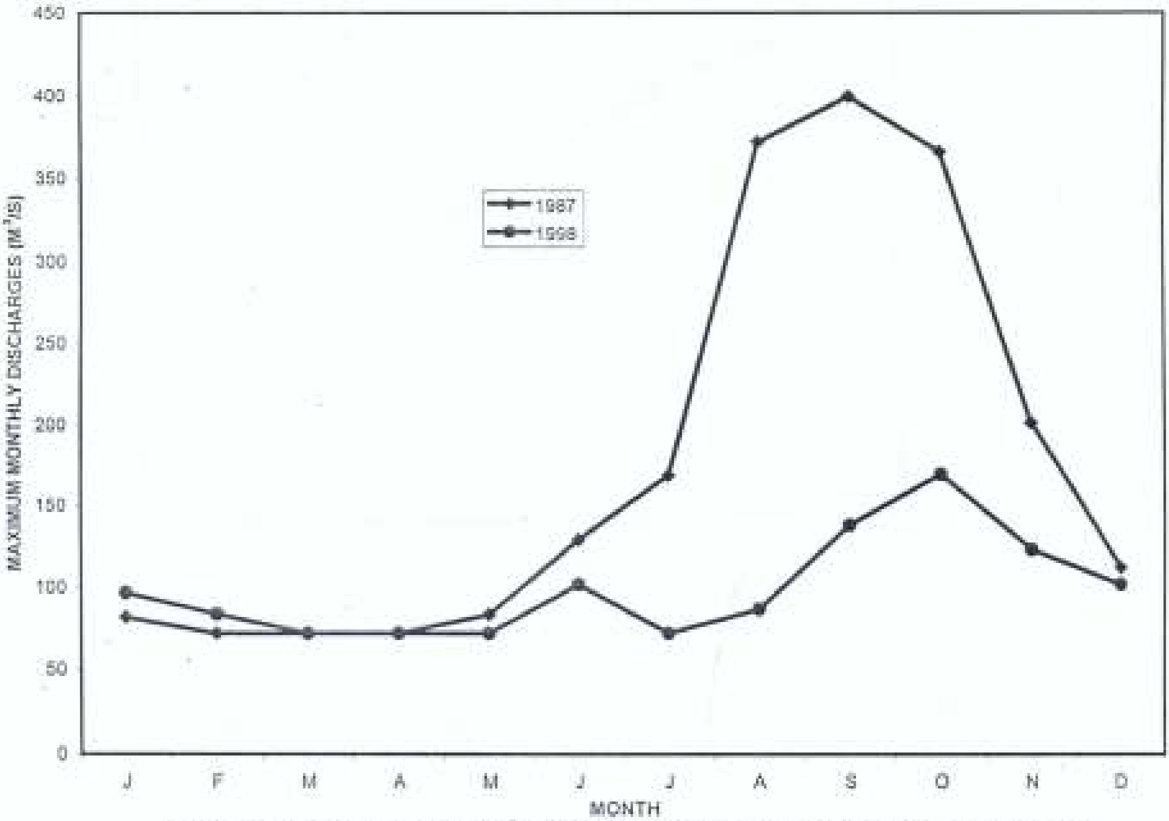

FIG 4.2 MONTHLY HYDROGRAPH OF RIVER OSUN AT APOJE GAUGING STATION IN 1987 AND 1998

4.3 THE RATING CURVE.

By plotting the gauge heights against their corresponding peak instantaneous discharges, a rating curve was generated for Apoje station.

Fig 4.3 shows that an exponential equation perfectly fit the variation of the gauge heights with the peak discharges for the Apoje gauging station. This is in aggrement with (Chow et al, 1988).

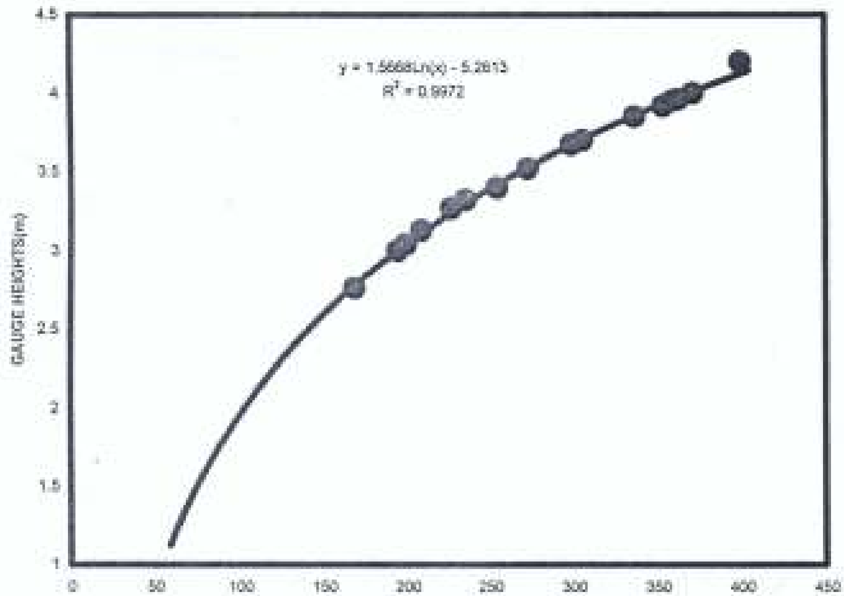

FIG ANNUAL MAXIMUM DISCHARGES (m3/s)

RATING CURVE OF RIVER OSUN AT APOJE GAUGING STATION (1982-1999)

Table 4.1: Predicted Gauge Heights for Upper Flood Limits at 95% Confidence Interval

Return Periods (Yrs)	Upper Flood Limits/PMF (m^3s^{-1})	Gauge Heights (m)
2	315.1	3.45
5	347.1	3.90
10	430.4	4.24
25	451.1	4.32
50	458.2	4.34
100	461.7	4.35

The regression equation relating the gauge heights and the peak instantaneous discharges is expressed by

$$H = 1.5668 \ln Q - 5.2613 \qquad (4.1)$$

where , H = gauge height (m), Q = peak instantaneous discharges (m^3/s).

Table 4.1 contains the predicted gauge heights for the upper flood limits at different return periods. For design purposes, the Upper flood Limits at the various return periods are higher and therefore can be relied upon for design of hydraulic structures at Apoje sub- basin.

4.4 STATISTICAL PARAMETERS AND DISTRIBUTIIONS.

The total annual discharge of each year was determined by adding the monthly discharges together. The result is presented in Table 4.2. The maximum "annual total discharge" of 61,747.7m^3/s was observed in 1992 while the minimum "total annual discharge" of 23,1987m^3/s was observed in the year 1998. A coefficient of variation of 0.25 was obtained for the total annual discharge. Also, a coefficient of skew ness of zero under the Normal and the Log Normal Distributions show that the maximum annual discharges were normally distributed about their mean discharges (see Table 4.3). The logarithmic transformation of the annual maximum discharges was done under the Log Normal and Log Pearson Type (111) distributions and a coefficient of 0.05 was obtained. This shows that the logarithmic transformation of the annual maximum discharges led to a reduction in the coefficient of variation when compared with Normal distribution.

Table 4.2: The Total Annual Discharges of River Osun at Apoje Gauging Station from 1982 to 1999.

WATER YEARS	ANNUAL TOTAL DISCHARGES (m^3/s)
1982	32,983.5
1983	33,796.1
1984	42,073.0
1985	43,401.6
1986	32,475.9
1987	44,845.2
1988	49,134.6
1989	51,238.2
1990	52,544.0
1991	53,069.0
1992	61,747.7
1993	58,588.0
1994	51,246.5
1995	48,054.9
1996	46,094.2
1997	41,173.4
1998	23,198.7
1999	26,540.4

Table 4.3: Statistical parameters used in fitting the distributions.

	Maximum Discharges (m^3s^{-1})	Minimum Discharges (m^3s^{-1})	Mean Discharges (m^3s^{-1})	Standard Deviation of Discharges (m^3s^{-1})	Coefficient Of Variation	Coefficient Of Skewness
Annual Total Discharge	61748	23,199	44,125	10,724	0.24	-
Normal Distribution	400	169	294.4	74.9	0.25	0.00
Log Normal Distribution	2.60	2.23	2.46	0.12	0.05	0.00
Log Pearson Type (III) Distribution	2.60	2.23	2.46	0.12	0.05	-0.5

In this study, the Normal distribution was fitted to the annual maximum instantaneous discharges using the mean and the standard deviation in Table 4.3 the expression is given by

$$Q = 294.4 + z \times 74.9 \quad m^3 s^{-1} \tag{4.2}$$

Likewise, the Log Normal and Log Pearson Type (III) distributions were fitted to the peak instantaneous discharges by the mathematical expressions:

$$Q = 10^{[(2.46 + z \times 0.12)]} \quad (m^3 s^{-1}) \tag{4.3}$$

$$Q = 10^{[(2.46 + K_T \times 0.12)]} \quad (m^3 s^{-1}) \tag{4.4}$$

respectively, where Q, Z and K_T are as defined previously. These equations were used in predicting the discharges in **Appendices B1- B7.**

4.5 PLOTTING POSITIONS AND RETURN PERIODS.

The various plotting positions used in this study include those of Hazen, Weibull, Blom, Cunnane, California, Gringorton and Chegodayev. The return periods in years for the ranked peak instantaneous discharges were estimated by finding the reciprocals of the probabilities of exeedence determined by the plotting positions. The variation of the return periods with the plotting positions for each observed peak discharge is shown in Table 4.4.

For a peak instantaneous discharge of 400m^3/s, the Hazen's formula predicted a return period of 38 years while the Weibull's and California formulae allocated return periods of 20 and 19 years respectively. The return periods allocated by Gringorton and Chegodayev's plotting positions were 34 and 28 years respectively. For a peak discharge of 255m^3/s, a return period of 2 years was arrived at for all the plotting positions. For a peak discharge of 169m^3/s, the return period allocated by all the plotting positions was 1year. In using the Hazen's formula to determine the return period for the observed peak discharges, it indicates that a discharge of 400m^3/s will occur at least once every 38 years. The Weibull's and the California formulae are biased because shorter return periods are given to the discharges of the same value when compared with that of Hazen's. This is one of the limitations of the Weibull's formula in practice especially for non-uniform distributions (Van-Thanh *et al*, 1989).

Table 4.4: Variations of Return Periods with Plotting Positions.

Historic Annual Maximal	Return Periods Tr (Years)						
	Hazen (Yrs)	Weibull (Yrs)	Blom (Yrs)	Cunnane (Yrs)	California (Yrs)	Gringorton (Yrs)	Chegoda. (Yrs)
400	38	20	31	32	19	34	28.
372	13	10	12	12	10	12	11
372	13.	10	12	12	10	12	11
372	13	10	12	12	10	12	11
363	8	7	7	7	6	7	7
360	5	5	5	5	5	5.	5
354	4	4	4	4	4	4	4
337	3.5	3	3	3	3	3	3
306	3	3	3	3	3	3	3
299	2.5	2.5	2.5	2.5	2	2.5	3
273	2.	2	2	2.	2	2.2	2
255	2.	2	2	2.	2	2.	2
236.5	2	2	2	2	2	2	2
227.5	2	2	2	2	2	1	2
209.5	1.5	1.5	1.5	1.5	1.5	1	1.5
200	1	1.	1	1	1	1	1
195	1	1	1	1	1	1	1
169	1	1	1	1	1	1	1

In this study, three statistical distributions, the Normal, Log Normal and Log Pearson Type (III) distributions were fitted to the annual maximum discharges of River Osun at Apoje gauging station. Therefore, the question of the best fit among the three statistical distributions is of interest. The best distribution(s) is/are determined by means of criteria depending on the differences between the observed and predicted distributions (Kottegoda, 1980). According to (Benjamin, Garry and Jean, 2001), the least square regression equations were used in estimating the magnitudes of the predicted discharges at specific return periods of 25,50 and 100 years. In the present study, flood frequency curve was generated for each distribution whose probability of exceedence was determined by the stated plotting positions. Similarly, comparisons were made between the observed peak instantaneous discharges and those predicted by the using the regression equation under each distribution for the purpose of selecting the best among them.

4.6 HAZEN'S PLOTTING POSITION.

Appendix B1 contains the predicted floods using the Normal, Log Normal and Log Pearson Type (111) distributions. Fig 4.4a shows the graph of the normal distribution for the observed and the predicted discharges of River Osun at Apoje gauging station. From this figure, it can be seen that at the return periods of 25, 50 and 100 years, the observed discharges were 404, 412 and 417m^3/s while at the same return periods, the predicted discharges were 410, 416 and 420m^3/s respectively. The coefficient of correlation of 0.98 was obtained for the observed discharges, while for the predicted discharges, it was 0.99. These means that the observed and the predicted discharges are well correlated and that the historic data are accurate and reliable.

Fig 4.4b shows the graph of the Log Normal distribution for the observed and the predicted discharges of River Osun at Apoje gauging station. At return periods of 25, 50 and 100 years, the observed discharges were 404, 413 and 417m^3/s respectively while predicted discharges were 434, 441 and 445m^3/s respectively for the same return periods.

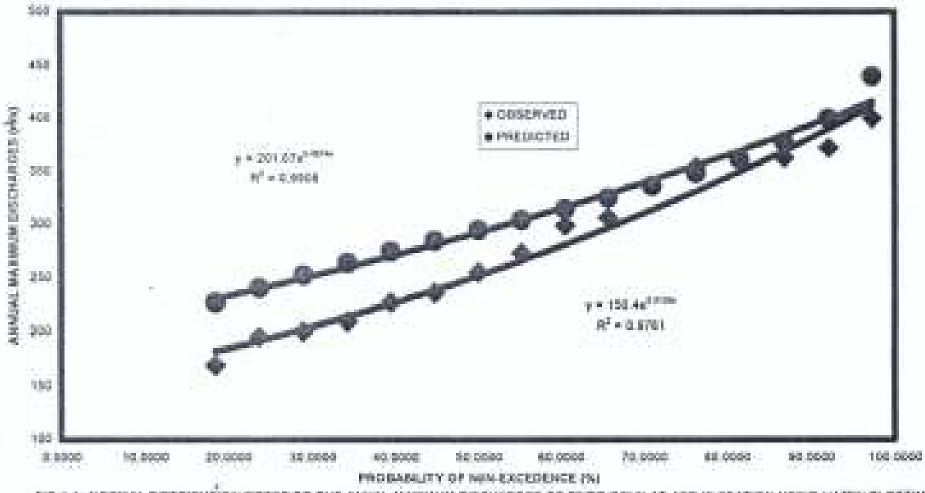

FIG 4.4a NORMAL DISTRIBUTION FITTED TO THE ANUAL MAXIMUM DISCHARGES OF RIVER OSUN AT APOJE STATION USING HAZEN PLOTTIM POSITION

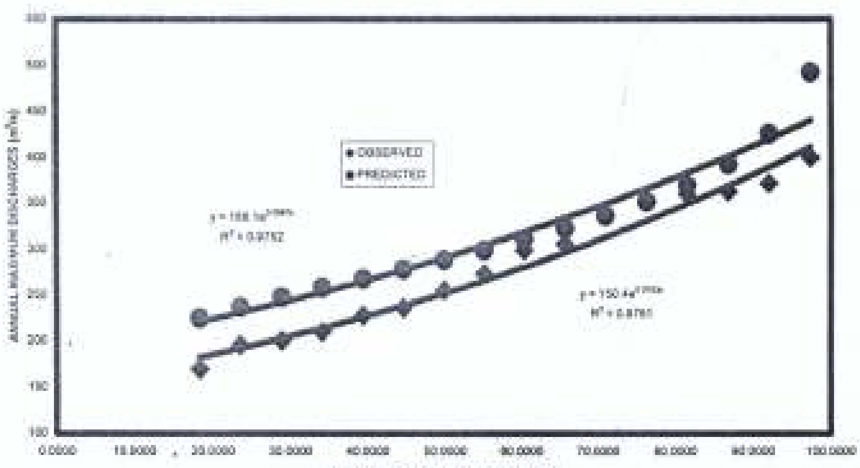

FIG 4.4b LOG NORMAL DISTRIBUTION FITTED TO THE ANNUAL MAXIMUM DISCHARGES OF RIVER OSUN AT APOJE STATION HAZEN PLOTTING POSITION

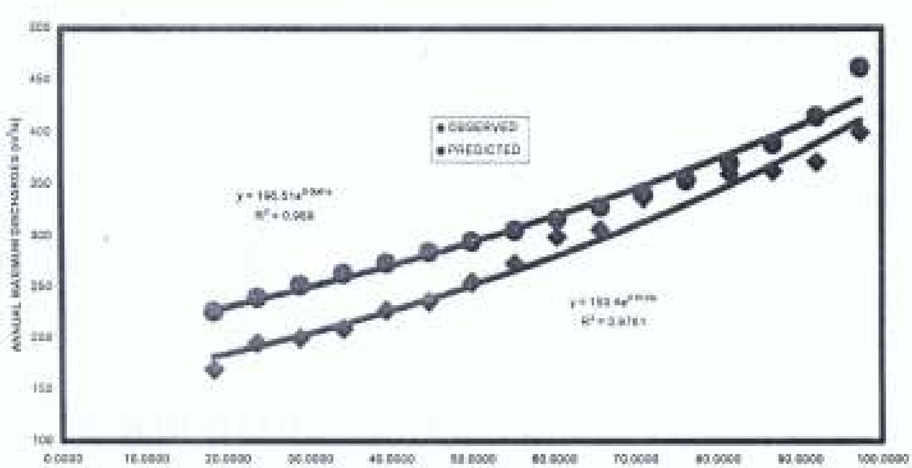

FIG 4.4c LOG PEARSON TYPE (111) DISTRIBUTION FITTED TO THE ANNUAL MAXIMUM DISCHARGES OF RIVER OSUN AT APOJE STATION USING HAZEN PLOTTING POSITION

The coefficients of correlation for the observed discharges was 0.98 while for the predicted discharges, it was 0.98 meaning that the predicted and the observed discharges have the same degree of fit.

Fig 4.4c shows the graph of the Log Pearson Type (111) distribution for the observed and the predicted discharges of River Osun at Apoje gauging station. The corresponding values at 25,50 and 100 years return periods for the observed discharges were 404, 413 and 417m^3/s, and for the predicted discharges were 427, 435 and 438m^3/s respectively. The coefficients of correlation were 0.98 for the observed and 0.99 for the predicted. From Fig 4.6a, b and c, the predicted discharges obtained from the Log Pearson Type (111) distribution were smaller than those obtained from the Log Normal distribution at the return periods of 5 years and above. This was due to the introduction of the skew coefficient of (–0.5) in determining the frequency factors under the Log Pearson Type (111) distribution and this was in compliance with (Chow, et. al 1988). At return periods below 5 years, the predicted discharges in Fig 4.4c are higher than those in Fig 4.4b. At return periods of 2 years and below, the predicted discharges in Fig 4.6c compares favourably with those in Fig 4.4a. The highest coefficient of determination was obtained for the predicted discharges under the Log Pearson Type (111) and and Normal distributions. But the predicted discharges under the log Pearson Type (iii) distribution are higher than those under the Normal distribution. Therefore, using the Hazen's plotting position for the three distributions for the stream flows at Apoje sub-basin, the Log Pearson Type (111) distribution can be taken as the most reliable.

4.7 WEIBULL'S PLOTTING POSITION.

Appendix B2 contains the predicted floods using the Normal, Log Normal and Log Pearson Type (111) distributions.

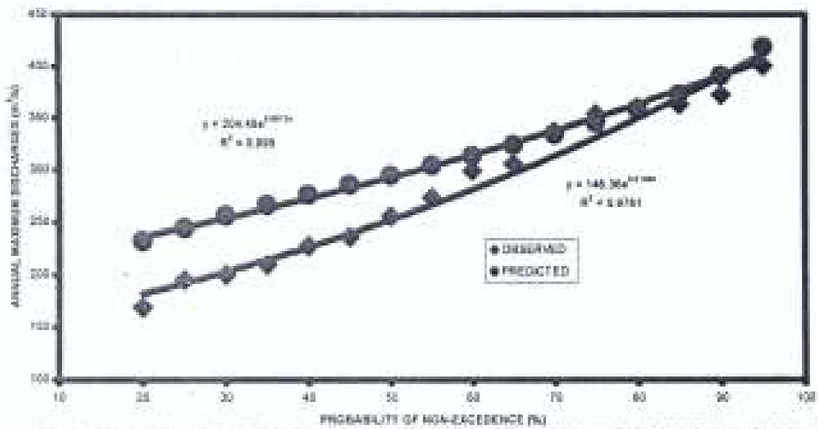

FIG 4.5a NORMAL DISTRIBUTION FITTED TO THE ANNUAL MAXIMUM DISCHARGES OF RIVER OSUN AT APOJE STATION USING WEIBULL'S PLOTTING POSITION

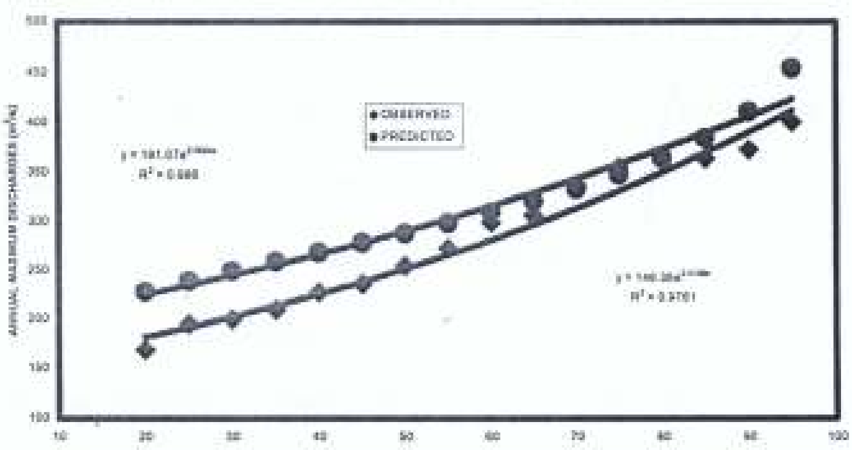

FIG 4.5b LOG NORMAL DISTRIBUTION FITTED TO ANNUAL MAXIMUM DISCHARGES OF RIVER OSUN AT APOJE STATION USING WEIBULL'S PLOTTING POSITION

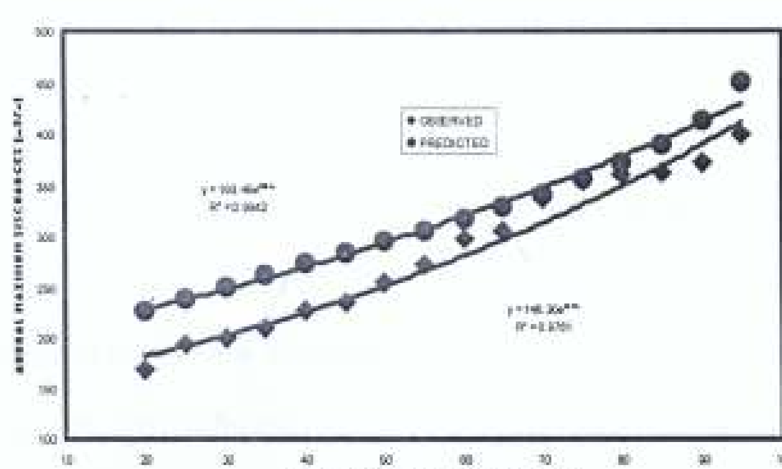

FIG 4.5c LOG PEARSON TYPE (111) DISTRIBUTION FITTED TO ANNUAL MAXIMUM DISCHARGES OF RIVER OSUN AT APOJE STATION USING WEIBULL,S PLOTTING POSITION

Fig 4.5a shows the graph of the normal distribution for the observed and the predicted discharges of River Osun at Apoje gauging station. From this figure, it can be seen that at the return periods of 25, 50 and 100 years, the observed discharges were 417, 426 and 431m^3/s whiles at the same return periods, the predicted discharges were 408, 414 and 417m^3/s respectively. A coefficient of correlation of 0.98 was obtained for the observed discharges, while for the predicted discharges, it was 0.99. This indicates that the predicted discharges were more correlated in this distribution than the observed discharges.

Fig 4.5b shows the graph of the Log Normal distribution for the observed and the predicted discharges of River Osun at Apoje gauging station. At return periods of 25, 50 and 100 years, the observed discharges were 417, 426 and 431m^3/s respectively while predicted discharges were 428, 435 and 439m^3/s respectively for the same return periods. The coefficients of correlation for the observed discharges was 0.98 while for the predicted discharges, it was 0.99. This shows that the predicted discharges in this distribution are more correlated than the predicted discharges.

Fig 4.5c shows the graph of the Log Pearson Type (111) distribution for the observed and the predicted discharges of River Osun at Apoje gauging station. The corresponding values at 25,50 and 100 years return periods for the observed discharges were 417, 426 and 431 m^3/s and for the predicted discharges were 433, 440 and 444 m^3/s respectively. The coefficients of correlation were 0.98 for the observed and 0.99 for the predicted discharges respectively. When compared, the predicted discharges obtained from the Log Pearson Type (111) distribution were smaller than those obtained from the Log Normal distribution at the return periods of 10 years and above. However, the predicted discharges under the Normal distribution compare favourably with those under the LogPearson Type (iii) distribution. Therefore, using the Weibull's plotting position for the three distributions for the stream flows at Apoje sub-basin, the predicted discharges under the Log Pearson Type (111) distribution are higher in magnitude and can be taken as the most reliable at any desired return periods.

4.8 BLOM'S PLOTTING POSITION.

Appendix B3 contains the predicted floods using the Normal, Log Normal and Log Pearson type (111) distributions.

Fig 4.6a shows the graph of the Normal distribution for the observed and the predicted discharges of River Osun at Apoje gauging station. From this figure, it can be seen that at the return periods of 25, 50 and 100 years, the observed discharges were 409, 422 and 427m^3/s while at the same return periods, the predicted discharges were, 413 and 419 and 422 m^3/s respectively. The coefficients of correlation of 0.98 was obtained for the observed discharges, while for the predicted discharges, it was 0.99.

Fig 4.6b shows the graph of the Log Normal distribution for the observed and the predicted discharges of River Osun at Apoje gauging station. At return periods of 25, 50 and 100 years, the observed discharges were 409, 442 and 427m3/s respectively while predicted discharges were 409, 439 and 443m^3/s respectively for the same return periods. The coefficients of correlation for the observed discharges was 0.98 while for the predicted discharges, it was 0.98. In this distribution, the predicted and the observed discharges are well correlated.

Fig 4.6c shows the graph of the Log Pearson Type (111) distribution for the observed and the predicted discharges of River Osun at Apoje gauging station. The

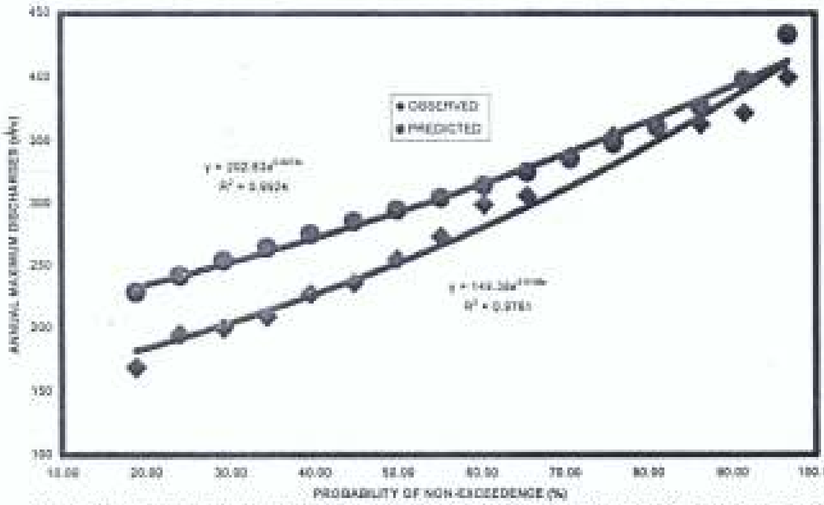

FIG 4.4a NORMAL DISTRIBUTION FITTED TO ANNUAL MAXIMUM DISCHARGES OF RIVER OSUN AT APOJE STATION USING BLOM'S PLOTTING POSITION.

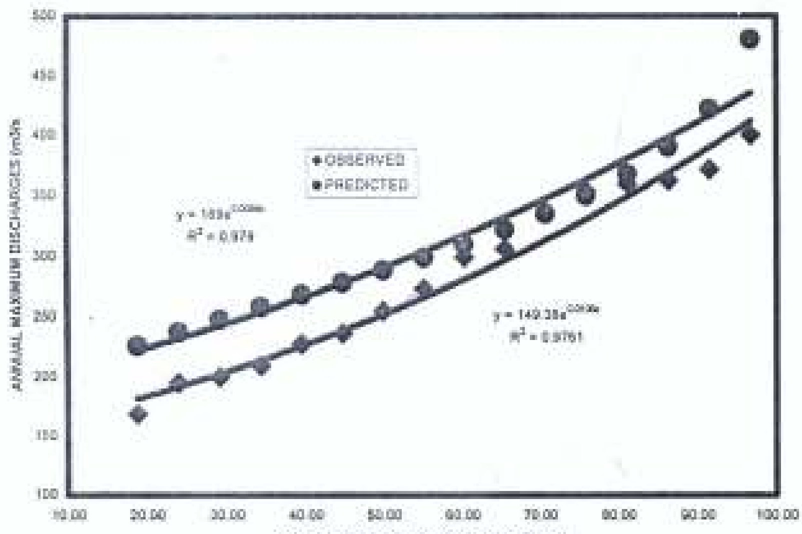

FIG 4.4b LOG NORMAL DISTRIBUTION FITTED TO ANNUAL MAXIMUM DISCHARGES OF RIVER OSUN AT APOJE STATION USING BLOM'S PLOTTING POSITION

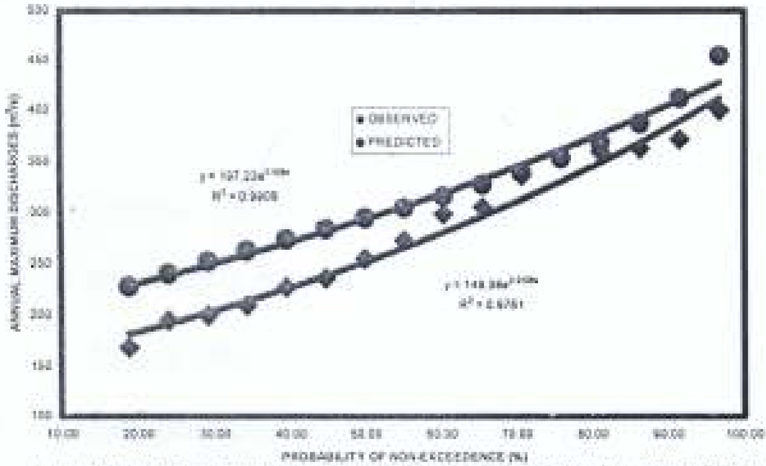

FIG 4.4c LOG PEARSON TYPE (111) DISTRIBUTION FITTED TO ANNUAL MAXIMUM DISCHARGES OF RIVER OSUN AT APOJE STATION USING BLOM'S PLOTTING POSITION

corresponding values at 25, 50 and 100 years return periods for the observed discharges were 409, 422 and 427 m^3/s and for the predicted discharges 425, 434 and 436 m^3/s respectively. The coefficients of correlation were 0.98 for the observed and 0.99 for the predicted. At return periods of 7 years and above, the predicted discharges in Fig 4.6c are lower than those in Fig 4.6b. At return periods of 2 years and above, predicted discharges in Fig 4.6c compares favourably with those in Fig 4.6a. The predicted discharges under the Log Pearson Type (111) distribution are higher than those under the Normal distribution. Therefore, using the Blom's plotting position for the three distributions for the stream flows at Apoje sub-basin, the Log Pearson Type (111) distribution is selected as the most reliabl at any return periods.

4.9 CUNNANE'S PLOTTING POSITION.

Appendix B3 contains the predicted floods using the Normal, Log Normal and Log Pearson type (111) distributions

Fig 4.7a shows the graph of the Normal distribution for the observed and the predicted discharges of River Osun at Apoje gauging station. From this Figure, it can be seen that at the return periods of 25, 50 and 100 years, the observed discharges were 410, 419 and 423m^3/s while at the same return periods, the predicted discharges were, 412 and 418 and 421 m^3/s respectively. Coefficients of Correlation of 0.98 was obtained for the observed discharges, while for the predicted discharges, it was 0.99.

Fig 4.7b shows the graph of the Log Normal distribution for the observed and the predicted discharges of River Osun at Apoje gauging station. At return periods of 25, 50 and 100 years, the observed discharges are 410, 419 and 423m^3/s respectively while predicted discharges were 435, 443 and 447m^3/s respectively for the same return periods

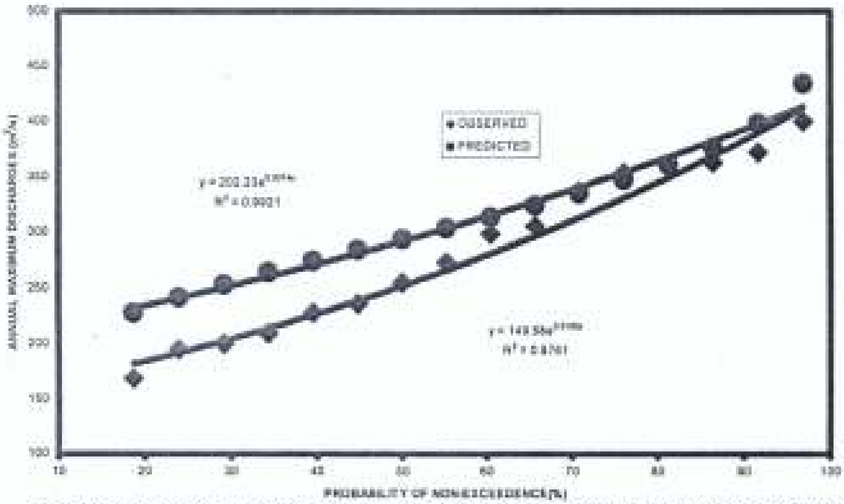

FIG 4.7a NORMAL DISTRIBUION FITTED TO THE ANNUAL MAXIMUM DISCHARGES OF RIVER OSUN AT APOJE STATION USING CUNNANE'S PLOTTING POSITION

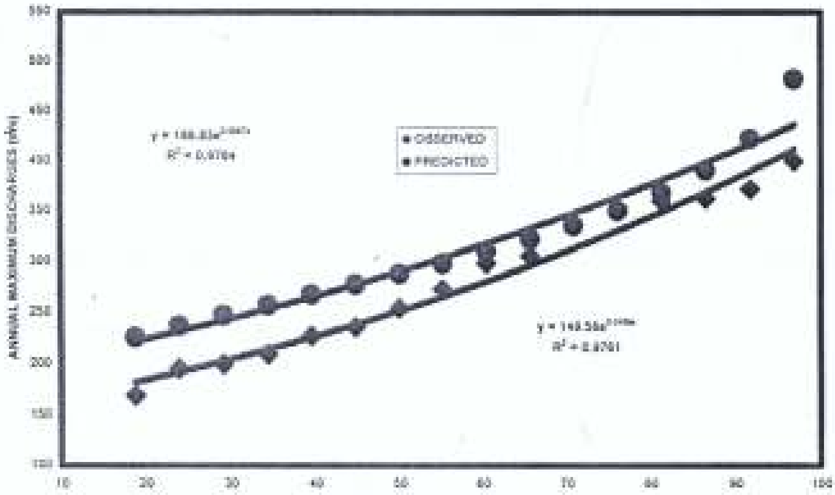

FIG. 4.7b LOG NORMAL DISTRIBUTION FITTED TO THE ANNUAL MAXIMUM DISCHARGES OF RIVER OSUN AT APOJE STATION USING CUNNANE'S PLOTTING POSITION

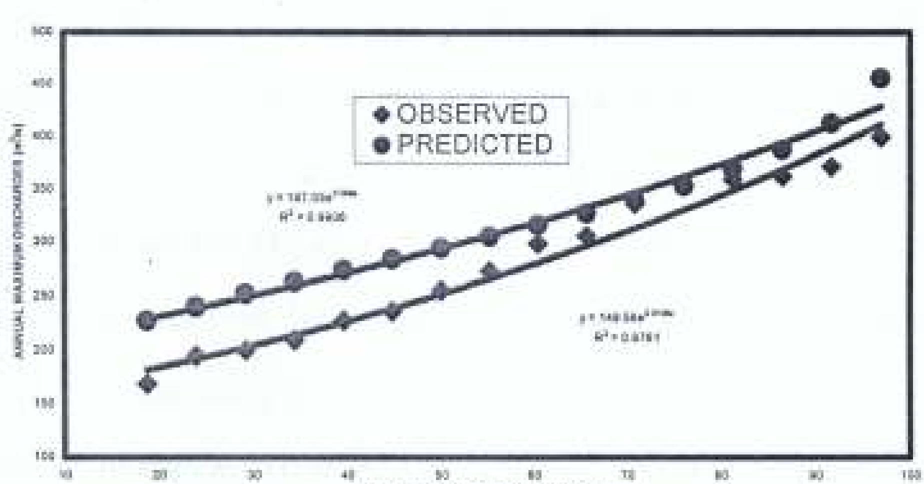

FIG 4.7c LOG PEARSON TYPE (111) ISTRIBUTION FITTED TO ANNUAL MAXIMUM DISCHARGES OF RIVER OSUN AT APOJE STATION USING CUNNANE'S PLOTTING POSITION

The coefficients of correlation for the observed discharges was 0.98 while for the predicted discharges, it was 0.98. Here the predicted and the observed discharges are well correlated.

Fig 4.7c shows the graph of the Log Pearson Type (111) distribution for the observed and the predicted discharges of River Osun at Apoje gauging station. The corresponding values at 25, 50 and 100 years return periods for the observed discharges were 410, 419 and 423 m^3/s and for the predicted discharges were 425, 432 and 435 m^3/s respectively. The coefficients of correlation were 0.98 for the observed and 0.99 for the predicted. At return periods of 7 years and above, the predicted discharges obtained from the Log Pearson Type (111) distribution were smaller than those obtained from the Log Normal distribution. However at return periods of 2 years and below, the predicted discharges in Fig 4.7 and 4.7a compares favourably. The Log Pearson Type (111) distribution produced predicted discharges of higher magnitudes than those of Normal distribution though with the same R^2 values. Therefore, the Log Pearson Type (111) distribution using Cunnane's plotting positionis hereby taken as the most dependable.

4.10 CALIFORNIA PLOTTING POSITION.

Appendix B5 contains the predicted floods using the Normal, Log Normal and Log Pearson type (111) distributions. Fig. 4.8a shows the graph of the Normal distribution for the observed and the predicted discharges of River Osun at Apoje gauging station. From this Figure, it can be seen that at the return periods of 25, 50 and 100 years, the observed discharges are 415, 424 and 429m^3/s while at the same return periods, the predicted discharges are, 407 and 413 and 416 m^3/s respectively. A coefficient of correlation of 0.98 was obtained for the observed discharges, while

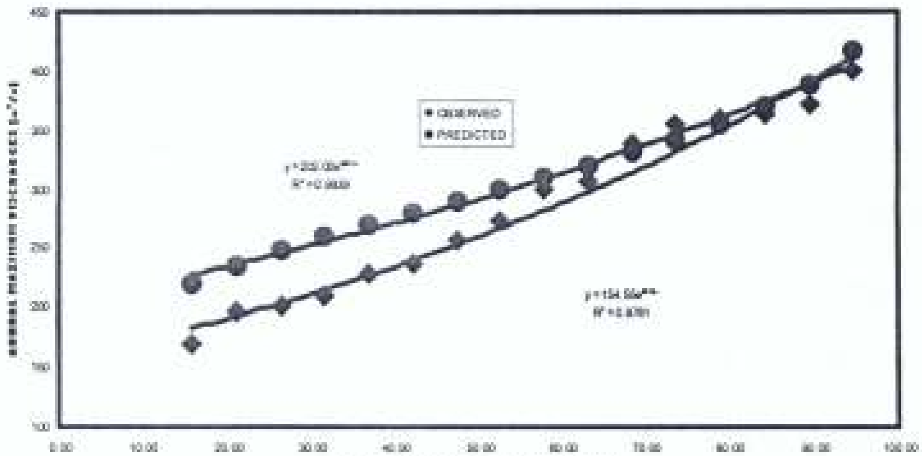

FIG 4.8a NORMAL DISTRIBUTION FITTED TO ANNUAL MAXIMUM DISCHARGES OF RIVER OSUN AT APOJE STATION USING CALIFORNIAL PLOTTING POSITON

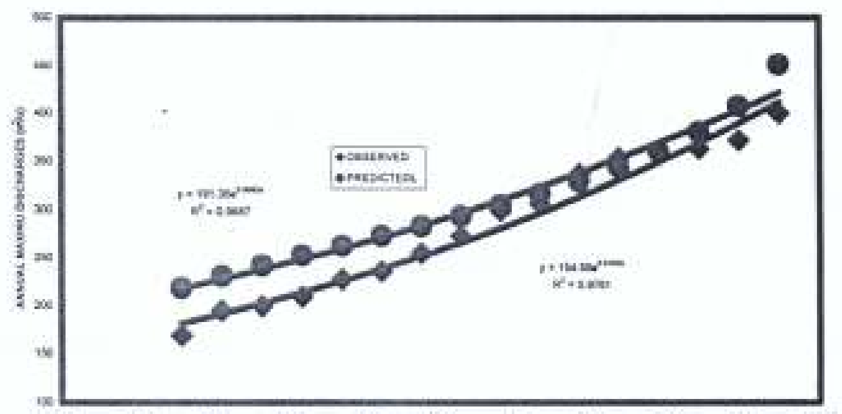

FIG 4.8b LOGNORMAL DISTRIBUTION FITTED TO ANNUAL MAXIMUM DISCHARGES OF RIVER OSUN AT APOJE STATION USING CALIFORNIAL PLOTTING POSITION

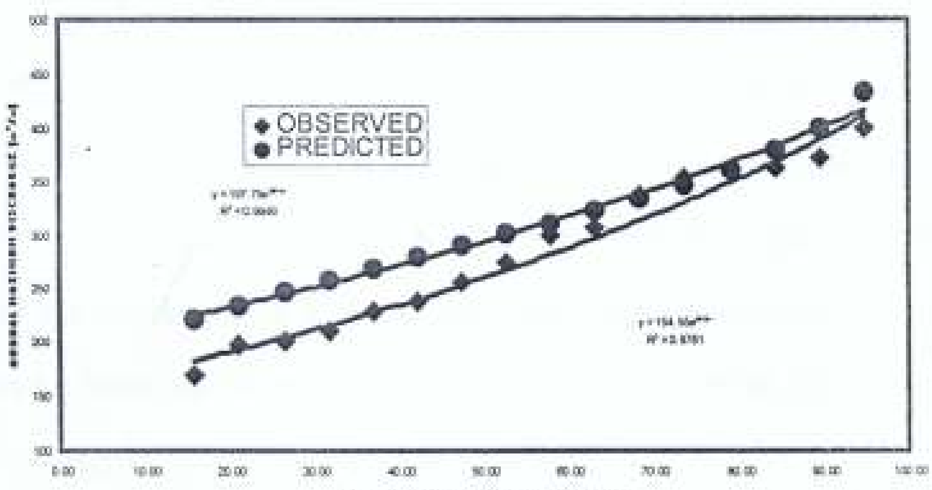

FIG 4.8c LOG PEARSON TYPE (111) DISTRIBUTION FITTED TO ANNUAL MAXIMUM DISCHARGES OF RIVER OSUN AT APOJE STATION USING CALIFORNIA FORMULA

for the predicted discharges, it was 0.99. The high values of (R^2) indicate that the predicted and the observed discharges are well correlated.

Fig 4.8b shows the graph of the Log Normal distribution for the observed and the predicted discharges of River Osun at Apoje gauging station. At return periods of 25, 50 and 100 years, the observed discharges were 436, 448 and 453m^3/s respectively while predicted discharges were 455, 465 and 470m^3/s respectively for the same return periods. The coefficients of determination for the observed discharges was 0.86 while for the predicted discharges, it was 0.99. Under this distribution, the predicted discharges are more correlated than the observed discharges.

Fig 4.8c shows the graph of the Log Pearson Type (111) distribution for the observed and the predicted discharges of River Osun at Apoje gauging station. The corresponding values at 25, 50 and 100 years return periods for the observed discharges were 415, 424 and 429m^3/s and for the predicted discharges were 422, 429 and 432m^3/s respectively. The coefficients of correlation were 0.99 for the predicted and 0.98 for the observed discharges. From Fig 4.8c, the predicted discharges were smaller in magnitudes than those obtained from the Log Normal distribution in Fig. 4.8c at return periods of 6 years and above. At return periods of about 2 years and below, the predicted discharges in Fig 4.8c compares favourably with those in Fig 4.8a. Therefore, using the California's plotting position for the three distributions for the stream flows at Apoje sub-basin the Log Pearson Type (111) distribution can be taken as the most reliable since it produced the highest value of fit at any desired return periods.

4.11 GRINGORTON'S PLOTTING POSITION.

Appendix B6 contains the predicted floods using the Normal, Log Normal and Log Pearson type (111) distributions.

Fig 4.9a shows the graph of the normal distribution for the observed and the predicted discharges of River Osun at Apoje gauging station. From this figure, it can be seen that at the return periods of 25, 50 and 100 years, the observed discharges were 407, 415 and 420m^3/s while at the same return periods, the predicted discharges were, 411 and 417 and 420 m^3/s respectively. Coefficients of correlation of 0.99 was obtained for the observed discharges, while for the predicted discharges, it was 0.99. The high coefficient of correlation indicates that the predicted and the observed discharges are well fitted to this distribution.

Fig 4.9b shows the graph of the Log Normal distribution for the observed and the predicted discharges of River Osun at Apoje gauging station. At return periods of 25, 50 and 100 years, the observed discharges were 407, 415 and 420m^3/s respectively while predicted discharges were 411, 417 and, 420m^3/s respectively for the same return periods. The coefficients of correlation for the observed discharges was 0.98 while for the predicted discharges, it was 0.99.

Fig 4.9c shows the graph of the Log Pearson Type (111) distribution for the observed and the predicted discharges of River Osun at Apoje gauging station. The corresponding values at 25, 50 and 100 years return periods for the observed discharges were 407, 415 and 420 m^3/s and for the predicted discharges were 435, 442 and 446 m^3/s respectively. The coefficients of correlation were 0.98 for the predicted and 0.98 for the observed discharges. From Fig 4.9c, the predicted discharges obtained from the Log Pearson Type (111) distribution were smaller than those obtained from the Log Normal distribution at return periods of 5 years and

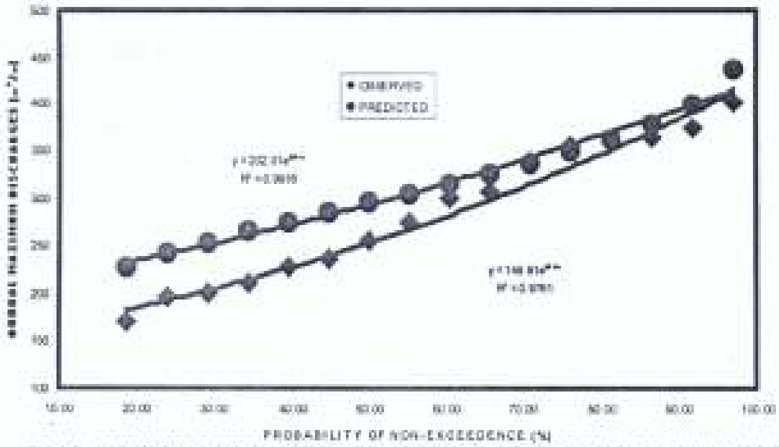

FIG 4.9a NORMAL DISTRIBUTION FITTED TO ANNUAL MAXIMUM DISCHARGES OF RIVER OSUN AT APOJE STATION USING GRINGORTON FORMULA

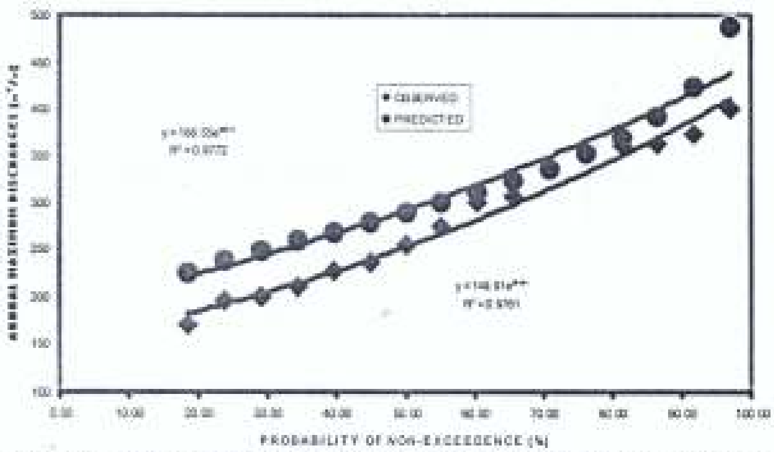

FIG 4.9b LOG NORMAL DISTRIBUTION FITTED TO ANNUAL MAXIMUM DISCHARGES OF RIVER OSUN AT APOJE STATION USING GRINGORTON FORMULA

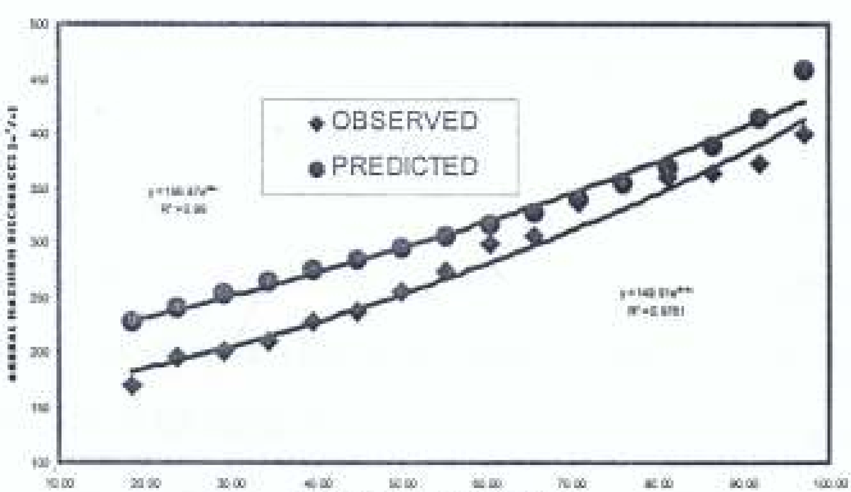

FIG 4.9c LOG PEARSON TYPE (111) DISTRIBUTION FITTED TO ANNUAL MAXIMUM DISCHARGES OF RIVER OSUN AT APOJE STATION USING GRINGORTON FORMULA

above. But there was a good comparison between the predicted discharges in Fig 4.9c and Fig 4.9a at return periods of 2 years and below. The highest coefficient of determination was obtained for the predicted discharges under the Log Pearson Type (111) distribution. Therefore, using the Gringorton's plotting position for the three distributions for the stream flows at Apoje sub-basin, the Log Pearson Type (111) distribution can be taken as the most reliable since it produced the highest value of fit at any desired return periods.

4.12 CHEGODAJEV'S PLOTTING POSITION.

Appendix B6 contains the predicted floods using the Normal, Log Normal and Log Pearson type (111) distributions

Fig 4.10a shows the graph of the Normal distribution for the observed and the predicted discharges of River Osun at Apoje gauging station. At the return periods of 25, 50 and 100 years, the observed discharges were 412, 420 and 425m^3/s while at the same return periods, the predicted discharges were, 409 and 415 and 418 m^3/s respectively. Coefficients of correlation of 0.98 was obtained for the observed discharges, while for the predicted discharges, it was 0.99.

Fig 4.10b shows the graph of the Log Normal distribution for the observed and the predicted discharges of River Osun at Apoje gauging station. At return periods of 25, 50 and 100 years, the observed discharges were 412, 420 and, 4425m^3/s respectively while predicted discharges were 433, 440 and, 444m^3/s respectively for the same return periods. The coefficients of determination for the observed discharges was 0.98 while for the predicted discharges, it was 0.99. On this distribution, the predicted discharges are more correlated than the observed discharges.

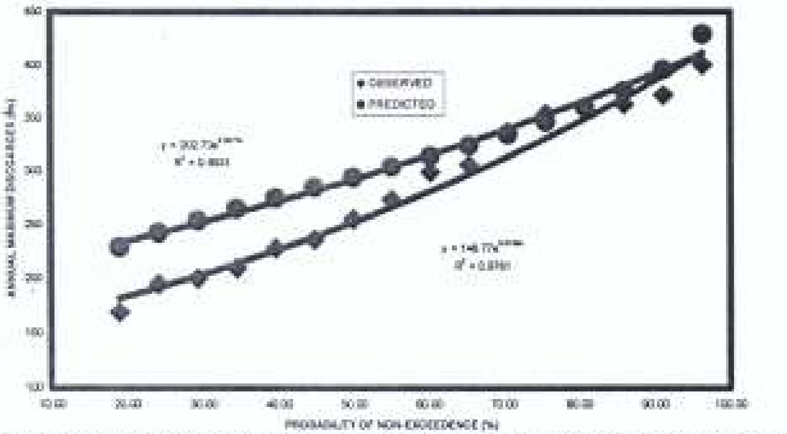

FIG 4.10a NORMAL DISTRIBUTION FITTED TO ANNUAL MAXIMUM DISCHARGES OF RIVER OSUN AT APOJE STATION USING CHEGODAJEV FORMULA

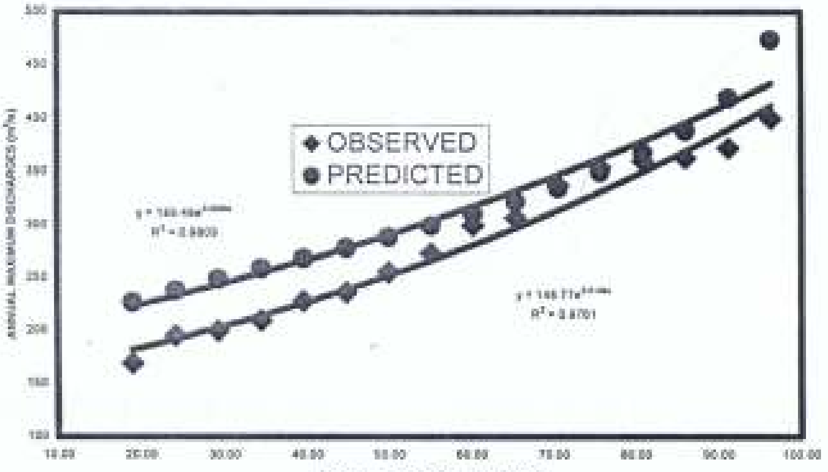

FIG 4.10b LOG NORMAL DISTRIBUTION FITTED TO ANNUAL MAXIMUM DISCHARGES OF RIVER OSUN AT APOJE STATION USING CHEGODAJEV FORMULA.

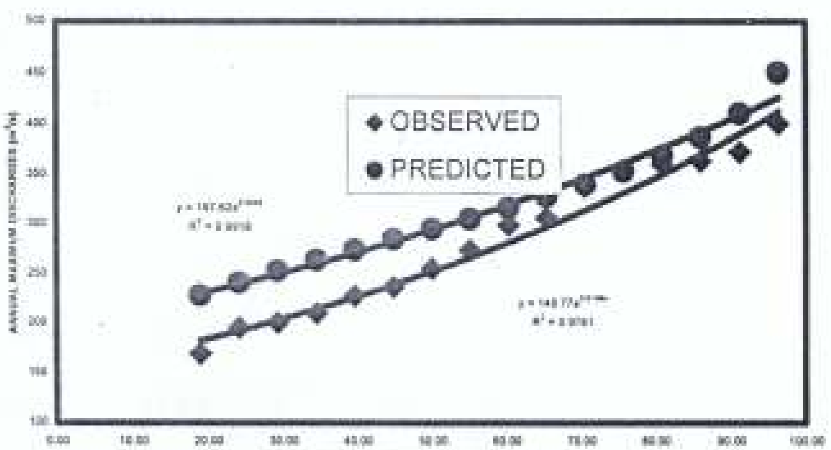

FIG. 4.10c LOG PEARSON TYPE (111) DISTRIBUTION FITTED TO ANNUAL MAXIMUM DISCHARGES OF RIVER OSUN AT APOJE STATION USING CHEGODAJEV'S FORMULA

Fig 4.10c shows the graph of the Log Pearson Type (111) distribution for the observed and the predicted discharges of River Osun at Apoje gauging station. The corresponding values at 25, 50 and 100 years return periods for the observed discharges were 412, 420 and 4425 m^3/s and for the predicted discharges were 426, 432 and 436 m^3/s respectively. The coefficients of determination were 0.98 for the predicted and 0.99 for the observed discharges. From Fig 4.10c, the predicted discharges obtained from the Log Pearson Type (111) distribution were smaller than those obtained from the Log Normal distribution at return periods of 5 years and above. The highest coefficient of determination was obtained for the predicted discharges under the log Pearson Type (111) distribution. The Log Pearson Type (111) distribution using the Chegodajev plotting position produces the predicted discharges of higher magnitudes and is therefore taken as the most reliable.

Table 4.5 Plotting Positions and their Coefficients of Determination.

Plotting Positions	Distributions			Remark
	Normal	Log Normal	Log Pea.Type(111)	
Hazen	0.99	0.98	0.99	Well correlated
Weibull	0.99	0.99	0.99	Well correlated
Blom	0.99	0.98	0.99	Well correlated
Cunnane	0.99	0.98	0.99	Well correlated
California	0.99	0.98	0.99	Well correlated
Gringorton	0.99	0.98	0.99	Well correlated
Chegodayev	0.99	0.98	0.99	Well correlated

Based on the comparison made between the frequency curves fitted by Normal, Log Normal and Log Pearson Type (III) distributions, it is evident that the higher the return periods, the lower the differences between the predicted and the observed annual maximum discharges. Table 4.5 shows that all the plotting positions and the probability distributions gave a good fit of the annual maximum discharges of River Osun at Apoje gauging station. Also, the highest coefficient of correlation R^2 of 0.99 was estimated for the predicted annual maximum discharges using the Normal and Log Pearson Type (111) distributions. The Log Pearson Type (111) distribution fitted to the systematic annual maximum discharges of River Osun at Apoje gauging station because the predicted discharges are higher than those of Normal distribution at the same return periods. This is in compliance with (Alatise, 1998; Chen *et al*, 2002). In the light of this, the Log Pearson Type (111) distribution is recommended as the probability distribution to be used for the flood frequency analyses of River Osun at Apoje gauging station.

4.13 THE UPPER AND LOWER FLOOD LIMITS.

Reliability is the probability that a system will perform specific functions under some conditions for a specific period of time. In order to determine the upper and the lower flood limits, a reliability test at 95% confidence interval was carried out on the peak annual discharges of River Osun at Apoje station. The frequency curve of the reliability tests is as shown in Fig. 4.11.

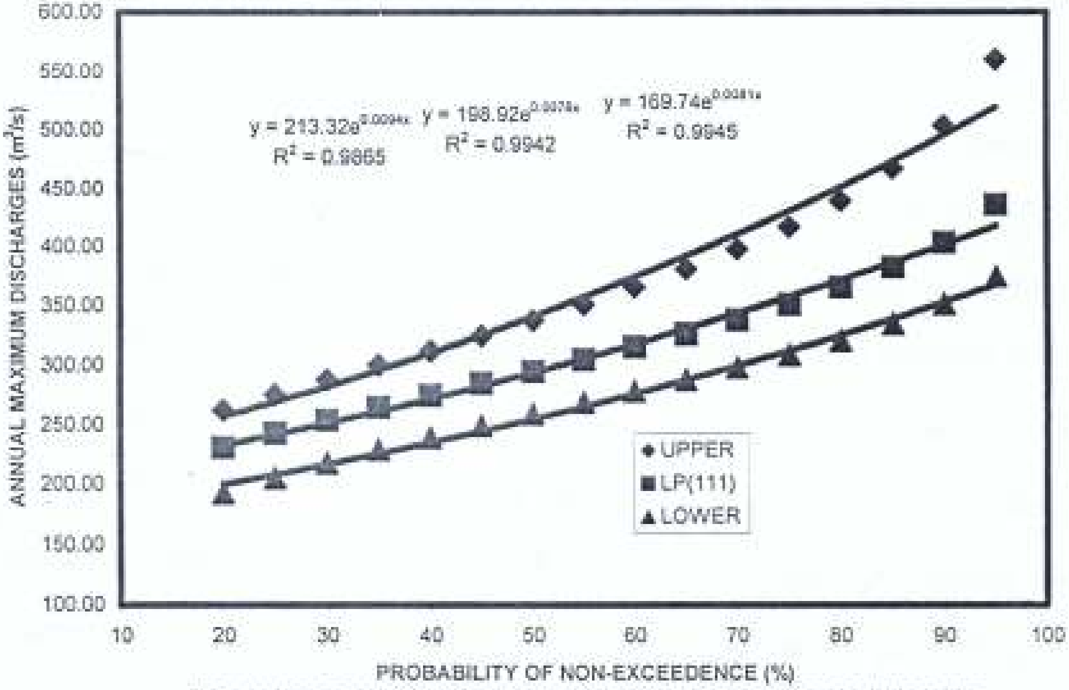

FIG 4.11 UPPER AND LOWER FLOOD LIMITS AT 95% CONFIDENCE INTERVER.

Appendix C contains the Upper and Lower discharge limits predicted using equations **2.26** and **2.27**. Fig 4.11 shows the graphical illustration of the upper and the lower discharges with reference to the control discharges predicted using the Log Pearson Type (111) distribution. At return periods of 25, 50 and 100 years, the upper discharges were 526, 536 and 541m^3/s respectively. The control discharges at these return periods were 421, 427and 431m^3/s. Also Fig. 4.11 shows that the higher the return periods, the higher the divergence of the upper discharge limits from the control discharges, unlike the lower discharges limits which maintains a relatively constant differences at return periods of 25, 50 and 100 years. The upper discharge limits are higher and therefore can be relied upon for the design of hydraulic structures at Apoje sub-basin.

4.14 RESERVOIR CAPACITIES OF RIVER OSUN AT APOJE SUB-BASIN IN 1982 AND 1983.

Fig 4.12 shows the mass curve of River Osun at Apoje gauging station in 1982. The annual cumulative volume of water was 82.20 x 10^9m^3 (see Appendix D1). The rate of withdrawal per month was 7.26 x 10^9m^3 and is represented by the straight line joining April and December. The coefficient of variation of the monthly total volume was 0.36. The reservoir capacity in both May and June was 1800, million m^3 water while in August and September, the reservoir capacities were 2600 and 3600 million m^3 of water respectively. Spillage began in October. The spillway capacities required in both October and November was 3,100 million m^3 of water. The maximum reservoir capacity required in 1982 water year was determined by adding the reservoir capacity of 3,600 million m^3 of water in September and spillway capacity of 3100 million m^3 of water in November which gave 6,700 million m^3 of water. This shows that the maximum reservoir capacity in 1982 could not meet the monthly demand and this will lead to shortage in water supply.

Fig 4.13 shows mass curve of River Osun at Apoje gauging station. The annual cumulative inflow was 88.91 x 10^9 m^3 of water (see Appendix D2), while the rate of monthly withdrawal was 7.41x10^9m^3 and was represented by the straight line joining January to December. The coefficient of variation of the monthly total volumes was 0.28. From February to April, the reservoir capacities were 3100, 5200 and 5900 million m^3 of water respectively. From May to the required reservoir capacities were 7,300; 6,200 and 5,100 million m^3 of water respectively. The reservoir became full in September and spillage began in October. The spillway capacities required in October and November were 2,300 and 6.900 million m^3 of water. The maximum reservoir capacity required in 1983 water year was determined by adding the reservoir capacities of 7,300 million m^3 of water in June and spillway capacity of 6900 million m^3 water in November which gave 14,200 million m^3 of water.

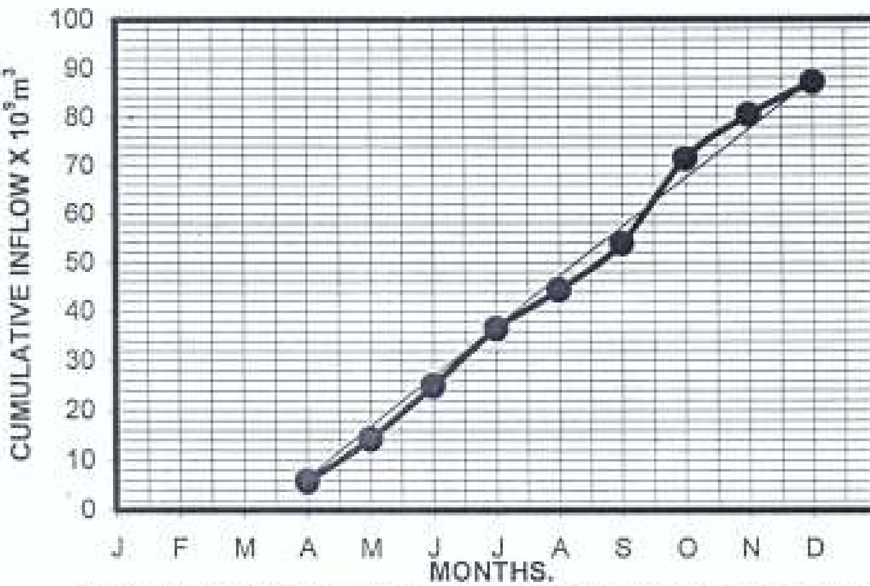

FIG 4.12 MASS CURVE OF RIVER OSUN AT APOJE GAUGING STATION (1982)

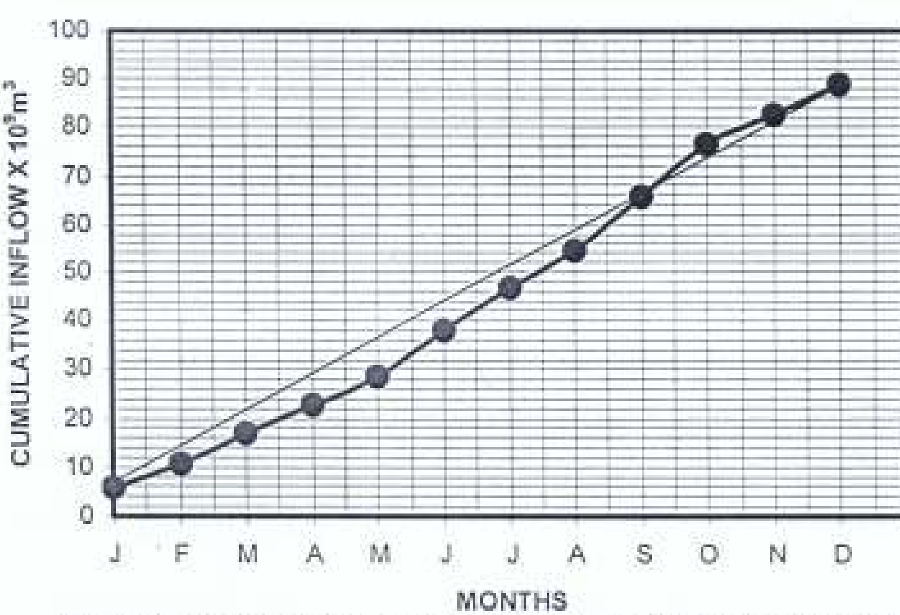

FIG 4.13 MASS CURVE OF RIVER OSUN AT APOJE GAUGING STATION (1983)

4.15 RESERVOIR CAPACITIES OF RIVER OSUN AT APOJE GAUGING STATION IN 1984 AND 1985.

Fig 4.14 shows the mass curve of River Osun Apoje gauging station. 1984 The annual cumulative volume of water for the year was monthly total volume 111.10 x $10^9 m^3$ of water (see Appendix D3). The rate of monthly withdrawal was by dividing 9.26 x $10^9 m^3$ of water. The coefficient of variation of the monthly total volume was 0.4. The reservoir capacities from February to May were 4900, 7200, 13,100 and 21,200 million m^3 of water respectively. However from June to August, the reservoir capacities were 14300, 11300 and 5500 million m^3 of water respectively. The reservoir became full in September and spillage began in October. The spillway capacities in October and November were 3800 and 2300 million m^3 of water respectively. The maximum reservoir capacity for the 1984 was determined by adding the reservoir capacity of 21,200 million m^3 of water in May and the spillway capacity of 3,800 million m^3 in October which gave 25000 million m^3 of water. This shows that the maximum reservoir capacity could support the rate of monthly withdrawal in 1984.

Fig 4.15 shows the mass curve of River Osun Apoje gauging station in 1985. The annual cumulative volume was 106.32 x $10^9 m^3$ of water (see Appendix D4). The rate of monthly withdrawal was 8.86 x $10^9 m^3$ of water and is represented by the straight line joining January and December. The coefficient of variation of the monthly total volume was 0.7 From February to May, the required reservoir capacities were 10200, 17600, 19800 and 23,200 million m^3 of water respectively, while from June to September, the reservoir capacities were 22900, 15200, 4400 and 7000 million m^3 of water respectively. In each month of the year, there would be shortage of supply. The maximum reservoir capacity for the 1985 was determined by adding the reservoir capacity of 23,200 million m^3 of water in May and the spillway capacity of 2400 million m^3 in October and this gave 25600 million m^3 of water. This shows that the maximum reservoir capacity can support the rate of monthly withdrawal.

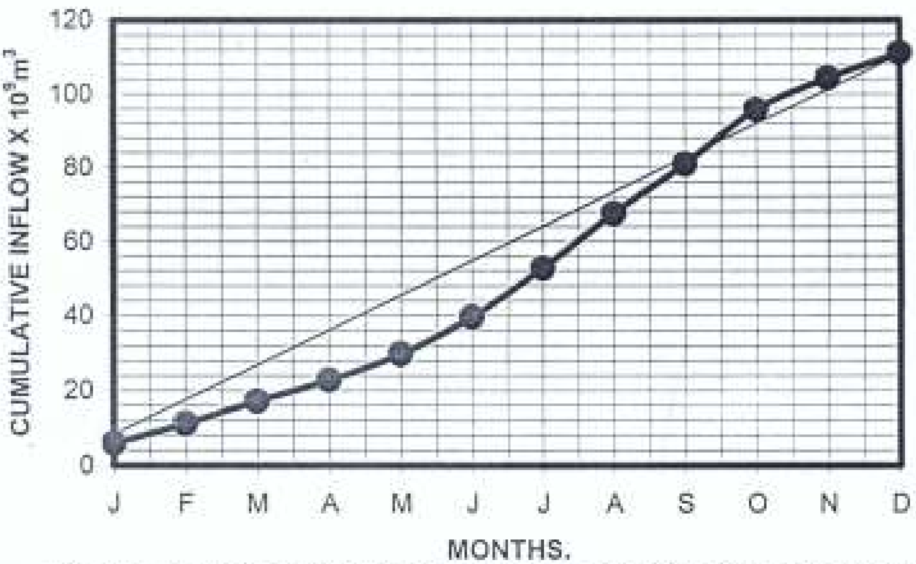

FIG 4.14 MASS CURVE OF RIVER OSUN AT APOJE GAUGING STATION (1984)

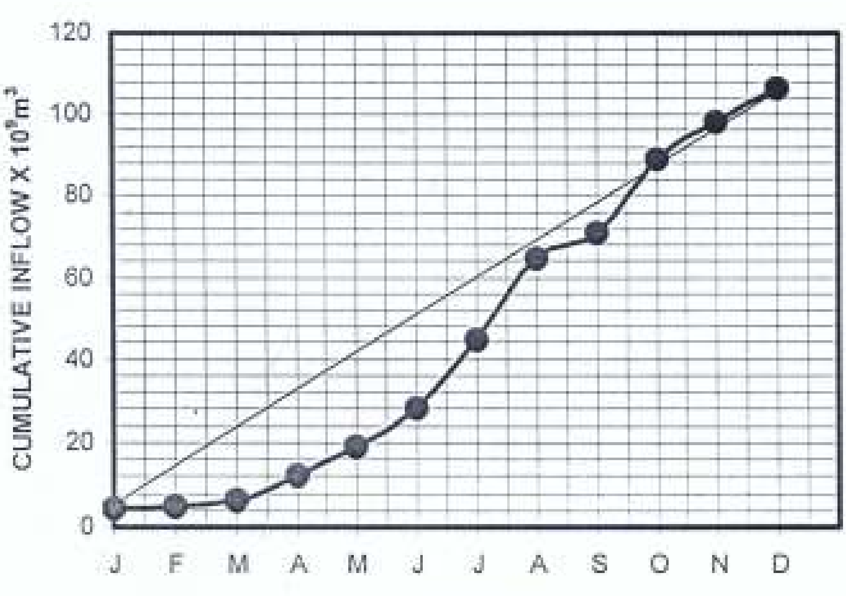

FIG 4.15 MASS CURVE OF RIVER OSUN AT APOJE GAUGING STATION (1985)

4.16 POTENTIAL RESERVOIR CAPACITIES OF RIVER OSUN AT APOJE STATION FROM 1986 TO 1999.

Table 4.6 shows the potential Reservoir capacities of River Osun at Apoje station between 1986 and 1999.

Fig 4.16 shows the mass curve in 1986. The coefficient of variation of the monthly total volume in (Appendix D5) was 0.53. There would be shortage of supply from February to May because the reservoir capacities in these months could not support the monthly demand. The reservoir was full in June and overflow began in July. The maximum reservoir capacity of 12,500 million m^3 of water was obtained by adding the reservoir capacity in May and spillway capacity in September and this would cope with the monthly demand in 1986.

Fig. 4.17 shows the mass curve in 1987. In February there was shortage and spillage began in November. The coefficient of variation the monthly total volume (Appendix D5) was 0.59. The maximum reservoir capacity of 25,800 million m^3 of water would adequately cope with the monthly demand in 1987.

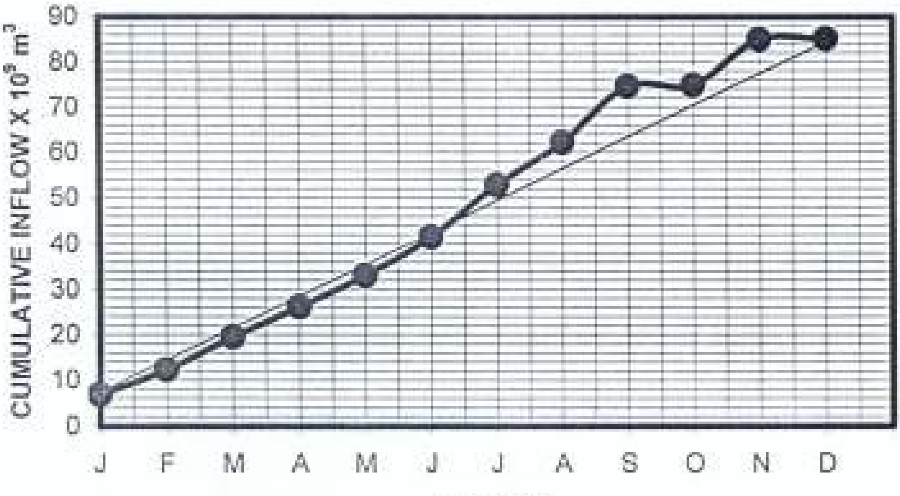

FIG 4.16 MASS CURVE OF RIVER OSUN AT APOJE GAUGING STATION (1986)

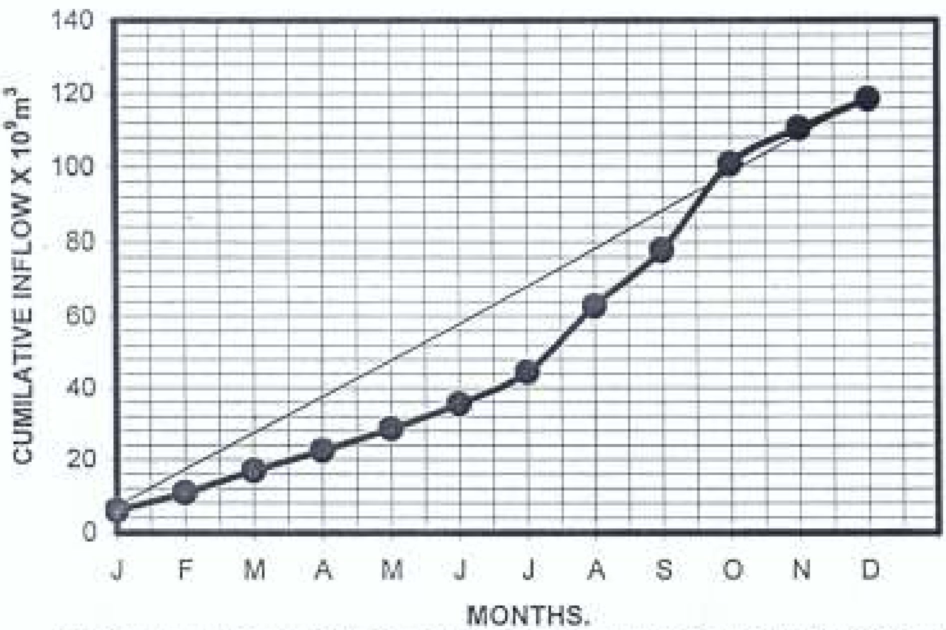

FIG 4.17 MASS CURVE OF RIVER OSUN AT APOJE GAUGING STATION (1987)

Fig 4.18 shows the mass curve in 1988 water year. The coefficient of variation of the monthly total volumes in Appendix D7 was 0.45. There was shortage of water supply in February to March and August to September. The reservoir was full in October and spillage began in November. The maximum February reservoir capacity of 20,300 million m^3 of water could meet the monthly demand in 1988 water year.

Fig 4.19 shows the mass curve in the year 1989. There was a shortage of supply in February, April and September. The reservoir was full in October spillage began in November. The maximum reservoir capacity of 23,500 million m^3 of water could meet the monthly demand in 1989. The coefficient of variation of the monthly total volume was 0.45 [Appendix D8]

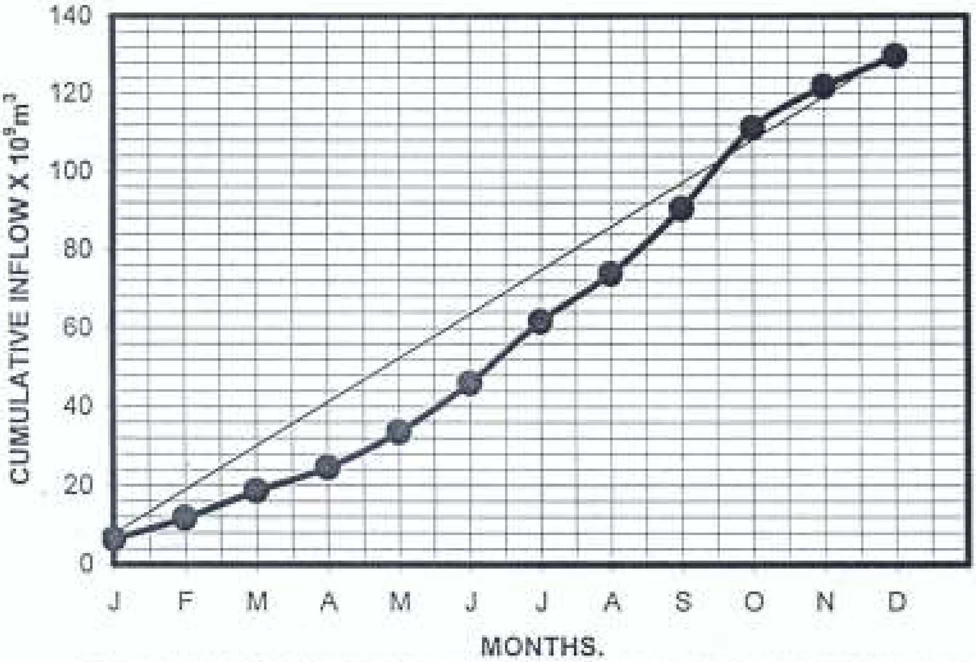

FIG 4.18 MASS CURVE OF RIVER OSUN AT APOJE GAUGING STATION (1988)

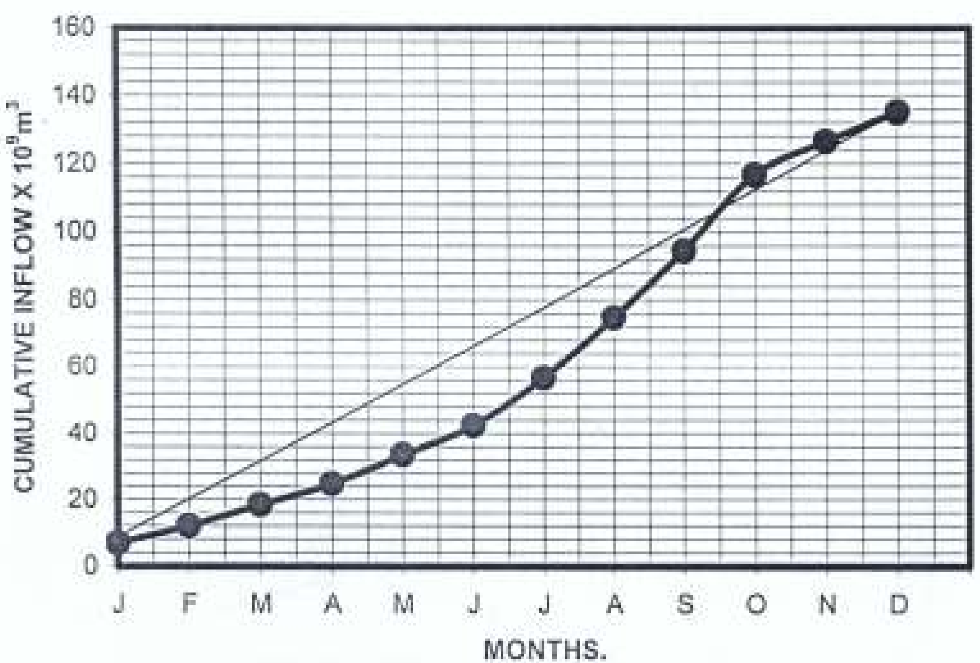

FIG 4.19 MASS CURVE OF RIVER OSUN AT APOJE GAUGING STATION (1989)

Fig 4.20 shows the mass curved in 1990 water year. In February, March and September there was shortage of water supply. The reservoir was full in October while the rate of monthly demand overlapped the cumulative inflow in November and December. The maximum reservoir capacity of 18,900million m^3 of water was adequate to cope with the monthly demand in 1990. The coefficient of variation of monthly total volume was 0.34 [Appendix D9]

Fig 4.21 shows the mass curve in 1991. It is evident that there was shortage of water supply from February to March. The reservoir was full in May while spillage began in June. The reservoir was full again in September while the rate of monthly withdrawal overlapped the cumulative inflow from October to December. The maximum reservoir capacity of 9,000 million m^3 of water could not meet the monthly demand in 1991 and therefore there was shortage of water throughout 1991. The monthly volumes had a coefficient of variation of 0.26.

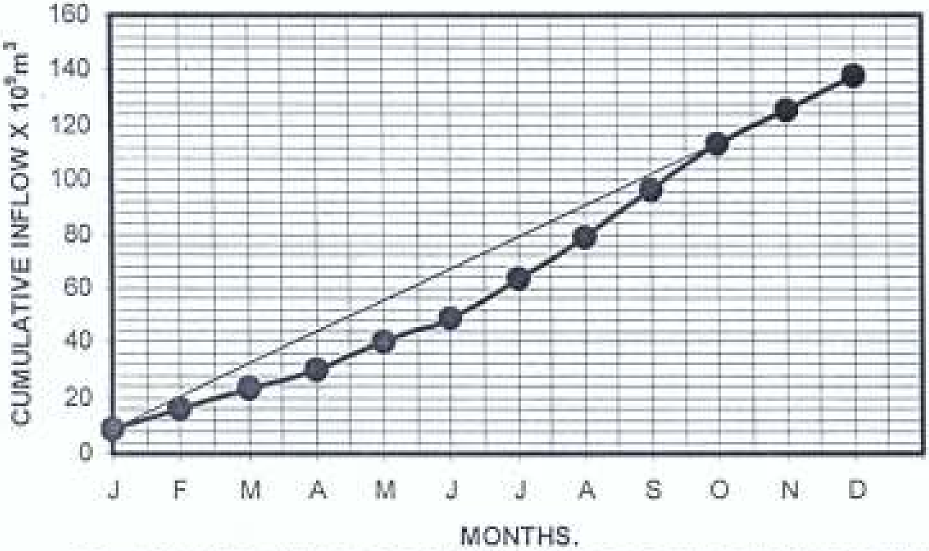

FIG 4.20 MASS CURVE OF RIVER OSUN AT APOJE GAUGING STATION (1990)

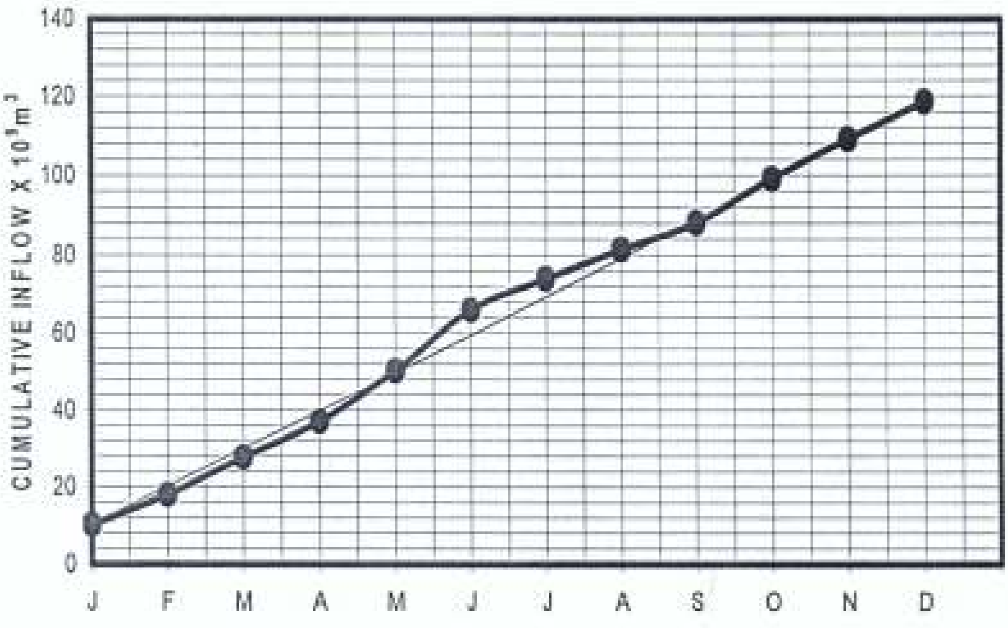

FIG 4.21 MASS CURVE OF RIVER OSUN AT APOJE GAUGING STATION (1991)

Fig 4.22 shows the mass curve in 1992. There were shortages of water supply from February to March and September to October. The reservoir was full in August and spillage began in November. The maximum reservoir capacity of 21,900 million m3 of water was adequate to cope with the monthly demand in 1992. The coefficient of variation of the monthly volumes was 0.41.

Fig. 4.23 shows the mass curve in 1993. There were shortages from February to April and July to October. The reservoir capacity of 17,100 million m3 of water could cope with the monthly demand in 1993. The coefficient of variation of the monthly volumes was 0.28.

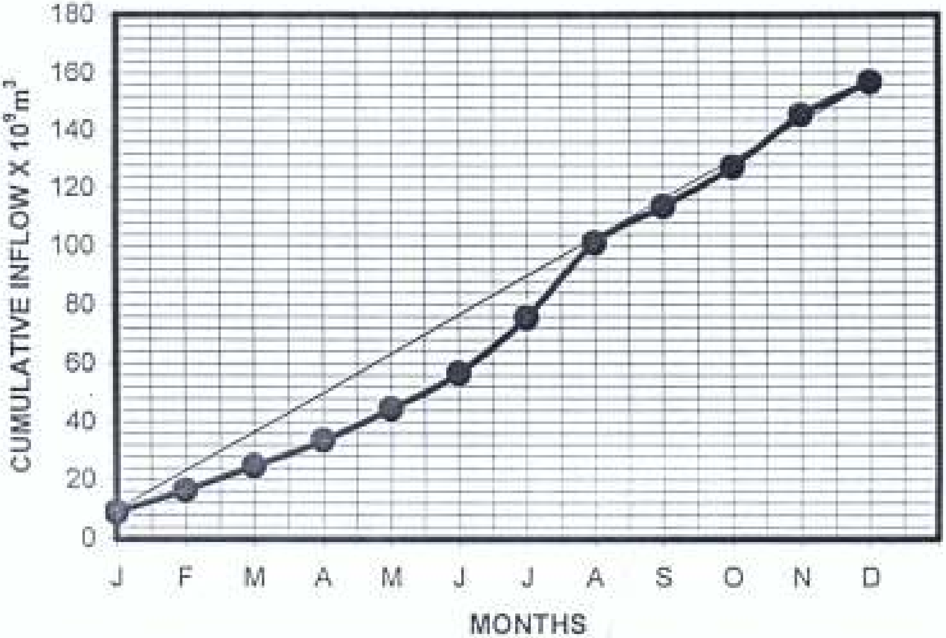

FIG. 4.22 MASS CURVE OF RIVER OSUN AT APOJE GAUGING STATION (1992)

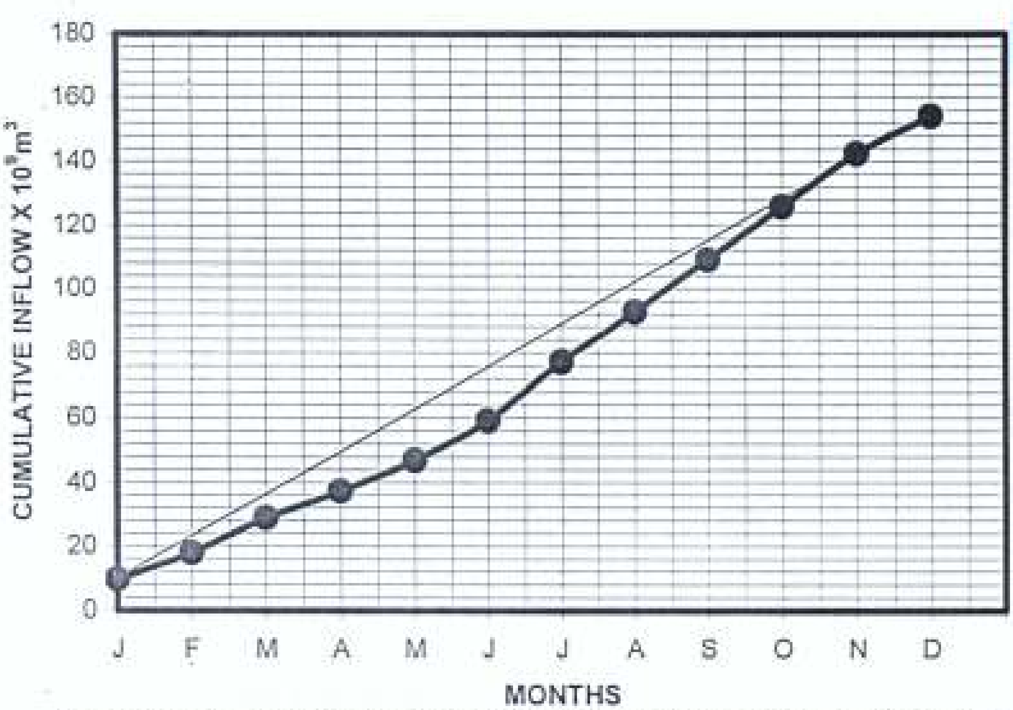

FIG 4.23 MASS CURVE OF RIVER OSUN AT APOJE GAUGING STATION (1993)

Fig. 4.24 shows the mass curve in 1994 water year. Water shortages were experienced from February to April and July. The reservoir was full in August and spillage began in October. The maximum reservoir capacity of 18.900 could support the monthly demand in 1994. The monthly volumes had the coefficient of variation of 0.3

Fig 4.25 shows the mass curve in 1995 water year. In 1995, there was water shortage in February and September because the reservoir capacities could not cope with the monthly demand. The reservoir was full in October while spillage began in November. The maximum reservoir capacity of 29,000 million m^3 of water was sufficient to supply the monthly demand in 1995. The coefficient of variation of the monthly total volumes in Appendix D14 was 0.58

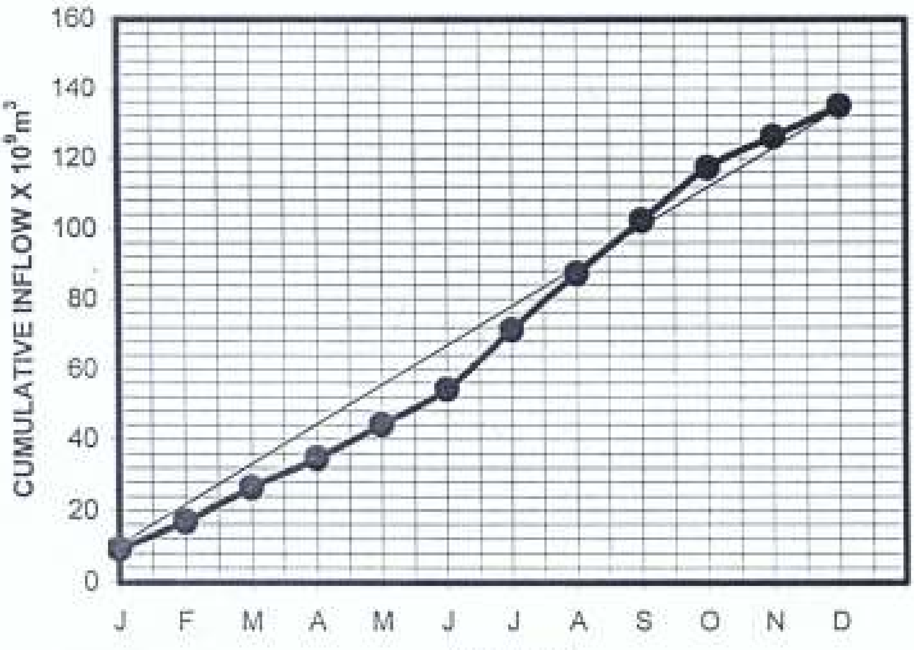

FIG 4.24 MASS CURVE OF RIVER OSUN AT APOJE GAUGING STATION (1994)

Fig 4.26 shows the mass curve in 1996. There was shortage of water supply in February and overflow began in November. The maximum reservoir capacity of 21,600 million m^3 of water was adequate to supply the monthly demand in 1996. The monthly total volumes (appendix D15) had a coefficient of variation of 0.47.

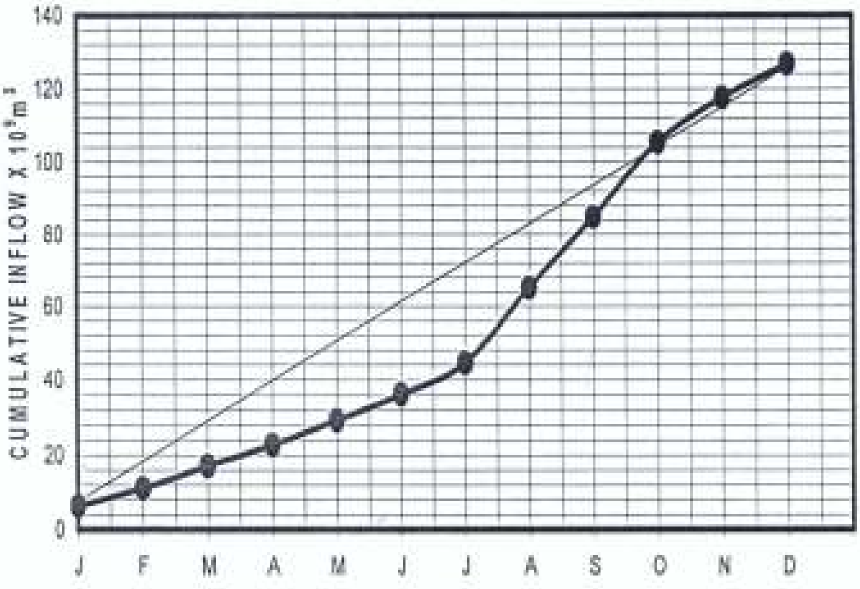

FIG 4.25 MASS CURVE OF RIVER OSUN AT APOJE GAUGING STATION (1995)

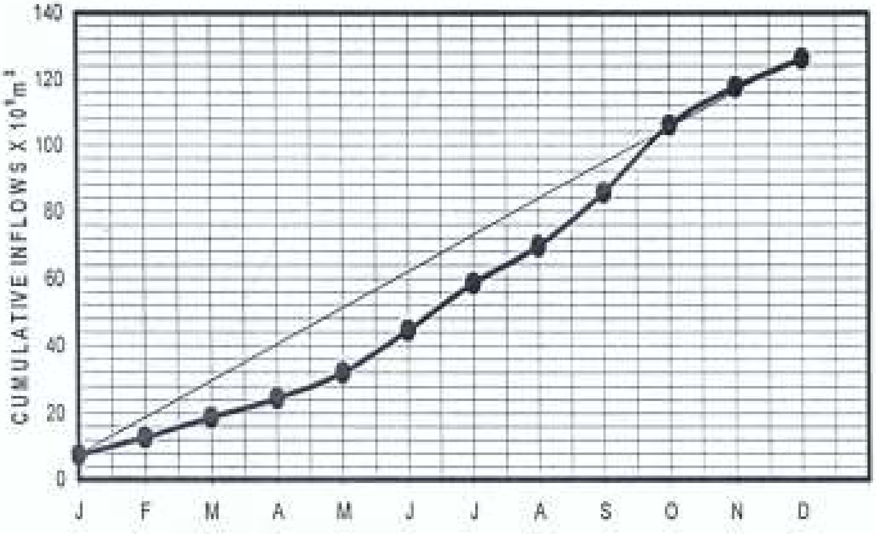

FIG 4.26 MASS CURVE OF RIVER OSUN AT APOJE GAUGING STATION (1996)

Fig 4.27 shows the mass curve in 1997. There was shortage of water supply from February to June because the reservoir capacities in those months could not support the monthly demand. The reservoir was full in July and spillage was experienced from August till November. The maximum reservoir capacity of 18,300 could not support the monthly demand so there was shortage of water supply throughout 1997. The coefficient of variation of the monthly total volume (Appendix D16) was 0.22.

Fig 4.28 shows the mass curve in 1998. In February, March and November, the reservoir capacities could not support the monthly demand; therefore there was shortage of water supply. The maximum reservoir capacity of 21,200 million m^3 of water was sufficient to supply the monthly demand of water in 1998. The monthly volumes in Appendix D17 had a coefficient of variation of 0.85.

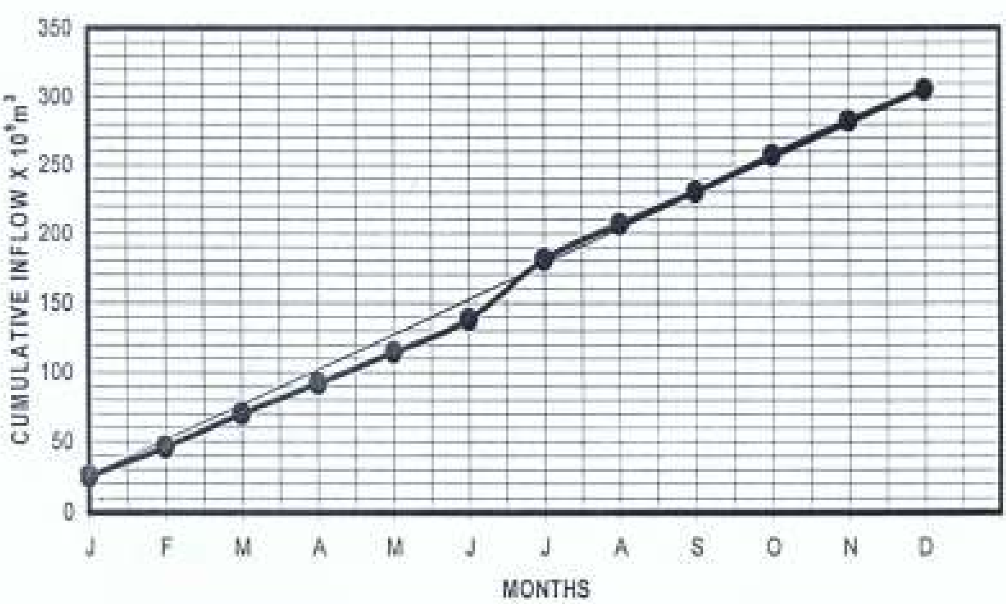

FIG 4.27 MASS CURVE OF RIVER OSUN AT APOJE GAUGING STATION (1997)

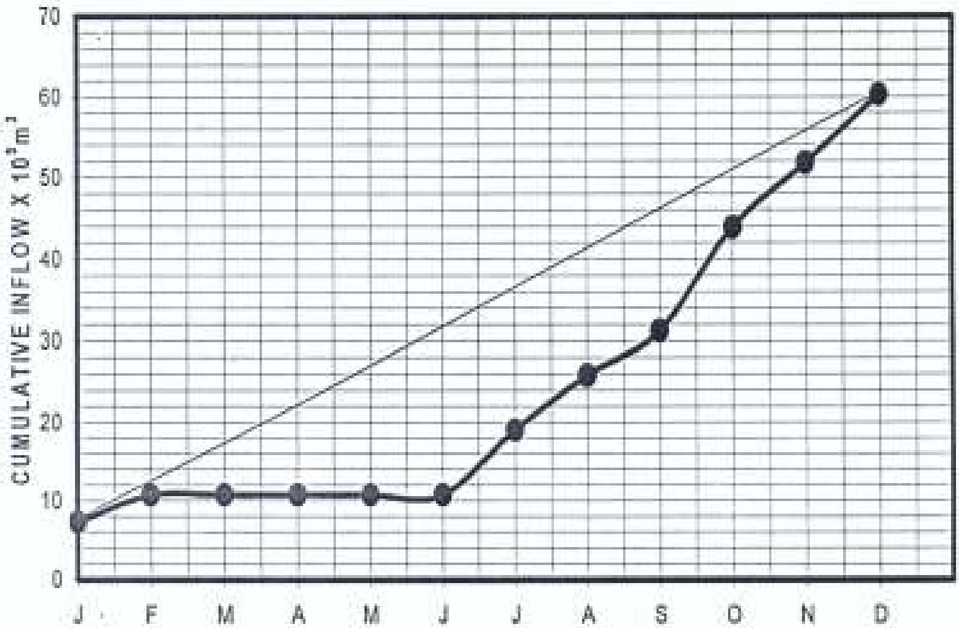

FIG 4.28 MASS CURVE OF RIVER OSUN AT APOJEGAUGING STATION (1998)

Fig 4.29 shows the mass curve in 1999 water year. The reservoir was full in February. In March, October and November, there was shortage of water supply because the reservoir capacities full short of the monthly demand. The maximum reservoir capacity of 20,300 million m^3 of water was adequate to support the monthly demand in 1999.

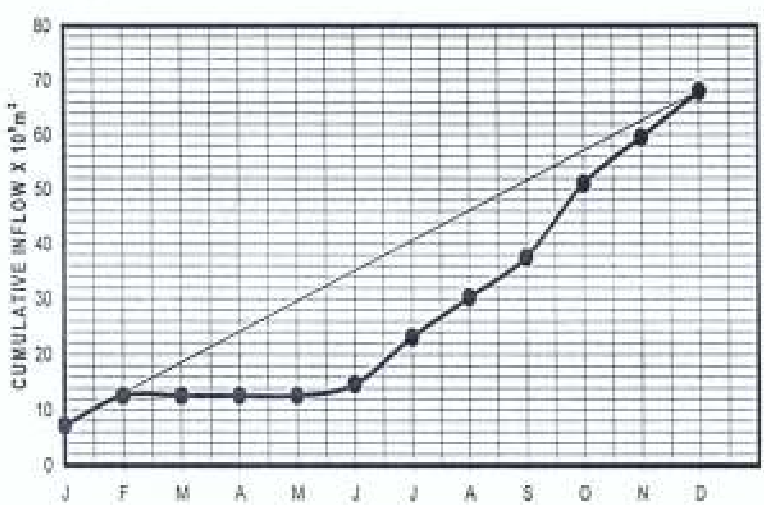

FIG 4.29 MASS CURVE OF RIVER OSUN AT APOJE GAUGING STATION (1999)

Table 4.6 contains the summary of the potential reservoir capacities of River Osun at Apoje gauging station from 1986 to 1999. The reservoir capacity in each year was determined by finding the mean of the reservoir capacities from March to June. The spillway capacity in each year was determined by finding the mean of the spillway capacities from July to October. The maximum reservoir capacity of each year is the summation of the mean reservoir and spillway capacities there were variations in the annual cumulative volumes of water at Apoje station. The highest cumulative volume of 305,000 million m^3 was observed in 1997 while the lowest cumulative volume of 60,000 million m^3 was observed in 1999. The mean cumulative volume for the 18-year period was 125,000 million m^3. The standard deviation and the coefficient of variation of the cumulative volumes from 1982 to 1999 were 57,720 million m^3 and 0.42 respectively. The reservoir capacity in each of the water years was higher than their corresponding spillway capacity except in 1986 and 1991 respectively. This indicates that the excess water during the period of heavy rainfall could be stored. There were water shortages in 1991 and 1997 because their maximum reservoir capacities were lower than their corresponding monthly demands. The highest "maximum reservoir capacity" of 21,375 million m^3 was estimated for 1989. During the 18-year study period, February of each year experienced water shortages when compared with other months. This was due to the decrease in rainfall amount that contributed immensely to river flow.

Since the purpose of building a reservoir is to cater for the diverse needs such as agriculture, hydroelectric power generation, water supply, a reservoir of about 22,000 million m^3 of water would be needed at Apoje sub-basin.

Table 4.6: Potential Reservoir Capacities of River Osun at Apoje Gauging Station from 1986 to 1999.

Water Years	Reservoir Capacities March-June (Million m^3)	Spillway Capacities July-October (Million m^3)	Maximum Reservoir Capacity (Million m^3)	Comments
1986	1,733	6,400	8,133	Adequate
1987	16,575	-	16,575	Adequate
1988	15,550	-	15,550	Adequate
1989	16,575	4,800	21,375	Adequate
1990	14,450	-	14,450	Adequate
1991	2,550	5000	7,550	Inadequate
1992	16,600	-	16,600	Adequate
1993	13,375	-	13,375	Adequate
1994	10,075	5200	15,275	Adequate
1995	18,575	-	18,575	Adequate
1996	15,950	-	15,950	Adequate
1997	8,725	3233	11,958	Inadequate
1998	12,850	-	12,850	Adequate
1999	13,600	-	13,600	Adequate

CHAPTER FIVE

5.1 CONCLUSION AND RECOMMENDATIONS.

Based on the results of the analysis carried out, the following conclusions were arrived at:

1. The annual maximum discharges River Osun at Apoje gauging station vary in magnitude and space. The ranged from 169 m^3/s to 400 m^3/s within a space of 18 years;
2. The Welbull's and the California plotting positions were more better because they gave a good match between the predicted ansd the observed discharges for the distributions of Osun River basin at Apoje gauging station.
2. The Log Pearson Type (III) and the Normal distributions fitted to the observed annual maximum than the Log Normal distributions. This is because; the highest coefficients of determination of 0.99 was achieved under the stated distributions;
3. The negative skew coefficient determined under the log Pearson type (III) distribution indicates that the Apoje sub – basin is prone to flood therefore the upper flood limits should be used for designing any hydraulic structures for the station;
4. The estimated Floods by Log Pearson Type (III) distribution were smaller in magnitudes than those estimated by log Normal distribution even at the same return period, this was due to the use of skew coefficient in determining the frequency factors under this distribution;
6. The annual cumulative volume of River Osun at Apoje sub-basin varies from 68,000 million m^3 to 305,550 million m^3 within duration of 18 years. The mean of the cumulative volumes was 125,080 million m^3 while the standard

deviation and the coefficient of variation of the cumulative volume were 57,720 million m^3 and 0.42 respectively;

7. The estimated reservoir capacity for Apoje sub-basin was 22,000 million m^3 this indicates that there are abundant water resources in Apoje sub – basin which can be harnessed for economic development.

5.2 Recommendations.

In the light of the above findings, the following recommendations were suggested:

1. There should be continuos and regular streamflow measurements at the gauging station.
2. There should be a commencement of long abandoned farming activities in the area.

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APPENDICES

APPENDIX A: Hydrologic Data at Apoje Gauging Station (1982-1999).

Water Years.	Number of Days of Measurement	Annual Total Discharges (m^3/s).	Systematic Annual Maximum Discharges (m^3/s).	Gauge Heights (m).
1982	275	32984	400	4.20
1983	365	33796	372	4.01
1984	366	42073	372	4.00
1985	313	43401	372	4.00
1986	365	32476	363	3.96
1987	365	44845	360	3.95
1988	366	49135	354	3.92
1989	365	51238	337	3.85
1990	365	52544	306	3.70
1991	317	53069	299	3.67
1992	342	61748	273	3.52
1993	365	58588	255	3.40
1994	365	51241	236	3.32
1995	366	48055	227.5	3.27
1996	364	48094	209.5	3.13
1997	365	41173	200	3.04
1998	227	23199	195	3.00
1999	259	26540	169	2.76

Source: *Ogun- Osun River Basin and Rural Development Authority, Abeokuta, Ogun state, Nigeria.*

APPENDIX B1

The Use of Hazen's Formula in Plotting the Observed and the Predicted Floods Fitted Using

Normal, Log Normal and Log Pearson Type (111) Distributions

Systematic Annual Maximal $Q(m^3s^{-1})$	Ranks (m)	Probability of non-Exceedence (%)	Return Periods Tr (Years)	Frequency Factors Z	Predicted Floods using Normal Distribution $Q(m^3s^{-1})$	Predicted Floods using Log-Normal Distribution Q (m3/s)	Frequency Factors Kt	Predicted Floods using Log-Pearson Type (111) Distribution $Q(m^3s^{-1})$
400	1	97.37	38.00	1.9384	439.6	492.7	1.7100	462.6
372	2	92.11	12.67	1.4124	400.2	426.1	1.3213	415.5
372	2	92.11	12.67	1.4124	400.2	426.1	1.3213	415.5
372	2	92.11	12.67	1.4124	400.2	426.1	1.3213	415.5
363	3	86.84	7.60	1.1190	378.2	392.9	1.0877	389.5
360	4	81.58	5.43	0.8993	361.8	369.8	0.9046	370.3
354	5	76.32	4.22	0.7162	348.1	351.52	0.7466	354.5
337	6	71.05	3.46	0.5545	335.9	336.2	0.6029	340.7
306	7	65.79	2.92	0.4063	324.8	322.7	0.4676	328.2
299	8	60.53	2.53	0.2666	314.4	310.5	0.3371	316.6
273	9	55.26	2.24	0.1320	304.3	299.1	0.2084	305.5
255	10	50.00	2.00	0.0000	299.4	288.4	0.0795	294.8
236.5	11	47.74	1.81	-0.1320	284.5	278.1	-0.0522	284.3
227.5	12	39.47	1.65	-0.2666	274.4	267.9	-0.1894	273.7
209.5	13	34.21	1.52	-0.4063	263.9	257.8	-0.3349	262.9
200	14	28.95	1.54	-0.5545	252.9	247.4	-0.4928	251.7
195	15	23.68	1.31	-0.7163	240.8	236.6	-0.6692	239.7
169	16	18.42	1.23	-0.8993	227.0	224.9	-0.8742	226.5

APPENDIX B2

The Use of Weibull's Formula in Plotting the Observed and the Predicted Floods Fitted

Using Normal, Log Normal and Log Pearson Type (111) Distributions.

Systematic Annual Maximal $Q(m^3s^{-1})$	Ranks (m)	Probability of non-Exceedence (%)	Return Periods Tr (Years)	Frequency Factors Z	Predicted Floods using Normal Distribution $Q(m^3s^{-1})$	Predicted Floods using Log-normal Distribution Q (m3/s)	Frequency Factors Kt	Predicted Floods using Log-Pearson Type (111) Distribution $Q(m^3s^{-1})$
400	1	95	20	1.6452	418.1	454.4	1.4981	450.0
372	2	90	10	1.2817	390.9	410.9	1.2188	414.2
372	2	90	10	1.2817	390.9	410.9	1.2188	414.2
372	2	90	10	1.2817	390.9	410.9	1.2188	414.2
363	3	85	6.67	1.0364	372.5	384.0	1.0197	390.4
360	4	80	5.0	0.8415	357.9	363.9	0.8552	371.8
354	5	75	4.00	0.6742	345.4	347.5	0.7096	356.2
337	6	70	3.33	0.5240	334.2	333.3	0.5753	342.2
306	7	65	2.857	0.3849	323.7	320.7	0.4478	329.4
299	8	60	2.5	0.2529	313.8	309.3	0.3241	317.6
273	9	55	2.222	0.1253	304.3	298.5	0.2020	306.2
255	10	50	2.00	0.0000	294.9	288.4	0.0795	295.3
236.5	11	45	1.818	-0.1253	285.5	278.6	-0.0455	284.5
227.5	12	40	1.667	-0.2529	275.96	268.9	-0.1753	273.8
209.5	13	35	1.532	-0.3849	266.1	259.3	-0.3124	262.9
200	14	30	1.428	-0.5240	255.7	249.5	-0.4600	251.6
195	15	25	1.333	-0.6742	244.4	239.4	-0.6229	239.7
169	16	20	1.250	-0.8415	231.9	228.6	-0.8088	226.8

APPENDIX B3

The Use of Blom's Formula in Plotting the Observed and the Predicted Floods Fitted Using Normal, Log Normal and Log Pearson Type (111) Distributions.

Systematic Annual Maximal $Q(m^3s^{-1})$	Ranks (m)	Probability of non-Exceedence (%)	Return Periods Tr (Years)	Frequency Factors Z	Predicted Floods using Normal Distribution $Q(m^3s^{-1})$	Predicted Floods using Log-Normal Distribution Q (m3/s)	Frequency Factors Kt	Predicted Floods using Log-Pearson Type (111) Distribution $Q(m^3s^{-1})$
400	1	96.73	30.80	1.8461	433.2	480.3	1.6446	454.3
372	2	91.56	11.85	1.3762	397.9	421.8	1.2931	412.3
372	2	91.56	11.85	1.3762	397.9	421.8	1.2931	412.3
372	2	91.56	11.85	1.3762	397.9	421.8	1.2931	412.3
363	3	86.36	7.33	1.0969	377.1	390.5	1.0695	387.6
360	4	81.17	5.31	0.8840	361.1	368.2	0.8916	368.9
354	5	75.97	4.16	0.7052	347.7	350.4	0.7369	353.5
337	6	70.78	3.42	0.5466	335.8	335.4	0.5957	340.0
306	7	65.58	2.91	0.4000	324.9	322.2	0.4625	327.7
299	8	60.39	2.52	0.2630	314.6	310.1	0.3337	316.2
273	9	55.19	2.23	0.1303	304.7	298.9	0.2068	305.4
255	10	50.00	2.00	0.0000	294.9	288.4	0.2068	294.8
236.5	11	44.80	1.81	-0.1303	285.1	278.2	-0.0505	284.4
227.5	12	39.61	1.66	-0.2630	275.2	268.2	-0.1857	273.9
209.5	13	34.42	1.52	-0.4000	264.9	258.2	-0.3290	263.3
200	14	29.22	1.41	-0.5466	253.9	247.9	-0.4841	252.3
195	15	24.03	1.32	-0.7052	242.1	237.3	-0.6569	240.5
169	16	18.83	1.23	-0.8840	228.7	225.9	-0.8568	227.6

APPENDIX B4

The Use of Cunnane's Formula in Plotting the Observed and the Predicted Floods Fitted Using Normal, Log Normal and Log Pearson Type (111) Distributions.

Systematic Annual Maximal $Q(m^3s^{-1})$	Ranks (m)	Probability of non-Exceedence (%)	Return Periods Tr (Years)	Frequency Factors Z	Predicted Floods using Normal Distribution $Q(m^3s^{-1})$	Predicted Floods using Log-Normal Distribution Q (m3/s)	Frequency Factors Kt	Predicted Floods using Log-Pearson Type (111) Distribution $Q(m^3s^{-1})$
400	1	96.88	32	1.8631	433.9	482.6	1.6567	455.8
372	2	91.67	12	1.3832	398.0	422.7	1.2986	412.9
372	2	91.67	12	1.3832	398.0	422.7	1.2986	412.9
372	2	91.67	12	1.3832	398.0	422.7	1.2986	412.9
363	3	86.46	7.38	1.1012	376.9	390.9	1.0731	387.9
360	4	81.25	5.33	0.8870	360.8	368.5	0.8941	369.2
354	5	76.04	4.17	0.7074	347.4	350.7	0.7388	353.7
337	6	70.04	3.43	0.5481	335.5	335.6	0.5971	340.1
306	7	65.63	2.91	0.4018	324.5	322.3	0.4635	327.8
299	8	60.42	2.53	0.2637	314.2	310.2	0.3344	316.3
273	9	52.21	2.23	0.1306	304.2	299.0	0.2071	305.4
255	10	50.00	2.00	0.0000	294.4	288.4	-0.0795	294.8
236.5	11	44.79	1.81	-0.1306	284.6	278.2	-0.0508	284.4
227.5	12	39.58	1.66	-0.2637	274.6	268.1	-0.1864	273.9
209.5	13	34.38	1.52	-0.4018	264.3	258	-0.3302	263.3
200	14	29.17	1.41	-0.5481	253.3	247.9	-0.4859	253.2
195	15	23.96	1.32	-0.7074	241.42	237.2	-0.6594	240.4
169	16	18.75	1.23	-0.8870	227.9	225.7	-0.8602	227.4

APPENDIX B5

The Use of California Formula in Plotting the Observed and the Predicted Floods Fitted

Using Normal, Log Normal and Log Pearson Type (111) Distributions.

Systematic Annual Maximal $Q(m^3s^{-1})$	Ranks (m)	Probability of non-Exceedence (%)	Return Periods Tr (Years)	Frequency Factors Z	Predicted Floods using Normal Distribution $Q(m^3s^{-1})$	Predicted Floods using Log-Normal Distribution Q (m3/s)	Frequency Factors Kt	Predicted Floods using Log-Pearson Type (111) Distribution $Q(m^3s^{-1})$
400	1	94.74	19.0	1.6202	415.8	451.3	1.4794	434.0
372	2	89.47	9.50	1.2523	388.2	407.6	1.1953	401.3
372	2	89.47	9.50	1.2523	388.2	407.6	1.1953	401.3
372	2	89.47	9.50	1.2523	388.2	407.6	1.1953	401.3
363	3	84.21	6.33	1.0031	369.5	380.5	0.9920	379.4
360	4	78.95	4.75	0.8044	354.7	360.2	0.8233	362.1
354	5	73.68	3.80	0.6333	341.8	343.6	0.6734	347.4
337	6	68.42	3.17	0.4791	330.3	329.2	0.5345	334.3
306	7	63.16	2.71	0.3356	319.5	316.4	0.4019	322.3
299	8	57.89	2.38	0.1988	309.3	304.7	0.2727	310.9
273	9	52.63	2.11	0.0658	299.3	293.7	0.1441	300.1
255	10	47.37	1.90	-0.0658	289.5	283.2	0.0147	289.6
236.5	11	42.11	1.73	-0.1988	279.5	272.9	-0.1195	279.0
227.5	12	36.84	1.53	-0.3356	269.3	262.9	-0.2604	268.4
209.5	13	31.58	1.46	-0.4791	258.5	252.6	-0.4116	257.4
200	14	26.32	1.36	-0.6333	246.9	242.1	-0.5778	245.8
195	15	21.05	1.27	-0.8044	234.2	230.9	--0.7670	233.3
169	16	15.79	1.19	-1.0031	219.3	218.6	-0.9929	219.2

APPENDIX B6

The Use of Gringorton 's Formula in Plotting the Observed and the Predicted Floods Fitted

Using Normal, Log Normal and Log Pearson Type (111) Distributions.

Systematic Annual Maximal $Q(m^3s^{-1})$	Ranks (m)	Probability of non-Exceedence (%)	Return Periods Tr (Years)	Frequency Factors Z	Predicted Floods using Normal Distribution $Q(m^3s^{-1})$	Predicted Floods using Log-Normal Distribution Q (m3/s)	Frequency Factors Kt	Predicted Floods using Log-Pearson Type (111) Distribution $Q(m^3s^{-1})$
400	1	97.07	34.14	1.8918	436.1	486.4	1.6771	458.4
372	2	91.84	12.26	1.3947	398.9	424.0	1.3076	413.9
372	2	91.84	12.26	1.3947	398.9	424.0	1.3076	413.9
372	2	91.84	12.26	1.3947	398.9	424.0	1.3076	413.9
363	3	86.61	7.47	1.1082	377.4	391.7	1.0789	388.6
360	4	81.38	5.37	0.8919	361.2	369.0	0.8983	369.7
354	5	76.15	4.19	0.7109	347.6	351.0	0.7419	354.0
337	6	70.92	3.44	0.5507	335.7	335.8	0.5994	340.4
306	7	65.69	2.62	0.4035	324.6	322.4	0.4651	327.9
299	8	60.46	2.53	0.2649	314.2	310.3	0.3354	316.4
273	9	55.23	2.23	0.1312	304.2	299.1	0.2076	305.4
255	10	50.00	2.00	0.0000	294.4	288.4	0.0795	294.8
236.5	11	44.77	1.81	-0.1312	284.6	278.1	-0.0514	284.3
227.5	12	39.54	1.65	-0.2649	274.6	268.1	-0.1876	273.8
209.5	13	34.31	1.52	-0.4035	264.2	257.9	-0.3320	263.1
200	14	29.09	1.41	-0.5507	253.2	247.7	-0.4886	251.9
195	15	23.85	1.31	-0.7109	241.2	236.9	-06633	240.1
169	16	18.62	1.23	-0.8919	227.6	225.4	-0.8658	227.0

APPENDIX B7

The Use of Chegodayev's Formula in Plotting the Observed and the Predicted Floods Fitted Using Normal, Log Normal and Log Pearson Type (111) Distributions.

Systematic Annual Maximal $Q(m^3s^{-1})$	Ranks (m)	Probability of non-Exceedence (%)	Return Periods Tr (Years)	Frequency Factors Z	Predicted Floods using Normal Distribution $Q(m^3s^{-1})$	Predicted Floods using Log-normal Distribution Q (m3/s)	Frequency Factors Kt	Predicted Floods using Log-Pearson Type (111) Distribution $Q(m^3s^{-1})$
400	1	96.39	27.71	1.7985	429.1	474.0	1.6103	450.0
372	2	91.24	11.41	1.3557	395.9	419.5	1.2771	410.5
372	2	91.24	11.41	1.3557	395.9	419.5	1.2771	410.5
372	2	91.24	11.41	1.3557	395.9	419.5	1.2771	410.5
363	3	86.08	7.19	1.0841	375.6	389.1	1.0590	386.4
360	4	80.93	5.24	0.8751	359.9	367.3	0.8840	368.2
354	5	75.77	4.30	0.6987	346.7	349.8	0.7312	352.9
337	6	70.62	3.40	0.5419	334.9	334.9	0.5915	339.6
306	7	65.46	2.89	0.3974	324.2	321.9	0.4594	327.4
299	8.	60.31	2.52	0.2609	313.9	309.9	0.3317	316.1
273	9	55.15	2.23	0.1293	304.1	298.9	0.2058	305.3
255	10	50.00	2.00	0.0000	294.4	288.4	0.0795	294.8
236.5	11	44.85	1.81	-0.1293	284.7	278.3	-0.0495	284.5
227.5	12	39.69	1.66	-0.2609	274.9	268.3	-0.1836	274.1
209.5	13	34.54	1.52	-0.3974	264.6	258.4	-0.3255	263.6
200	14	29.38	1.41	-0.5419	253.8	248.3	-0.4791	252.6
195	15	24.23	1.32	-0.6987	242.1	237.8	-0.6498	241.0
169	16	19.07	1.24	-0.8751	228.85	226.5	-0.8467	228.2

APPENDIX C.

The Upper, Lower and the Control Floods Limits Fitted by Log Pearson Type (111) Distribution.

Probability of Non-exceedence (%)	Upper Frequency Factors.	Upper Floods Limits (m^3/s)	Control Floods Fitted By Log Pearson Type (111) Distribution (m^3/s)	Lower Frequency Factors.	Lower Floods Limits (m^3/s)
95	2.4028	560.2	450.0	0.9513	375.1
90	2.0169	503.5	414.2	0.7124	351.1
90	2.0165	503.5	414.2	0.7124	351.1
90	2.0165	503.5	414.2	0.7124	351.1
85	1.7462	467.2	390.4	0.5369	334.5
80	1.5269	439.8	371.8	0.3879	321.0
75	1.3365	417.2	356.2	0.2523	309.2
70	1.1644	397.9	342.2	0.1237	298.4
65	1.0046	380.7	329.4	-0.0019	288.3
60	0.8533	365.1	317.6	-0.1275	278.4
55	0.5657	350.7	306.2	-0.1255	268.8
50	0.4252	337.2	295.3	-0.3878	259.1
45	0.2839	324.4	284.5	-0.5271	249.2
40	0.1397	299.8	273.8	-0.6764	239.2
35	-0.0102	287.6	262.9	-0.8391	228.7
30	-0.1697	275.2	251.6	-1.0197	217.6
25	-0.3451	262.2	239.7	-1.2249	205.6
20	-0.5475	247.9	226.8	-1.4658	192.4

APPENDIX D.

APPENDIX D1

Monthly Total Discharges, Monthly Total Volumes and the Cumulative Volumes for 1982.

MONTHS	MONTHLY TOTAL DISCHARGES (m^3/s)	NUMBER OF DAYS OF MEASUREMENT	MONTHLY TOTAL VOLUMES X $10^{10}(m^3)$	CUMULATIVE VOLUMES X 10^9 (m^3)
J				
F				
M				
A	2252.4	30	0.58	5.8
M	3143	31	0.84	14.3
J	4149	30	1.08	25.0
J	4320.5	31	1.16	36.6
A	2920.1	31	0.78	44.4
S	3656.5	30	0.95	53.9
O	6541	31	1.75	71.4
N	3507.5	30	0.91	80.5
D	2493.5	31	0.67	87.2
			8.72	

APPENDIX D2

Monthly Total Discharges Monthly Total Volumes and the Cumulative Volumes for 1983

MONTHS	MONTHLY TOTAL DISCHARGES (m^3/s)	NUMBER OF DAYS OF MEASUREMENT	MONTHLY TOTAL VOLUMES X $10^{10}(m^3)$	CUMULATIVE VOLUMES X 10^9 (m^3)
J	2232	31	0.60	6.0
F	2016	28	0.49	10.9
M	2232	31	0.60	16.8
A	2209.3	30	0.57	22.6
M	2288.1	31	0.61	28.7
J	3531.8	30	0.92	37.8
J	3373	31	0.90	46.9
A	2895.1	31	0.78	54.6
S	4282.5	30	1.11	65.7
O	3952.5	31	1.06	76.3
N	2518.5	30	0.65	82.8
D	2265.3	31	0.61	88.9
			8.89	

APPENDIX D3

Monthly Total Discharges, Monthly Total Volumes and the Cumulative Volumes for 1984

MONTHS	MONTHLY TOTAL DISCHARGES (m^3/s)	NUMBER OF DAYS OF MEASUREMENT	MONTHLY TOTAL VOLUMES $X10^{10}(m^3)$	CUMULATIVE VOLUMES X 10^9 (m^3)
J	2232	31	0.60	6.0
F	2088	29	0.52	11.2
M	2232	31	0.60	17.2
A	2196.8	30	0.57	22.9
M	2598.1	31	0.70	29.8
J	3813.5	30	0.99	39.7
J	4839	31	1.30	52.7
A	5548	31	1.49	67.5
S	5107.5	30	1.32	80.8
O	5589	31	1.50	95.8
N	3299.5	30	0.86	104.3
D	2529.6	31	0.68	111.1
			11.11	

APPENDIX D4

Monthly Total Discharges, Monthly Total Volumes and the Cumulative Volumes for 1985

MONTHS	MONTHLY TOTAL DISCHARGES (m^3/s)	NUMBER OF DAYS OF MEASUREMENT.	MONTHLY TOTAL VOLUMES $X\ 10^{10}(m^3)$	CUMUMILATIVE VOLUMES X 10^9 (m^3)
J	1951.6	25	0.42	4.2
F	720	10	0.06	4.8
M	1152	16	0.16	6.4
A	2212.3	30	0.57	12.2
M	2485.9	31	0.67	18.8
J	3562	30	0.92	28.1
J	6251.5	31	1.67	44.8
A	7385.7	31	1.98	64.6
S	4389.5	17	0.64	71.0
O	6739	31	1.80	89.1
N	3584	30	0.93	98.4
D	2968.1	31	0.79	106.3
			10.63	

APPENDIX D5

Monthly Total Discharges, Monthly Total Volumes and the Cumulative Volumes for 1986

MONTHS	MONTHLY TOTAL DISCHARGES (m^3/s)	NUMBER OF DAYS OF MEASUREMENT.	MONTHLY OTAL VOLUMES X $10^{10}(m^3)$	CUMULATIVE VOLUMES X $10^9(m^3)$
J	2568.8	31	0.69	6.9
F	2258.8	28	0.55	12.3
M	2681.6	31	0.72	19.5
A	2564.8	30	0.66	26.2
M	2577	31	0.69	33.1
J	3250.9	30	0.84	41.5
J	4243	31	1.14	52.9
A	3470.5	31	0.93	62.2
S	4811	30	1.25	74.6
O	98.31	31	0.03	74.9
N	3898	30	1.01	85.0
D	53.15	31	0.01	85.1
			8.51	

APPENDIX D6

Monthly Total Discharges, Monthly Total Volumes and the Cumulative Volumes for 1987

MONTHS	MONTHLY TOTAL DISCHARGES (m^3/s)	NUMBER OF DAYS OF MEASUREMENT.	MONTHLY TOTAL VOLUMES X $10^{10}(m^3)$	CUMULATIVE VOLUMES X $10^9(m^3)$
J	2254.7	31	0.60	6.0
F	2016	28	0.49	10.9
M	2232	31	0.60	16.9
A	2160	30	0.56	2.25
M	2285	31	0.61	28.6
J	2741.6	30	0.71	35.7
J	3264.2	31	0.87	44.5
A	6814.5	31	1.83	62.7
S	5812.5	30	1.51	77.8
O	8566.5	31	2.29	100.7
N	3754.3	30	0.97	110.5
D	2943.9	31	0.79	118.3
			11.83	

APPENDIX D7

Monthly Total Discharges, Monthly Total Volumes and the Cumulative Volumes for 1988

MONTHS	MONTHLY TOTAL DISCHARGES (m^3/s)	NUMBER OF DAYS OF MEASUREMENT.	MONTHLY TOTAL VOLUMES X 10^{10}(m^3)	CUMULATIVE VOLUMES X 10^9(m^3)
J	2400.3	31	0.64	6.4
F	2150.1	29	0.54	11.8
M	2565.5	31	0.69	18.7
A	2271.8	30	0.59	24.6
M	3346	31	0.90	33.5
J	4800	30	1.24	46.0
J	5842.5	31	1.56	61.6
A	4519.5	31	1.21	73.7
S	6439.5	30	1.67	90.4
O	7756	31	2.08	111.2
N	4078	30	1.06	121.8
D	2965.4	31	0.79	129.7
			12.97	

APPENDIX D8

Monthly Total Discharges, Monthly Total Volumes and the Cumulative Volumes for 1989

MONTHS	MONTHLY TOTAL DISCHARGES (m^3/s)	NUMBER OF DAYS OF MEASUREMENT.	MONTHLY TOTAL VOLUMES X 10^{10}(m^3)	CUMULATIVE VOLUMES X 10^9(m^3)
J	2544.8	31	0.68	6.8
F	2171	28	0.53	12.1
M	2391.1	31	0.64	18.5
A	2465.2	30	0.64	24.9
M	3259.5	31	0.87	33.6
J	3442.5	30	0.89	42.5
J	5328.5	31	1.43	56.8
A	6736	31	1.80	74.8
S	7628.5	30	1.98	94.6
O	8273.1	31	2.22	116.8
N	3801	30	0.99	126.6
D	3197	31	0.86	135.2
			13.52	

APPENDIX D9

Monthly Total Discharges, Monthly Total Volumes and the Cumulative Volumes for 1990

MONTHS	MONTHLY TOTAL DISCHARGES (m^3/s)	NUMBER OF DAYS OF MEASUREMENT.	MONTHLY TOTAL VOLUMES X 10^{10}(m^3)	CUMULATIVE VOLUMES X 10^9(m^3)
J	3373	31	0.90	9.0
F	2795.5	28	0.68	15.8
M	2699.5	31	0.72	23.0
A	2794	30	0.72	30.3
M	3566.5	31	0.96	39.8
J	3573.5	30	0.93	49.1
J	5330.5	31	1.43	63.4
A	5902	31	1.58	79.2
S	6748.5	30	1.75	96.7
O	6509.5	30	1.69	113.5
N	4717.5	30	1.22	125.8
D	4534	31	1.21	137.9
			13.79	

APPENDIX D10

Monthly Total Discharges, Monthly Total Volumes and the Cumulative Volumes for 1991

MONTHS	MONTHLY TOTAL DISCHARGES (m^3/s)	NUMBER OF DAYS OF MEASUREMENT.	MONTHLY TOTAL VOLUMES X 10^{10}(m^3)	CUMULATIVE VOLUMES X 10^9(m^3)
J	3827.5	31	1.03	10.3
F	3256	28	0.79	18.1
M	3610	31	0.97	27.8
A	3583.5	30	0.93	37.1
M	4945.5	31	1.32	50.3
J	6066	30	1.57	66.1
J	4366	21	0.79	74.0
A	5368.5	16	0.74	81.4
S	4560.5	17	0.67	88.1
O	6087	21	1.10	99.1
N	3858	30	1.00	109.1
D	3540.5	31	0.95	118.6
			11.86	

APPENDIX D11

Monthly Total Discharges, Monthly Total Volumes and the Cumulative Volumes for 1992

MONTHS	MONTHLY TOTAL DISCHARGES (m^3/s)	NUMBER OF DAYS OF MEASUREMENT.	MONTHLY TOTAL VOLUMES X $10^{10}(m^3)$	CUMULATIVE VOLUMES X $10^9(m^3)$
J	3340	31	0.89	8.9
F	2959	29	0.74	16.4
M	3187	31	0.85	24.9
A	3361	30	0.87	33.6
M	4063	31	1.09	44.5
J	4709	30	1.22	56.7
J	7035	31	1.88	75.5
A	9576.2	31	2.56	101.2
S	6595.5	22	1.25	113.7
O	5661.5	28	1.37	127.4
N	6967.5	30	1.81	145.5
D	4293	31	1.15	157.0
			15.70	

APPENDIX D 12

I Monthly Total Discharges, Monthly Total Volumes and the Cumulative Volumes for 1993

MONTHS	MONTHLY TOTAL DISCHARGES (m^3/s)	NUMBER OF DAYS OF MEASUREMENT.	MONTHLY TOTAL VOLUMES X $10^{10}(m^3)$	CUMULATIVE VOLUMES X $10^9(m^3)$
J	3648	31	0.98	9.8
F	3374.5	28	0.82	17.9
M	4029	31	1.08	28.7
A	3276	30	0.85	37.2
M	3632.5	31	0.97	46.9
J	4715.5	30	1.22	59.2
J	6791	31	1.82	77.4
A	5707	31	1.53	92.6
S	6417	30	1.66	10.9.3
O	6179.5	31	1.66	125.8
N	6417.5	30	1.66	142.5
D	4400.5	31	1.18	154.2
			15.42	

APPENDIX D 13

Monthly Total Discharges, Monthly Total Volumes and the Cumulative Volumes for 1994

MONTHS	MONTHLY TOTAL DISCHARGES (m^3/s)	NUMBER OF DAYS OF MEASUREMENT.	MONTHLY TOTAL VOLUMES X $10^{10}(m^3)$	CUMULATIVE VOLUMES X $10^9(m^3)$
J	3499	31	0.94	9.4
F	3165.5	28	0.77	17.0
M	3496.5	31	0.94	26.4
A	3232.5	30	0.84	34.8
M	3521	31	0.94	44.2
J	3891	30	1.01	54.3
J	6310.5	31	1.69	71.2
A	6008	31	1.61	87.3
S	5774.5	30	1.50	102.3
O	5592.5	31	1.50	117.2
N	3456.5	30	0.90	126.2
D	3299	31	0.88	135.0
			13.50	

APPENDIX D 14

Monthly Total Discharges, Monthly Total Volumes and the Cumulative Volumes for 1995

MONTHS	MONTHLY TOTAL DISCHARGES (m^3/s)	NUMBER OF DAYS OF MEASUREMENT.	MONTHLY TOTAL VOLUMES X $10^{10}(m^3)$	CUMULATIVE VOLUMES X $10^9(m^3)$
J	2388.9	31	0.64	6.4
F	2016	28	0.49	11.3
M	2232	31	0.60	17.3
A	2160	30	0.56	22.9
M	2416.7	31	0.65	29.3
J	2649.2	30	0.69	36.2
J	3148	31	0.84	44.6
A	7640.1	31	2.05	65.1
S	7636.5	30	1.98	84.9
O	7669.5	31	2.05	105.4
N	4710	30	1.22	117.6
D	3388	31	0.91	126.7
			12.67	

APPENDIX D15

Monthly Total Discharges, Monthly Total Volumes and the Cumulative Volumes for 1996

MONTHS	MONTHLY TOTAL DISCHARGES (m^3/s)	NUMBER OF DAYS OF MEASUREMENT.	MONTHLY TOTAL VOLUMES X $10^{10}(m^3)$	CUMULATIVE VOLUMES X $10^9(m^3)$
J	2937.2	30	0.76	7.6
F	2112.4	29	0.53	12.9
M	2232	31	0.60	18.9
A	2215.2	30	0.57	24.6
M	2781.9	31	0.75	32.1
J	4842.5	30	1.26	44.6
J	5410	30	1.40	58.7
A	4134.5	31	1.11	69.7
S	6116.5	30	1.59	85.6
O	7690	31	2.06	106.2
N	4460	30	1.16	117.7
D	3162	31	0.85	126.2
			12.62	

APPENDIX D16

Monthly Total Discharges, Monthly Total Volumes and the Cumulative Volumes for 1997

MONTHS	MONTHLY TOTAL DISCHARGES (m^3/s)	NUMBER OF DAYS OF MEASUREMENT.	MONTHLY TOTAL VOLUMES X $10^{10}(m^3)$	CUMULATIVE VOLUMES X $10^9(m^3)$
J	3373.5	31	2.55	25.5
F	3059.5	28	2.09	46.3
M	3172	31	2.40	70.3
A	3030.5	30	2.21	92.4
M	2962.4	31	2.24	114.8
J	3249.5	30	2.37	138.6
J	5637.5	31	4.26	181.1
A	3414.5	31	2.58	206.9
S	3231.5	30	2.36	230.5
O	3541.5	31	2.67	257.3
N	3447	30	2.52	282.5
D	3054	31	2.31	305.5
			30.55	

APPENDIX D 17

Monthly Total Discharges, Monthly Total Volumes and the Cumulative Volumes for 1998

MONTHS	MONTHLY TOTAL DISCHARGES (m^3/s)	NUMBER OF DAYS OF MEASUREMENT.	MONTHLY TOTAL VOLUMES X $10^{10}(m^3)$	CUMULATIVE VOLUMES X $10^9(m^3)$
J	2733.3	31	0.73	7.3
F	1812.3	22	0.34	10.8
M	0		0.00	10.8
A	0		0.00	10.8
M	0		0.00	10.8
J	0		0.00	10.8
J	3087.5	31	0.83	19.0
A	2526.9	31	0.68	25.8
S	2105.7	30	0.55	31.3
O	4731.5	31	1.27	43.9
N	3039.5	30	0.79	51.8
D	3162	31	0.85	60.3
			6.03	

APPENDIX D 18

Monthly Total Discharges, Monthly Total Volumes and the Cumulative Volumes for 1999

MONTHS	MONTHLY TOTAL DISCHARGES (m^3/s)	NUMBER OF DAYS OF MEASUREMENT.	MONTHLY TOTAL VOLUMES X $10^{10}(m^3)$	CUMULATIVE VOLUMES X $10^9(m^3)$
J	2697	31	0.72	7.2
F	2247	28	0.54	12.7
M			0.00	12.7
A			0.00	12.7
M			0.00	12.7
J	1496	16	0.21	14.7
J	3150	31	0.84	23.2
A	2672.9	31	0.72	30.3
S	2814	30	0.73	37.6
O	5011	31	1.34	51.0
N	3280.5	30	0.85	59.5
D	3172	31	0.85	68.0
			6.80	