DEVELOPMENT OF REGIONAL FLOW DURATION CURVES FOR SELECTED RIVERS IN UPPER NIGER RIVER BASIN, NIGERIA

It is common knowledge that the end user of stream flow data may necessarily not have any prior knowledge of the quality control measures applied in their generation. Thus, this study provides an intuitive attempt to provide an independent quality indicator to boost the confidence in the use of stream flow data by developing regional flow duration curves for selected ungauged station of the upper Niger River Basin, Nigeria. Toward this end, stream flow data for seven gauging stations covering some sub-river basin in the upper Niger River Basin was effectively mobilised; in this case, monthly stream flow data covering a range of eleven to fifty three years period. The flow duration curves from the gauging stations were fitted with three distribution models; i.e., the logarithmic, power and exponential regression equations. For regionalisation, the parameterisation was carried out in terms of the drainage area alone; just for simplicity of models. Results obtained showed that the exponential regression equation, in terms of R^2 had the best fit. Though the regionalised model was simple, measurable agreement was obtained during the calibration and validation phases. However, considering the length of data used and probable variability in the stream flow regime, it is not possible to objectively generalise on the quality of the results. Against this backdrop, it suffices to take into cognisance the need to use an ensemble of catchment characteristics in the development of the flow duration curves and the overall regional models; this is important considering the implications of anthropogenic activities and hydro-climatic variations.

CHAPTER ONE

1.0 INTRODUCTION

1.1 Background of the Study

Hydrologic techniques such as estimation of water yield at specific locations, determination of response of a river basin for a given precipitation input and estimation of design storm/floods for water resources structures, if capably applied are essential for the sound development and management of water resources. The estimation of water potential for a particular water body finds relevance in the face of changing population dynamics. Generally water use out paces the growth in population and thus create demand for water that frequently impingent the available supply. As noted by Langbein and Iseri (1995), the water demand- supply conflict and the associated problem of it spatio-temporal fluctuations or variability require that robust approach for water analysis be available. It in important considering the fact that planning for water resources development must depend upon availability of hydrological information which forms the basis of sustainability. One of the largely used tools to evaluate flow rates and the river regime is the flow duration curve (FDC). This is a cumulative frequency curve that shows the percentage of time that a given discharge is equalled or exceeded during a fixed period (Ubertini, 2010). This plot provides information on the probability of occurrence or otherwise. The choice of this tool is important because it is one of the most informative methods of displaying the flow characteristics of a stream throughout the range of discharge, without regard to the sequence of occurrence. Basically, the water nature of the slope for a particular flow duration curve (FDC) gives an ideal of the storage, characteristic of a river. The flow duration curve may well indicate the watershed rainfall-runoff characteristic, and have been applied in many aspects of water resources development and water environment protection. These curves are derived in gauged river data, but are essential also in rivers where the systems for collecting and managing water information are inadequate.

Different approaches exist to solve this problem; they can be categorised into two groups, namely:

1. The statistical models: The flow estimation in ungauged sites is obtained by simple regression models, which help to connect flow statistics or parameters to known characteristics of the basins.
2. The second group is represented by rainfall – runoff models which allows for simulation and evaluation of the time series of river flows within a catchment. This method is useful in order to evaluate the interaction between the physical components of the water basin.

In line with the definition according to Bandaragoda (2001), a river basin in basically a hydrologic response unit; i.e, in defines an area which constitutes in the overall the geography unit. For any river system, there are quite a number of water related human interventions, including water storage, diversion, regulation, distribution, application, pollution, purification, and other associated acts; according to Sunaryo (2001), these action help to change the natural system. From time in memorial, human development have centered around the banks of rivers for water for agriculture, domestic and industrial purpose, hydro- power generation, e.t., this fact was reiterated by Adeloye (2002), especially the use of river for assimilation of human wastes. The availability of this water for various uses is highly variable, both temporally and spatially. As a consequence, it is often not possible to rely on river water in its natural occurrence to meet these needs without one form of human intervention or another (Adeloye, 2002). According to George (2005), water as a resource must be culturally defined because water by itself is not productive. Its use requires some minimum level of social and economic infrastructure for it to be productive. Nigeria have altogether over 162 large, medium and small dams that have been constructed and are being operated. They have a total storage capacity of more than 30109 m^3, i.e. less than 10% of the country's total potential surface water resources. Eighty five percent of the larger dams are located in the Sudano-Sahelian zone of the country. A sample of 52 dams indicates that 79% have domestic industrial water supply components, while 33% have irrigation as a major use to which the stored water is used for; 4% are also for hydro-electric power generation (HEP); 29% for fisheries and 16% for recreation. Because of these notable facts, Oyibande (1995) posted that the available dams irrespective of size contribute in quantifiable measure to flood mitigation and in advertency affect wetlands in their respective downstream areas.

The three largest hydropower dams are under operation and control the flow of the Niger and Kaduna rivers. These are Kainji, Jebba and Shiroro with total active capacity of 18.6 x 109m^3 and total power capacity of 1920 MW. In terms of storage usage however, irrigation accounts for 36%, water supply 3% and hydropower 61% (Oyebande, 1995). Accurate demand survey and assessment is a pre-requisite for efficient reservoir operation of these dams to meet the objectives of their construction. In Nigeria, the range of purposes served by storage reservoirs includes water Supply for irrigation, domestic and industrial uses, hydroelectric power (HEP), increasing water depth for navigation, flood control, reclamation of low-lying lands and recreation. Some of these uses conflict (e.g. flood control and HEP and other uses), thus, priorities and proper balancing need to be carefully considered. In order to carry out proper reservoir operation, certain basic data and drawings such as flow duration curves (FDC) are required at the operation and maintenance office. Unfortunately, most of the above drawings and data are not available in many operation and maintenance (O&M) offices at dam sites. As a result, most of the monitoring works needed for efficient and proper reservoir operation are not carried out. In reservoir management, the reservoir operational guideline should take into account reservoir inflow, evaporation losses, and irrigation demand both in wet and dry seasons, among other demands. Observed hydrological data at dam sites are indispensable for such considerations. Unfortunately, the scarcity of stream flow data is a common problem, as shown by the large number of studies addressing the regionalization of flow duration curves (FDCs) for different geographic regions around the world e.g, Le Boutivier and Waglen in Castallarin, A., Galeati, G., Brandimarte, C., Montanari, A., and Brath, A., (2004), Castallarin, A., camerani, G, and Brath, A., (2007), Viola, F, Noto, L.V Cannarozzo, M. and loggia La.G (2010), Patils, (2011). The situation in developing countries in this light in more dire; for instance, as reported by Mantra and Ahaneku (2009), the problem of stream flow data mobilisation in due to a lost of reasons like pecuniary related (i.e., fund,), and for a large extent, lack of proper data archiving and retrieval system.

Considering the dearth of stream flow data in terms of both quantity and integrity, the regionalization of FDCs appears therefore to be an essential operative tool when dealing with ungauged river basins or short stream flow records. That is, river basins of which the data is not identified or have limited amount of stream flow observations, the flow duration curve can be obtained by implementing the regional analysis. The ideal here is them search for a potential candidate curve based on available information of surrounding areas or basins to generate the possible flow duration curve. This concept in predicted on the assumption of hydro meteorological similarity of the neighbouring basins.

1.2 Statement of the Problem

The statement of the problem this study in informed by the following specific problem of increase hydrological importance Viz:-

1. Dearth of long and continuous stream flow data resulting from paucity of stream gauging station, in-a-in the corresponding relevant information base.
2. Improper inventory of available surface water bodies limits any probable quantitative water resource planning strategy.
3. Hydrological data archiving and retrieval/assessment pose a daunting challenge. That is, the choice of an appropriate data length and quality is extremely impaired, especially the absence of stage- discharge rating curve in most river basin.

1.3 Aim and objectives of the study

The aim of this study is to develop regional flow duration curves (FDC's) for the ungauged sub-basins of the Upper Niger River Basin. In view of the general aim of the research, the specific objectives of the study are to:

1. develop regional flow duration models for purposes of understanding the hydrologic signatures of the sub-basins.
2. quantify the reliability and effectiveness of the regional approach and derive flood and low flow indexes for the ungauged catchment.

3 determine the possibility of effective generalisation of the result of the regionalisation approach in the face of data austerity; i.e., likelihood of optimising the hydrologic parameters of the regional models.

1.4 Justification of the Study

Regionalization is shown to be a powerful tool in transferring hydrological information from gauged sites to other remote ungauged sites within the homogeneous region. The developed regional models are used in predicting accurately hydrological information, such as sediment rating curves, maximum flood events, unit graphs, and mean annual floods for ungauged basins and is needed for the hydrological design of water resources development systems. The result of this study will therefore be of tremendous benefit to the Government and non-governmental organizations involved in water resources planning and operations in Kaduna, Gbako and Gurara river sub-basins to sustain the rapid socio – economic development in the Federal Capital Territory (FCT) and its neighbouring states, especially quantification of water potential of rivers for hydroelectric power (HEP) generation, increasing water depth for navigation, flood control, reclamation of low-lying lands and recreation.

1.5 Scope and limitation of the study

The study is basically limited to the establishment of regionalised flow duration curves for the selected ungauged rivers in the Upper Niger River Basin and the determination of the model performance characteristics. Beside this, the study was mired by the absence of long and continuous stream flow data.

CHAPTER TWO

2.0 Literature Review

2.1 General Concept of Flow Duration Curves

The flow duration curve (FDC) is a cumulative frequency curve that shows the percentage of time that a given discharge is equalled or exceeded during a fixed period. It is one of the most informative methods of displaying the flow characteristics of a stream throughout the range of discharge, without regard to the sequence of occurrence. It is possible to observe this curve as the relationship between magnitude and frequency (Smakhtin, 2001). This graph in the usual construction does not have a probability meaning because discharge is correlated to successive time intervals and discharge characteristics are dependent on the season; hence the probability that discharge on a particular day exceeds a specified value, depends on the discharge on preceding days and of the time of the year as pointed out by (Mosley and McKerchar, 1993, Ganora et al. (2010). In spite of this, the FDC is often seen as the complement of the cumulative distribution function of the considered stream flow at a site. FDC construction is attributed to Clemens Herschel and is dated back to 1880 whilst the general use of the curve is dated 1915. Searcy, (1959) as reported in Ubertini (2010) summarized the properties and application of period of record for FDCs. This topic is widely studied as evidenced in the review by Smakhtin, (2001)as well as in literatures on the regionalization of FDCs (e.g., Croker K. M., Young M. D. Z. , Rees H. G., (2003), uncertainty analysis of FDCs: Yu, P. S., Yang, T. C., Wang, Y. C., (2002), development of stochastic model for FDCs: Cigizoglu and Bayazit, (2000); Sugiyama, H., Vudhivanich, V., Whitaker, A. C., Lorsirirat, K., (2003), and use of FDCs for watershed management: Good and Jacobs,(2001)).

2.2 Hydrologic Significance of the Flow-Duration Curve

The river measured at a gauging station is the surface outflow of the drainage basin above a specified point on the stream. Thus, the stream flow record integrates the effects of climate, topography, and geology, and gives a distribution of runoff both in time and in magnitude. When the flows are arranged according to frequency of occurrence and a flow-duration curve is plotted, the resulting curve shows the integrated effect of the various factors that affect runoff. A flow duration curve characterizes the ability of the basin to provide flows of various magnitudes. Information concerning the relative amount of time that flows past a site are likely to equal or exceed a specified value of interest is extremely useful for the design of structures on a stream. For example, a structure such as dam for irrigation can be designed to perform well within some range of flows, such as flows that occur between 20 and 80% of the time (or some other selected interval).

The shape of a flow-duration curve in its upper and lower regions is particularly significant in evaluating the stream and basin characteristics. The shape of the curve in the high-flow region indicates the type of flood regime the basin is likely to have, whereas, the shape of the curve in the low-flow region characterizes the ability of the basin to sustain low flows during dry seasons. A very steep curve (high flows for short periods) would be expected for rain-caused floods on small watersheds. Snowmelt floods, which last for several days, or regulation of floods with reservoir storage, will generally result in a much flatter curve near the upper limit. In the low-flow region, an intermittent stream would exhibit periods of no flow, whereas, a very flat curve indicates that moderate flows are sustained throughout the year due to natural or artificial stream flow regulation, or due to a large groundwater capacity which sustains the base flow to the stream

FDC is a key tool for the sustainable management of water resources as evidenced in the works of Pirt and Simpson (1983), Gustard A., A. Bullok and J.M. Dixon ,(1992), DNAF (1995), and Rehango and Joy (1998). The management of abstractions requires estimating FDC from gauged data or from ungauged sites through statistical or physical models in order to obtain the natural as well as the historical regimes that include the impact of abstraction and discharges returned to the river; a target regime that maintains the ecology of the river at an acceptable level (WMO, 2008). Fig 2.1 shows the river abstraction concepts based on FDC.

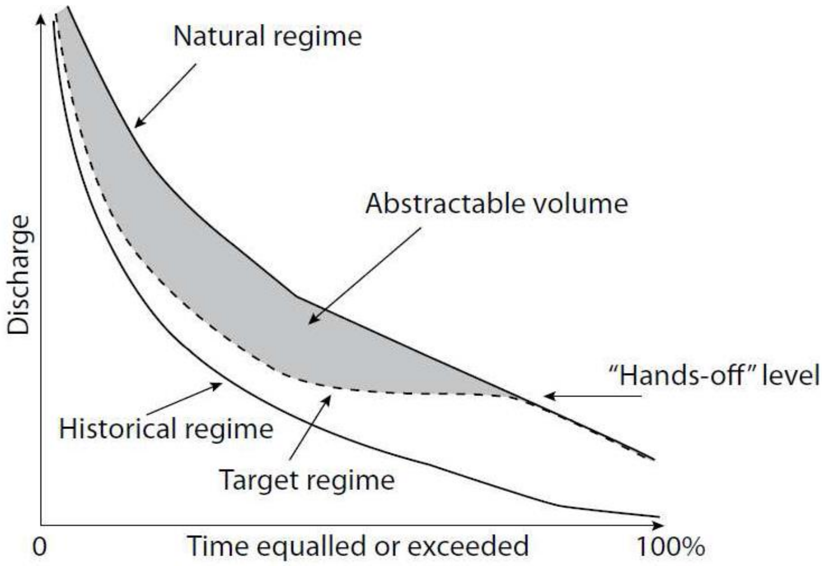


Fig 2.1: River abstraction concepts obtained by an FDC (WMO, 2008).

From FDC estimated by a model for ungauged sites or calculated from the data for the gauged one, it is possible to evaluate the abstractable volume and the hands–off flow. The abstractable volume represents the amount of water that can be withdrawn from a river without creating problems to the river's ecosystem and adversely impact on the downstream uses. The hands–off flow is the discharge after which the abstraction of the discharge has to cease. Another important use of FDC is the analysis of public water supply and agriculture. FDCs are used mainly for the preliminary design of simple abstraction schemes in public water supply project, whilst the primary application of water for agricultural use is to supply water for irrigation. The key planning element is the assessment of water availability for the definition of potential area that can be irrigated by the supply river. The assessment is carried out by considering the critical period of the year. In this case, a seasonal FDC can be used to compare potential demand for available water.

FDC is also used to estimate the dilution of domestic or industrial discharges intended for a river. Water quality models are used to evaluate the quality of the receiving stream and to determine the frequency distribution of the downstream water quality. For instance the Q95 obtained from the FDC is the most used flow variable to evaluate downstream water quality and to determine a constraint on discharge consent. Other important use of FDC is the evaluation of available water for hydropower. In fact, small–scale hydropower schemes use water from the river without artificial storage. The conventional method to evaluate the availability of water in a river is the FDC. The design must evaluate the fluctuating power demands, the protection of downstream abstractors' interest and the ecosystem's health (WMO, 2008). Ecosystem protection is one of the many uses of FDC. Percentiles from FDC can be used to estimate a minimum value of the river discharge in a way that if the discharge falls under this value, abstraction should be reduced or stopped.

Literature has widely referred to the several applications and studies on FDC; for instance, Vogel and Fennessey (1994) showed the several applications of the FDC in the hydrological practice. Similarly Warnile in 1994, showed that FDC can be used for hydropower studies. Male and Hogawa (1984) suggested the use of FDC in the project phase of waste water treatment plants when a trade-off is necessary between water quality requirements and costs. Alaouze (1989, 1991) estimated the optimal release schedule from reservoirs by using a procedure based on FDC. Similarly, Pitman (1993), and Mallory and McKenzie (1993) used FDCs in flow diversions designs. An effective illustration of the practical relevance or application of FDC's for the assessment of river habitants in the estimation of stream flow requirements was done by Estes and Osborn (1986), and Gordon, N.D., McMahon, T.A., Finlayson, B.L., (1992), In the same Hughes and Smakhtin (1996) used a non–linear spatial interpolation approach based on FDC to extend the observed daily time series while Smakhtin, V. Y., Toulouse, M., (1998). and Smakhtin, (1998) employed the FDC concept to generate flow time series; the ideal was to restore the natural stream flow sequence at ungauged sites. More generally, Gustard and Wesselink (1993), Lanen, H.A.J. van, Tallaksen, L.M., Kasparek, L., Querner, E.P., (1997) and Smakhtin *et al*. (1998) used FDC as a tool for rainfall–runoff model calibration and/or for the comparison of flow-time series simulated for different scenarios of development. It is against this background that Hughes *et al*., (1997) proposed a model which is based on FDCs; The model was designed to convert the original tabulated values of estimated ecological instream flow requirements for each calendar month into a time series of daily reservoir releases. The application of FDCs extends to a wide field of hydrology and water resources; e.g., Good and Jacobs, (2001) published works concerning the use of FDCs for watershed management.

2.3 Flow Dynamics and Regimes

2.3.1 Low Flow

In the discussion of flow regimes vis- a-vis the use of FDC, our knowledge of low flows is critical. The low flow analysis is in fact connected with different topics such as the amount of water that is present in a river during the dry season of the year or the length of the time between flood events or the frequency of this period. The choice of the topic depends on the use of the stakeholder groups' needs. In fact, the knowledge of low flows is important in various engineering scenarios such as the development and design of water supply schemes, waste load allocation, design of dams or reservoirs, and the definition of the amount and quality of water for irrigation, domestic and recreation use. Low flows originate from groundwater flow, melting glaciers or surface water coming from lakes. The period of low flows is generally the same for the same region and occurs each year. According to Smakhtin (2001), low flows study is made up of two components: a temporal one that concerns the magnitude, the variability of flows and the length of these events, and a spatial component that is related to the regional distribution of low flows characteristics. This element in particular is dealt with to enhance our capability to obtain these properties in catchments where there is a lack of measured data. To understand and study these elements of low flows, it is necessary to know the natural and anthropic causes of this phenomenon. In fact, several components influence low flows such as climate, geology, soil and topography but also abstractions and regulation of low flow domain through dams. Literature on low flow ranges on different subjects. Various papers are related with the review of low flow study and analysis of different engineering applications (e.g., Smakhtin (2001). These papers approach the causes of low flows, the several techniques used for low flow analysis and define where it is possible to apply this analysis. Besides, they analyzed the different characteristics that are computed by discharge data in order to obtain information on low flows used in different countries.

Low flow analysis is used in wide range of engineering applications often regulated by national and international water laws. One of the most important and common uses of low–flow analysis is the design and operation of public water–supply schemes. In this field it is important to understand whether the water amount present in a river is sufficient for abstractions for different use like water treatment plants or reservoir storage facilities. The analysis of low flows is similar if abstractions or irrigation scheme are planned, although agricultural demand has a higher annual and inter-annual variability. Water abstraction is also necessary for hydro-power use. This field requires an analysis of the complete range of flows, but low flow analysis is essential to decide how much water must bypass a hydro–plant to maintain downstream river ecology, and how much is available for hydropower production in dry season. For all these applications the forecast of flows is necessary to evaluate if restrictions on water use can minimize the risk of lack of water in the future.

Low flow analysis is often used also to estimate dilution of domestic or industrial discharge released into a river. In particular, legal authorizations are needed for discharge waste water. The frequency distribution of downstream water quality of a river is obtained through water quality models that use as input data rate and quality of the discharges. The complement of the cumulative distribution function or river flow duration curve is commonly used for development analysis. Moreover, ecosystems are most vulnerable during the dry season because the reduction of water creates a decrease of dissolved oxygen, the fragmentation of the habitat and the deterioration of water quality. Flow duration curves or percentiles obtained from flow duration curves and low flow indexes as mean annual minima for a given duration are often used in this field. In addition, rivers are also used for sport and recreation activities. In this case the artificial support of water levels and velocities can be important to maintain the attractiveness of a river.

As seen above, low flow analysis is needed in different fields and for each situation, different information is required. Simple indexes can be calculated, as recession constant, the mean value of the discharge time series and the proportion of base flow. Besides, it is possible to calculate more informative characteristics such as cumulative distribution functions of daily flows or to use extreme values techniques. The first one represents the relationship between the discharge and the percentage of time that it is exceeded. The second one is used to calculate the non–exceedance probability of annual minima. The biggest difference between this is that flow duration curves use all the recorded time series and hence evaluate the percentage of time the entire time series is exceeded. The extreme value technique is applied to annual minima data and allows the estimation of the non–exceedance probability in years or the return period when the values are below a given value. These estimates do not permit the forecasting of the period when low flow events will happen. Different methods exist to make long or low term estimation that allows calculating both the magnitude than the time of the low flow event. Obviously, when forecasting time increases, the uncertainty of the prediction increases too and for very long lead times the long term statistical mean gives the best predictions.

Different time scales can be used to perform low flow analysis but it is typically carried out using all available data with the minimum time scales. Sometimes it is necessary to use more appropriate time scales as weeks, months or seasons. For example, in designing and irrigation schemes, the analysis should focus on the season of the year when the abstractions for irrigation will take place. The same can be said for other use and then other period of the year. It could be then appropriate to calculate low flow indexes of different durations as 7-days, 10-days, 30-days and 90 days durations for specific design projects. Besides, low flow problems occur over a wide range of space scales. These range from reaches of the dimension of 100 m within basin of 10 km^2, to basins of 1 million km^2 of area. Usually these kinds of basins in developed countries have a big number of gauging stations and long-time series (often biggest than 50 years). Instead, in developing countries or also where there is a good net of gauging stations, it often happens that continuous observational data is of poor quality and often there is a lack of gauging stations in some reaches or in little basins. In these, cases it is not possible to estimate low flows characteristics directly from data but models could be used to assess low flow analysis. Furthermore, low flow analysis can be done on the scale of river reach, catchment, national and international or globally. In this case, the time and spatial scale of the problem will be a primary issue concerning the approach and the necessary data for the study.

Different procedures and different kinds of data can be applied during water management and supply scheme planning. The choice is done depending on the nature of the required output and the risk associated with the design decision. For example, the construction of a large reservoir will imply a high risk with the design decision. This kind of project needs gauging stations and the analysis of observed river flows. These data are necessary for hydrological design, typically the storage characteristics and spillway capacity. Conversely, a small scale abstraction licence will not have high risk associated with it. It often required in ungauged stations and usual the design will be based on low flow statistics. This complex scenario can be simplified using the "Design Scenario

Cube" (UNESCO, 1997). The three dimensions of the cube represent the three design requirements and are defined in this way:

1 The location of the design problem. This dimension establishes if there is a gauged station or nearby the studied area;
2 The operational requirement of the hydrological design and the financial capacity that determines if simple statistics or a long time series are required;
3 The characteristics of the data that can be natural or necessitate to be naturalized. The catchment water use dimension discriminates between natural or artificially influenced flows.

The cube makes it possible to create eight different possible combinations of these dimensions. Surely from this simple scheme, it is possible to arrive at a more complex situations.

2.3.2 Catchment processes and storage

The flow in a river is the result of complex processes that happen at a catchment scale. It Schematizes the catchment as a conceptual model, constituted by interlinked reservoirs, where recharge, storage and discharge processes take place. While recharge of the catchment mainly depends on precipitation, storage and discharge are functions of physiographic characteristics. Low flows occur therefore after periods of no rain or when precipitation falls as snow. Besides, this decreases the water stored in the soils and also the outflow to the river. The time of depletion depends on hydrological processes that operate near the channel as the storage properties within the catchment. Catchment soil and geology influence the capacity of the catchment's precipitation absorption and release as low flow.

Catchment input is precipitation or snowmelt, so low flow can decrease due to different causes as:

1) Extended dry periods, when potential evaporation is higher than precipitation;
2) Extended periods of low temperature, when precipitation falls as snow.

Warm periods and dry weather periods are associated with high pressure systems, when high temperatures, high radiation input, low humidity and wind increases evaporation and transpiration rates. Conversely, snowfall and snow storage happen during periods of low temperatures, associated with cold, polar air masses and decreasing temperatures. In this situation, the absence of snowmelt creates an accumulation of solid water and then low flows (Bowles and Riley, 1976; Gerard, 1981; Collins, 1982; Fountain and Tangborn, 1985; Gurnell, 1993; Hopkinson and Young, 1998). Winter low flows or summer low flows in mid and high latitude climates occur both annually. In low latitude climates, different dry season and then distinct low flow periods can occur during the year. In arid and semiarid climates, the combination of low precipitation and higher evaporation causes minimal river networks and ephemeral rivers. These kinds of basins are characterized by prolonged periods of zero flows and episodic high flows, often in the form of flash floods. Climate processes have a big influence on low flows, due to the influence on the magnitude and the variation of temperature, the potential evaporation and the precipitation that modifies the input water in the catchment. Climate maps furnish good information to understand low flows conditions of an area. However, climate varies spatially, particularly in mountain regions where there are strong altitude dependent temperature and precipitation gradients and temporally, at inter-annual, decadal and centennial time scales.

Climatic processes influence the increasing or decreasing of water in the catchment, in terms of deficit in one season or surplus in another, the soil vegetation and geology characteristics affect how these surplus and deficit propagate to the stream flow. The most significant properties are soil moisture and groundwater storage, aquifer properties and hydraulic resistance between aquifer and river. In particular, in impermeable urban surface, in sloping non–vegetated land, in compacted soils or exposed rocks it is common that water is stored in micro-depressions, and after having filled these, water goes to the stream-flow. Overland flows depend on precipitation intensity and whether it exceeds infiltration rates of the soils. As the soil is saturated, water may flow vertically towards the aquifer to recharge groundwater storage or move laterally through permeable soil layers toward the stream. Available soil moisture capacity is really important for low flows because it provides for a support to high annual transpiration. By means of macro-pores, cracks on the rocks and pipes, water can recharge aquifer or move toward the stream without recharging the soil layer. In semiarid or arid aquifers, recharge occurs through river beds of ephemeral rivers and originates from high precipitation on mountains regions.

Water arriving from the soil to the aquifers increases the groundwater level. The groundwater discharge to the stream occurs where stream channels intersect the main phreatic surface in a draining aquifer. For low flows to be sustainable:

(i) the draining aquifer must be recharged seasonally with adequate amounts of moisture;

(ii) the water table must be shallow enough to be intersected by the stream; and

(iii) the aquifer’s size and hydraulic properties must be sufficient to maintain flows throughout the dry season (Smakhtin, 2001).

A different example of groundwater re-emergence can occur where relatively slow moving groundwater drainage in fracture zones above the main water table has a significant lateral component which intersects the ground surface in the vicinity of channels (springs) (Smakhtin, 2001).

Geology obviously can at the reverse cause losses of water from the basin. These processes can be synthesized in

(i) groundwater recharge from stream flow where the phreatic surface lies below the channel;

(ii) bed losses, where unconsolidated alluvial material underlies the river channel; and

(iii) losses to relatively dry soils forming the banks of streams (Smakhtin, 2001).

Physiographic characteristics as soil type and geology are therefore the key components for low flows. Many studies reported the presence of a strong correlation between geological category and flow rates or low flow discharges (Nikic and Radoja, 2009). Different studies have found that catchment characterized by unconsolidated sedimentary rocks have normally low yields on low flow periods. On the contrary, metamorphosed sedimentary and igneous rocks showed higher flow values compared to their basin size. Lakes and reservoirs hydraulically connected with rivers have usually a strong impact on low flows. The adequate water level in a lake should be maintained during the dry season to allow lateral outflow into a stream. Exhaustive studies that deepen the effects of lakes on low flows are not wide enough but the importance of lakes for low flows in some regions may be derived from the inclusion of lake related parameters in prediction models for low flows (FREND, 1989).

2.3.3 Hydro-climatic and Anthropogenic Effects on Flow Patterns and Type

The anthropogenic influence is really important for low flows and can cause water increase or decrease in the river. The more important human impact on low flows is abstraction within sub–surface drainage area. This decreases the level of phreatic surfaces and therefore potential re–emergence for groundwater in stream channels. Localised reductions in the level of the water table may affect either hydraulic gradients or the length of channel that intersects the phreatic surface. The effects of groundwater pumping near the head of a perennial river may result in groundwater table depletion through interception of recharge water and induced recharge of the aquifer from the river itself (Smakhtin, 2001). Another important human impact of the flow is the change in forestation. This affects mainly evapotranspiration losses from the soil, affecting gain and losses to alluvial storage. In addition, the presence of vegetation increases interception losses and disturbance of the soil structure. Land use changes affect strongly low flows since it changes infiltration or evaporation characteristics and involves the groundwater recharge (Ferguson and Suckling, 1990). All these actions indirectly affect low flows, but there are processes that directly act on low flows, removing or adding water in the stream. A synthesis of these processes is given here. Water abstractions for industrial, agricultural and domestic use processes decrease the amount of water of the river and affect mainly the dry season and the frequency of these periods (Eheart and Tornil, 1999). It is important to remember that irrigation returns water to river channels and irrigation return flows can be a big part of a stream's water balance, according to some sources (McKenzie and Roth, 1994) up to 40% of water initially used for irrigation returns into the stream. Discharges of water from industrial and domestic sources can worsen the quality of water downstream. That is multiple abstractions and effluent discharges may affect the dry season flow. Construction of dams and river flow regulation regime can increase or decrease low flow discharge levels depending on the operational management of the reservoir. This can be seen as the most important impact on a river's low flow regime (Finlayson, B.L., Gippel, C.J., Brizga, S.O., 1994). Different human impacts can affect low flow regimes and many rivers of the world with perennial regime have become intermittent whilst many rivers have been created artificially. A complete review of the relationship between hydrological processes, low flows and anthropogenic impacts and representation of basin processes is given in Dingman (2002).

2.4 Flow measurements and Analysis

2.4.1 Low flow measures

The number of summary information derivable from daily time series that describes low flow regimes of a river is quite wide. The large number of methods is based mainly on the types of available data and the required output. Furthermore, it depends also on the not clear definition of low flow event, expressed as annual minima, as a threshold discharge or can be indicated by the time that the discharge is lower than a fixed value. Other reasons for diversification are the different methods of expressing the frequency. It is possible in fact, just as in extreme value analysis, considers it as the proportion of time that a discharge is exceeded (flow duration curves) or as the proportion of years that a given low flow occurs. Other methods can be obtained given the different durations or averaging periods that are required in many applications. The domain of low flow measures can be defined arbitrarily. Following Smakhtin (2001), it is possible to specify an upper and a lower bound. Different values can be chosen for the upper boundaries, more or less conservative. The less cautious value is the mean annual runoff (MAR), which is the mean value of the available annual flow totals. Dividing this value by the number of seconds in a year it is possible to obtain the long term mean daily discharge, defined as mean daily flow (MDF). The other possible boundary is the Median Flow (MF) obtained ranking the time series. This value is more conservative because stream flow time series are often positively skewed and MF is often smaller than MAR. The lowest value for the low flow domain can be identified as the absolute minimum value of the daily discharge (AMF).The information content of this value depends on the length of the time series and the presence of a lowest measuring threshold value for the gauging system. The lowest bound for low flow hydrology is surely zero that occurs on intermittent and ephemeral rivers.

2.4.2 Flow Estimation and Scaling

Base flow is an important component of hydrograph obtained from groundwater storage or other sources. To recognize base flow in the hydrograph, it is possible to use separation techniques that permit its division in a quick component and a delayed component. Through most of the dry season, stream flow is composed essentially of base flow, while in the wet season, the hydrograph is the sum of the direct response of the catchment to rainfall and the water stored component.

Base flow separation techniques try to estimate the surface runoff component of a flood and Concentrate on base flow separation starting from flood events (event based methods). Other types of methods try to generate base flow hydrographs from long time series using some kind of digital filters that permits to obtain from the daily stream flow a quick flow and a base flow hydrograph. The aim of base flow separation techniques is to obtain objective quantitative indexes related to the long term base flow response of a catchment. Such indexes include mean annual base flow volume and long-term average daily base flow discharge. However, the main low flow index obtained from base flow is

Base Flow Index (BFI). The index is calculated as the ratio between the volume of the base flow and the volume of total stream flow. The index is one dimensional and range from 0 to 1. The highest values of BFI are obtained in catchments able to sustain river flow during dry seasons and namely where there is a high groundwater contribution. Lowest values of BFI, on the contrary are typical of ephemeral streams. Different authors found characteristic values of BFI for a number of rivers in certain regions (FREND, 1989; Smakhtin and Watkins, 1997).
Base flow index is usually highly correlated with hydrological properties of soil, geology and other storage descriptors, such as lake percentage. Knowledge of base flow regimes is important for the development of catchment management strategies (especially for drought conditions) establishing relationships between aquatic organisms and their environment, estimation of small and medium water supplies, water quality and salinity management, calculating water budgets of lakes, etc, (Smakhtin, 200

The shape of the recession curve, that is to say the falling limb of the hydrograph; reveals the gradual depletion of water stored in a catchment during periods of little or no precipitation. The recession curve reflects how different catchment storages and processes control the river outflow (WMO, 2008). The main governing factor of the stream recession rate is the catchment geology (transmissivity, storativity of the aquifers) and the distance from stream channels to basin boundaries (Smakhtin, 2001). Rivers where groundwater has a high influence have a slow recession rate, whereas a fast rate is representative of rivers draining impermeable catchments with limited storage. Overland, interflow and groundwater flow have different characteristic recession rates, also if the range of these rates can overlap. In low flow context, the base flow recession rate is the more important component. Base flow recession can be expressed by various recession equations (FREND, 1989) but the most common is the exponential one identified by a recession constant which is the parameter of the model. There are two main estimation methods used to calculate this index and they mainly consist the computation of the slope of a master correlation curve (MRC) obtained between the envelope of different recession curves, or in performing a separate calculation of parameters for each separate recession segments (IRS).

The MRC method tries to overcome the variability in individual segments by constructing a mean recession curve. Different methods exist to calculate a master recession curve such as the correlation method (FREND, 1989) and the matching strip method (Sugiyama, 1996). Recession analysis is used for a wide range of engineering applications like water resource assessment, planning and management. Usually this technique is used for irrigation, water supply, hydroelectric power plants and waste dilution, estimating groundwater resources in a catchment, and hydrograph analysis (Bako and Owoade, 1988; Korkmas, 1990) used recession curves for estimating the available drainage storage. Two important uses of recession constant are those of Kelman (1980) that through these values fitted a stochastic daily stream flow model and Kottegoda *et al*., (2000) that applied recession characteristics for the generation of continuous daily stream flow time series. A complete recent review of stream flow recession analysis and its various applications has been treated by Tallaksen (1995).

Most statistical methods are concerned with extreme values, in particular with minimal values in low flow analysis. The low flow frequency analysis obtains the proportion of years when a flow is exceeded (return period or recurrence interval). The Low Flow Frequency Curve (LFFC) is constructed on the basis of historical records of annual flow minima (daily or monthly minimum discharges of flow volume). Usually different theoretical distribution are used to forecast beyond the limits of observed probabilities and to improve low flow estimates because the available observed flow records are not sufficient. In this case it is not possible to know the true parent distribution for low flows but it is important to associate reasonable distribution and estimate its parameters. The fitting procedure consists of different graphical and statistical methods that are used to decide the best theoretical distribution function. The distributions usually applied in literature are different forms of Weibull, Gumbel, Pearson Type III and log-normal distributions.

One of the main issues of low flow frequency analysis is the presence of observed stream flow time series of zero flow data. Zero data are typical of arid climates and very cold regions where the streams can be frozen in winter. Zero values can be present in a record also because the gauging station has a stream flow limit and the river level is below this threshold (censored data). The presence of zero flows has an important effect and it is not possible to ignore it in the statistical analysis of low flow series. Distribution fitted to series with zero flows will result in a positive probability of negative stream flows unless the distribution is explicitly constrained to have a lower bound of zero. Such results are physically meaningless (Smakhtin, 2001). In addition, the flexibility of distribution is reduced constraining it to zero lower bound In arid and semiarid regions, rivers are often intermittent or ephemeral and the annual minimum value is often zero so in these cases, it is not possible to apply a low flow frequency analysis. It is possible to obtain many information and indexes by LFFC. The more important indexes estimated by LFFC are:

1 slope of LFFC;

2 break in the curve near the modal value;

3 7-day 10 year low flow (7Q10) and 7 day 2 year low flow (7Q2);

4 Dry Weather Flow.

The slope of LFFC can be estimated as the difference between two flow values, normalized by catchment area, from low and high probability domain. The steepness of the slope indicates the variability of low flow regime. The LFFC can present a break in the curve near the modal value. This can be identified as the point where a change in drought characteristics occurs. Flows characterized by higher frequencies are those that have a tendency to normal conditions and cannot be considered as drought flows. Though not a general feature of the LFFC, this may be interpreted as a condition at which a river starts getting water exclusively from a deep subsurface storage (Smakhtin, 2001). The 7-day 10 year low flow (7Q10) and 7 day 2 year low flow (7Q2) are the most used indexes in the USA. These values represent the lowest average flows that occur for a consecutive 7-day period at the recurrence interval of 10 and 2 years.

The dry weather flow is the annual series of minimum 7 day average flow (Gustard A., A. Bullok and J.M. Dixon ,1992). It is used mainly in the UK for abstraction licensing. The 7 days average permits to eliminate the problem of day by day variability of the river flow and also measurement errors. Low flow frequency analysis belongs to extreme events frequency analysis but the study of it is limited compared to flood frequency analysis.

Information about the length of period below a selected threshold value is not provided by LFFC, neither by flow duration curves. Furthermore, these measures lack information concerning the stream flow deficit that can be obtained through a continuous time series. In fact, the construction of FDC excluded this information. Two time series that have a very different low flow sequence can have a similar shape of FDC. There are various existing approaches to overcome this problem. For example it has been recommended to evaluate the durations of the longest periods which are necessary to obtain a defined small percentage (1–10%) of the MAR. These indexes are similar to those obtained by FDC but without losing information on the time sequencing of discharges. It is necessary to calculate these intervals for every year rank these values and plot them in different ways.

Another approach is based on the theory of runs (Fleig, A., Tallaksen, L.M.,Hisdal H. and demuth, S.,2006) called threshold level method. This approach is used for yielding estimates of the frequency of low flow periods and is used in designing reservoirs where reservoir releases are made to support downstream abstractions. The run in this context is the number of days when a discharge is below a defined threshold flow. The method, called also threshold level method or "method of crossing theory", is applied to a global dataset in Fleig *et al.*, (2006). The threshold value considers the purpose of the study, the type of flow regime and the available data. For perennial flow regimes, usually it is used as limit value of the discharge with 70-90% exceedence on FDC. Tate and Freeman (2000) instead suggested using the discharge with 20% exceedence on FDC as threshold in ephemeral rivers where there is a flow only after significant rainfall event. An alternative is to use a variable threshold; i.e., by defining the truncation level *Q0*, it is possible to define also the deficit characteristics; viz:-

1. The duration, which is the period of time where the flow is below the threshold level. This period is defined as drought duration, low flow spell, or run length.
2. The volume or severity that is the cumulative water deficit.

3. The intensity, also defined as drought magnitude, that is the ratio between deficit volume and deficit duration;
4. The minimum flow of each deficit event using the series of deficit characteristics.

It is possible to derive other indexes as the average deficit duration or average deficit volume. Additionally, it is possible to rank these data, assign a probability or return period through plotting position by plotting them against an assigned return period fixed as the threshold value. It is then possible to associate a probability distribution to these points. They used daily stream-flow time series to obtain all components of drought deficit, duration, start date, number of continuous events in a given time interval, the largest stream-flow deficit and the largest duration in a given time interval. The Institute of Hydrology (1980), using different names, handled the same problem. Smakhtin and Watkins (1997) suggested analysing the durations (or deficits) and number of spells below a threshold flow and to display the results in the form of a histogram and/or a cumulative frequency curve. These methods give a quick impression of how responsive the river is on the basis of the available record.

The use of this approach is required in those fields where continuous available river discharge is needed for domestic water supply, irrigation, power generation, dilution of industrial pollutants, recreational planning, fish migration etc. Moreover, it is used to assess the storage capacity of a reservoir (McMahon and Mein (1986); in this manner it is also possible to study in ephemeral rivers the frequency of durations of continuous zero-flow periods using common statistical procedures. This problem is a specific case of run analysis which indicates the likelihood of extended periods of no–flow or drought (Armentrout and Wilson, 1987).

2.5 Development of Flow Duration Curves and Regionalisation

2.5.1 Construction of Flow Duration Curves

Two main methods exist for the construction of flow duration curves; these are, namely:

a. The total period method (FDC);

b. The calendar year method (AFDC).

Annual flow duration curve (AFDC) is constructed by ranking the data in a decreasing order of magnitude, assigning flow values to class intervals and calculating the number of occurrence in each class. Afterwards, it is possible to calculate cumulated class frequency and express them as a percentage of the total number of the time steps in the record period; lower boundaries of every discharge class are plotted against the percentage point (Ubertino, 2010). Another method to calculate these curves is through the definition of FDC as the complement of the cumulative distribution function based on the complete period of record (Ubertino, 2010) introduced a variety of non-parametric approaches for calculating the FDC. The procedure for the determination of the curve is simple. It takes the form as enumerated below:

1. rank the observed discharges data in ascending order;
2. plot each observation *Qp* (p-th quantile of daily stream flows) versus its duration (*Di)*or exceedence probability (*p)*. The excedeence probability *p* is expressed as:

$$p = 1 - P\{Q\,p \;\leq\; q\} \qquad 2.1$$

It is possible to calculate *p* using the Weibull plotting position after which it is possible to define the duration *Di* as:

$$Di = 100(p) = 100\times(1 - \frac{i}{n+1}), \text{ for } i = 1, 2, 3, \ldots, n \qquad 2.2$$

where n is the length of the sample.

The calendar year method or annual based flow duration curve is as reformulated by LeBoutillier and Waylen (1993), Vogel and Fennessey (1994).

The computational framework of the AFDC is as following:

a. For each year or hydrological year, FDC is calculated using the procedure for the period of recorded FDC;
b. For each exceedence probability p using all annual FDC, it is possible to calculate the measures of central tendency, namely mean and median (by summarizing the inter annual variability of the different annual FDC).

Starting by this construction, Vogel and Fennessey (1994) also introduced confidence intervals for quartile of flow duration curves. This is done because when one focuses on a particular quartile or percentile of an FDC, the FDC does not, by itself, expose the uncertainty associated with a particular quartile estimate (Vogel and Fennessey, 1994). The construction of non–parametric confidence intervals can be easily done considering the n values of discharges for each order as a random sample from which one can estimate $100(1 - p)\%$.The AFDC and the FDC are complementary rather than competitive concepts (Castellarin, 2004). FDCs shows the entire range of observed river discharges and, Fennessey (1994), Hughes and Smakhtin (1996), and Smakhtin *et al.*, (1997) showed that they can be effectively used for filling gaps and for extending daily stream flow series, and, when a regional FDC model is available, to generate stream flow series at ungauged river basins. This kind of curve presents, however, a negative implication. In fact the definition of quartile Q_p, calculated using the procedure of the period-of-record FDC, is the discharge that was exceeded p percent of the time over the entire period of record on which it is based. This differs from flood and low-flow frequency analysis, where the interpretation of the frequency curve does not depend on the period of record, (Vogel and Fennessey, 1994). Obviously, the number of years used for the frequency analysis leads to less sampling errors in quartile estimates, and therefore to more precise frequency curves.

It suffice to note that AFDC have been shown to be quite useful for making probabilistic statements about wet, typical and dry years, for computing confidence intervals associated with the AFDC representing the typical hydrological condition and for assigning return periods to individual AFDCs (Vogel and Fennessey, 1994). Besides, the median annual FDC is not influenced by the occurrence of extreme low-flow periods or extreme floods over the period of record, yet it still captures the frequency and magnitude of daily stream flow in a typical year. Since their introduction, a number of investigators have found AFDCs to be quite useful to solve a wide range of problems (Claps and Fiorentino, 1997); Smakhtin and Toulouse, (1998); Good and Jacobs, (2001); Sugiyama *et al.*, (2003). Different stochastic models are created to relate FDC and AFDC. The stochastic models are formed by a deterministic component that must reproduce the seasonality associated with daily flow series and by a stochastic component that must reproduce both the persistence and frequency distribution of the daily flow series.

LeBoutillier and Waylen (1993) introduced a five parameter stochastic model of daily streamflows which relates the FDC to the AFDC. This stochastic model developed by LeBoutillier and Waylen (1993) can reproduce the AFDC for a typical year though it significantly underestimates the variability of observed AFDCs around the central AFDC. Later, Castellarin *et al*. (2004) developed a mathematical model based on the index flood method that relates the period of record flow duration curve (FDC) to the mean and variance of the annual flow duration curve. This approach allows for the construction of

1. confidence intervals associated with AFDCs at ungauged sites,
2. assignment of return periods to individual AFDCs
3. development of regional models of flow duration curves,
4. generation of daily stream flow series at un gauged sites, and
5. development of a generalized stochastic model of daily stream flow (Castellarin *et al*., 2004).

Other probabilistic and parametric representations of FDC have been suggested by Quimpo, R. G., Alejandrino, A. A., McNally, T. A., 1983, Mimikou and Kaemaki (1985), Fennessey and Vogel (1990).

2.5.2.1 Data Type and Requirements

FDC can be constructed using different time resolutions: daily, weekly, monthly and annual. The most informative time resolution is the daily one. The construction of the FDC can be done also using other time resolution, more appropriate to the studied problem by using the moving average method and then calculating these curves using m-day or m-months average time series. The choice of the time unit, such as the day, the week or the month depends on the use of the graph. The details of the variations in flows are obscured if the time unit is long. Using monthly or annual data could be unsatisfactory for many streams because it may not permit the evaluation the variability of the flow.

The effects of varying time unit are different depending on the different river regime. If the flow of the river is regular day by day, the change of the time, from daily to weekly to monthly time unit will not change significantly the curves. On the other hand, on “flashy” rivers characterized by sudden flows that last only few hours, the change of time unit will have a really large influence.

2.5.2.1Temporal Resolution and Standardization

To improve the readability of the curve, discharge is often plotted on a logarithmic scale and sometimes the frequency axis is represented on a normal probability scale. When the logarithm of daily mean flow is normally distributed, the plot will follow a straight line. This procedure facilitates the readability of the graph and comparison of different curves. Standardization through division by the average flow is used to compare the regime of different basins. The shape of FDC is connected with the physiographic and climatic characteristics of the basin, and therefore to the catchment response (WMO, 2008). A time series with low variability is reflected in a flat curve typical of permeable catchments, or with a strong regularity caused by the presence of storage.

The FDCs characterized by steep curve are typical of impermeable catchments with high variability and sudden flows. The shape and general interpretation of any FDC has been directly or indirectly illustrated by Vogel and Fennessey (1994), Hughes and Smakhtin (1996), Mngodo (1997) and Smakhtin et al. (1997).

2.11.3 Low flow indexes obtained by FDC

Various indexes can be estimated by FDC. Particular importance is the low flow section of the FDC which is the part of the curve below the Mean Flow or *Q50*. This part of the curve gives information about the storage capacity of the river and the contribution of the base flow to the stream flow. The slope of the curve, steep or flat, indicates the base flow contribution and the permeability of the basin.

Other indexes can be extracted from FDC. In particular *Q95*, the 95 percentile flow, or rather the flow that is exceeded 95% of the period of record is a key index of low flow. The percentile used as low flow index depends on the type of the river being studied. However, the flows within the range of 70-99% time exceedance are usually widely used as design low flow. Some used example indices are: one- or n-day discharges exceeded 75, 90, 95% of the time called *Q75(7), Q75(10), Q90(1), Q95(1), Q95(10)* . Other indexes can be estimated to evaluate stream flow variability such as the ratio *Q20/Q50* and the ratio *Q50/Q90*, which in particular represent the low flow variability. The percentage of time that a stream is at zero flow conditions, indicates the intermittency characteristics of a river (Gorgens and Hughes, 1982; Smakhtin *et al.*, 1995).

2.5.3 Regionalization of flow – duration curves.

As previously outlined, the lack of stream gauges and the limited amount of stream flow observations characterize several geographical areas around the world. This condition, along with the world wide pursuit of the optimal estimation and management of water resource, led to the formulation and proposal of numerous procedures for regionalizing FDC, whose common objective is the estimation of FDCs at ungauged river basins or the enhancement of empirical FDCs constructed for stream gauges where only a limited amount of hydrometric information is available. Different possible approaches to classify these methods exist and surely all arbitrary (WMO, 2008), Castellarin *et al*., (2004), furthermore, because sometimes the different approaches are the sum of different techniques. FDC regionalization can be analyzed by various methods. Yu *et al.*, (2002) divided FDC methods into two groups. The first uses mathematical equations or statistical distributions to fit FDC constructed from gauged data. The second method is the regression between the discharge of some specific exceedance percentage (10, 20, 30,…, and 90%) with the catchment characteristics or annual average flow. Castellarin *et al.*, (2004) classified regionalization procedures into three categories: statistical, parametric and graphical approaches. The first category view FDC as the complement of the cumulative frequency distribution. The second and the third are the procedures which do not make any connection between FDC and the probability theory. Smakhtin *et al*., (1997) suggested that the regionalization technique is preferable in small-scale water projects because of cost and time savings. Regional flow duration curves can be constructed by using the available data of the regionalization techniques, which include stream flow data recorded at other existing gauging stations in the same region.

2.5.3.1 Concept

During the past decades, various models of FDC have been developed. Quimpo *et al.,* (1983) proposed an exponential model for daily FDC in the Philippines. Mimikou and Kaemaki (1985) proposed to use a cubic model for monthly FDC in the western and northwestern regions of Greece. Franchini and Suppo (1996) proposed to use an exponential equation for calculating average daily discharge in a limestone area of the Molise in Italy. Yu *et al*., (2002) found that the cubic model was also fitted well with the daily FDC in the upstream catchments of the Cho- Shuei Creek in central Taiwan. The model that is appropriate for a specific area should use data from its own area The various models proposed by previous researchers, i.e. Quimpo *et al*., (1983), Mimikou and Kaemaki (1985), Franchini and Suppo (1996), and Yu *et al*., (2002), have different results.

2.5.3.2 Development Framework for Regionalised Models

A stream gauge is rarely located at the desired hydropower site. Thus a regional model is needed to transfer stream flow magnitude from gauging stations to a desired hydropower site. In various countries, researchers have presented studies of the development of a regional model. Mimikou and Kaemaki (1985) proposed to use four parameters in the regional model of FDC in northwestern Greece. The four parameters were:

1) the drainage area;

2) the mean annual precipitation;

3) the hypsometric fall; and

4) the length of the main river course, from the divide of the basin to the measuring station.

Franchini and Suppo (1996) divided a regional analysis area in a limestone region of the south of Italy into 2 groups. Group 1 is the site that gives a clear downward concavity FDC. Group 2 is the site that gives a negative exponential behaviour FDC. In Group 1, the regional model consisted of four parameters which were:

1) the area of topographic basin;

2) the limestone area;

3) the mean annual rainfall excess; and

4) the mean flow in the month of minimum flow.

In group 2, there were five parameters in the regional model which were the area of topographic basin, the limestone area, the mean annual rainfall excess, the total limestone area contributing to the outlet, and limestone area inside the topographic basin. Yu *et al.*, (2002) compared cubic polynomial and area-index methods to model FDC at the upstream catchments of the Cho-Shuei Creek in central Taiwan. The cubic polynomial method is composed of annual rainfall, altitude, and drainage area parameters, whereas the area-index method is composed of only drainage area parameter. They found that the cubic polynomial method can give better prediction of FDC at ungauged sites than the area-index method, and that the cubic polynomial method has less uncertainty in estimating FDC than the area-index method Castellarin *et al*., (2004) studied the estimation of FDC in eastern-central Italy by using three models, which were the statistical method by Fennessey and Vogel (1990), the parameter method by Quimpo *et al*., (1983), Mimikou and Kaemaki (1985) and Franchini and Suppo (1996), and the graphical method by Smakhtin *et al*., (1997). For the parameter method, they used six parameters in the regional models, i.e. the drainage area, the permeable portion of the drainage area, the difference between mean and minimum elevations in metres above sea level, mean annual precipitation, and mean annual net precipitation. All regional models were cross-validated by means of a jack-knife resampling procedure. The results showed that the reliability of the three best performing models was similar to one another, and the empirical FDC based on limited data samples provided a better fit of the long-term FDC than the regional FDC. Castellarin *et al*., (2004) developed an index flow model which is the relationship between FDC and annual FDC of daily stream flow data in the eastern central of Italy. This model assumes that the daily stream flow is equal to the annual flow multiplied by a dimensionless daily stream flow. The authors defined annual flow as index flow. Other researchers (e.g. Quimpo *et al*., 1983 and Singh, R.D., Mishra, S.K., Chowdhary, H., 2001) proposed the use of only the drainage area in the regional model. Nevertheless Mimikou and Kaemaki (1985) found that the most significant parameter is the drainage area. According Castellarin *et al*. (2004), a reduced number of parameters is an important prerequisite for the regionalization of FDC. For practical proposes, the model should be kept as simple as possible and should not contain parameters beyond those that are necessary.

2.5.4 Analysis of Flow Duration Curves and Regionalised Models

The shape of the flow-duration curve is determined by the hydrologic and geologic characteristics of the drainage area, and the curve may be used to study the hydrologic response of a drainage basin to various types and distributions of inputs, i.e. snowmelt or rainstorms, or to compare the responses of one basin with those of another. A curve with a steep slope throughout results from stream flow that varies markedly, and is largely fed by direct runoff, whereas a curve with flat slope results from stream flow that is well sustained by surface releases or groundwater discharges. The slope of the lower end of the duration curve, i.e. low flow characteristics, shows the behaviour of the perennial storage in the drainage basin; a flat slope at the lower end indicates a large amount of storage and a steep slope indicates a negligible amount.

In unregulated streams, the distribution of low flows is controlled chiefly by the geology of the basin. Thus, the lower end of the flow-duration curve is often used to study the effect of geology on the groundwater runoff to the stream. Previous researches also show that, climatic and basin characteristics are veritable tools in the analyses of flow durations. The shape of the curve is determined by rainfall pattern, catchment size and physiographic characteristics, land use type, and the state of water resource development. Vogel and Fennessy (1994) stated that the shape of FDC was dependent on catchment area characteristics, particularly hydrogeology. Most previous research on approaches for determining FDC models depended on the availability of flow record in each studying area and the method to analyze the flow records. Variations in climate, mainly the type, quantity, intensity and frequency of precipitation, have a dominant effect on flow. In fact, the accuracy of the FDC model of an ungauged site is subject to the analysis of the record data from nearby gauging stations and it relies on local physiographic factors.

CHAPTER THREE

3.0 Materials and Methods

3.1 Materials

3.1.1 The Study Area

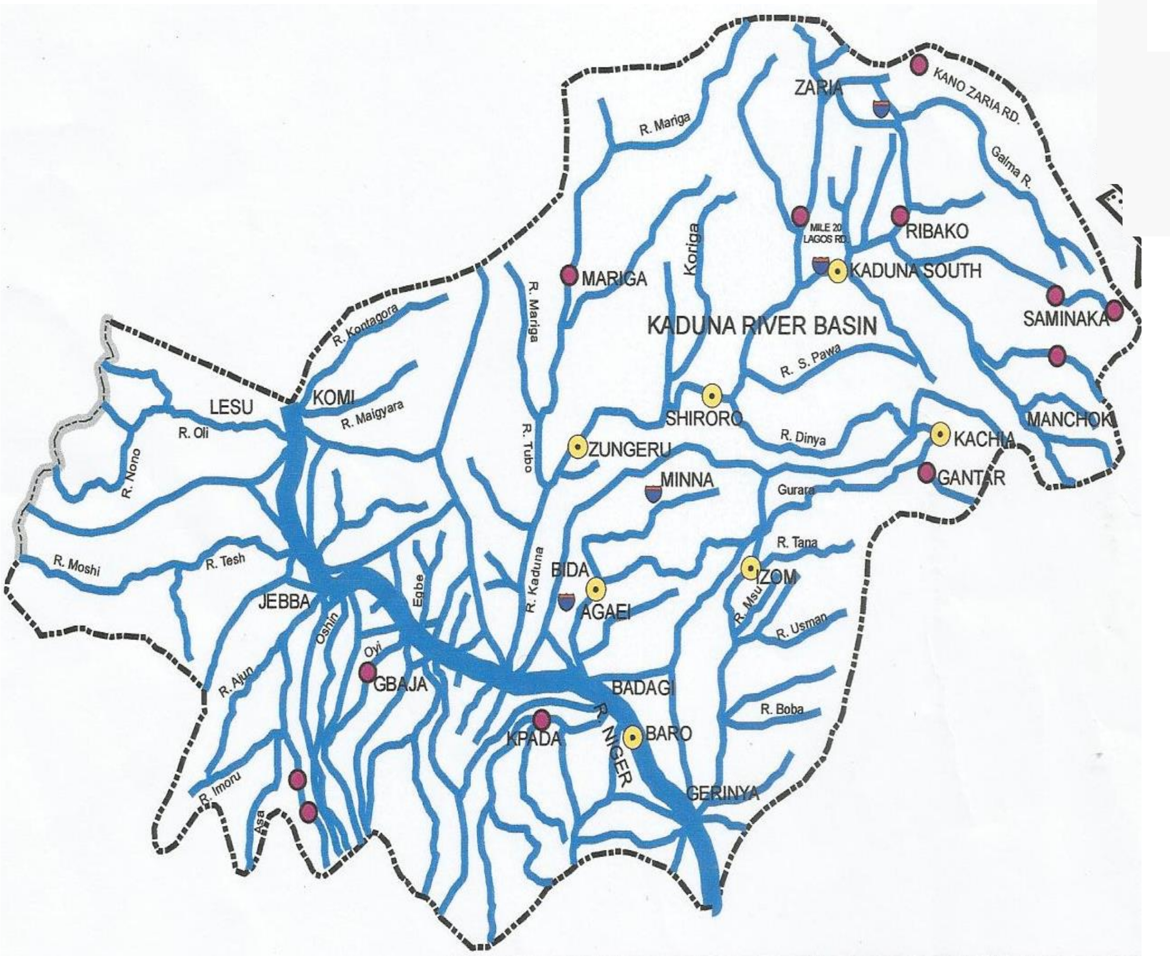
Three sub–basins of the Upper Niger River basin were selected for the study. These sub-basins, for the purposes of this study are

(1) Kaduna river basin

(2) Gbako and

(3) Gurara river basin

Figure3.1 below shows the various locations of the gauging stations within Niger State;



i.e.,

Fig 3.1: The gauging stations in the upper Niger River basin with respect to the three rivers mentioned above.

3.1.1Location and Hydrology of study Area

The Gurara River extends over a distance of about 570km from the plateau at an elevation of over 700m, through Jere at 530m and into the Niger confluence at an elevation of 40m. The Gurara River flows in a general direction of northeast to southwest in its upper reaches, and then turns southwards as it flows through Federal Capital Territory (FCT) to its confluence with the Niger. The vegetation is basically savannah (southern Guinea Savannah zone) grassland interspersed with remnants of tropical forest. The watercourses are particularly forested with large trees from the fringing forests, with a few patches of typical natural forest reserves.

The Kaduna River takes its origin northwest of Jos. The river flows in westerly and south westerly directions from the Plateau, at an elevation of 1500m through Kaduna town at an elevation of 633m, then through Shiroro George and into the Niger. The major left-hand tributaries of the Kaduna River are the Sarkin River and the Dinya River, while the major right-hand tributary is the Tubo River. The entire catchment has a vegetative cover classified as Guinea Savannah. The Gurara, Gbako and Kaduna River basins lie in the intermediate zone between semi- arid climate in the north and sub-humid climate in the south; the climate is influenced by the seasonal movement of the Inter tropical Convergence Zone, which results in wet and dry seasons. Rain starts in May (or April) and lasts till October, with the peak rainfall occurring in September. The dry season lasts between November and March with the mean annual rainfall of some locations in the scheme as follows: 1,300 mm (Minna), 1,500 mm (Abuja), 1,600 mm (Kafanchan), 1,250 mm (Kaduna) and 1,400 mm for Jos. The mean monthly maximum and minimum temperatures in the basins are 37.3°C and 19.7°C, respectively, with the hottest months being February, March and April.

A total of 7 gauged sites in the three selected river within the river basin controlling an area ranging from 900km^2 to 6200km^2 were used for the study. Records of average monthly gauged flows for the respective rivers were obtained from Niger State and Kaduna State Water Boards as well as PHCN. Records from gauge stations at Kachia and Izom (Gurara sub-basin), Kaduna, Shiroro and Zungeru (Kaduna sub-basin and Agaie in the Gbako sub- basin were collected and used

3.2 Methods

3.2.1 Data Collection and Quality Test

The study entailed using short stream flow records in some stations so consistency test was performed to ascertain the integrity of the data collected.

3.2.2 Development of Regular Flow Duration Curves

Details of stations (name, area and the period of the records) are as given in Table 3.1

The records for the seven stations were divided into two for the development of the FDC and regionalisation procedure. This split sampling approach required that some segments of the data were used for calibration and validation, respectively. To achieve this, five stations and the remaining two were used for calibration and validation, respectively.

Table 3.1: Calibration and validation data delineation

S/N	Station	Data length	Catchment Area (km^2)	Characteristics
1	Kaduna	28yrs	1647	Calibration
2	Shiroro	11yrs	3500	Calibration
3	Kachia	25yrs	4020	Calibration
4	Izom	24yrs	6200	Calibration
5	Baro	53yrs	5300	Calibration
6	Zungeru	17yrs	1750	Validation
7	Agaie	26yrs	900	Validation

In the development of the FDC, catchment area was taken as the major characteristics in all the models.

The FDCs were constructed by reassembling the flow time series values in the decreasing order of magnitude assigning flow values to class intervals and counting the number of occurrences (time steps) within each class intervals. Accumulated class frequencies were then calculated and expressed as a percentage of the total number of time steps in the record period. To this end, the lower limit of every discharge class interval was plotted against the percentage points and the discharges of exceedance percentage $Q_{P\%}$ (p= 1,5,10.............99) for each catchment were calculated using specific FDC

3.2.3 Regionalisation of Flow Duration Curves

Based on the submissions of Castellarin *et al.*, (2004), two methods were employed in the regionalisation protocol. These were (1) parametric and (2) statistical methods

3.2.3.1 Analytical Approach

In this approach, analytical functions were fitted to empirical flow duration curves in order to regionalize it. To do this, three basic probability distribution were considered; the logarithmic, power and exponential.

Mathematical models of the flow duration curves were developed based on the logarithmic, power, and exponential transformation framework. Resulting from this, the following equations were employed corresponding to the respective framework; that is, equation (3.1- 3.6)

$$Q = a_1 + a_2 \ln(D) \qquad 3.1$$

$$Q = b_1 D_2^{b} \qquad 3.2$$

$$Q = c_1 e^{c_2 D} \qquad 3.3$$

$$\frac{Q}{\bar{Q}} = d_1 + d_2 \ln(D) \qquad 3.4$$

$$\frac{Q}{\bar{Q}} = e_1 D_2^{e} \qquad 3.5$$

$$\frac{Q}{\bar{Q}} = f_1 e_2^{f\,D} \qquad 3.6$$

where *a* – *f* are the coefficients.

Q is the discharge; D is the Duration and $\bar{Q}$ is mean discharge.

By using the regression analysis, the models in Equations (3.1 – 3.3) and Equations (3.4 – 3.6) were fitted to each set of paired values of *Q* versus *D* and $Q/\bar{Q}$ versus *D*, respectively. The choice of a particular candidate model was based on the high value of R^2 (Regression coefficient of fit).In this case , R^2 values for model equations (3.1 −3.3) were found to be almost same as those of equations (3.4 −3.6) .logarithmic and exponential models shows the highest value of average R^2, model developing is created from data in Table 3.1 by plotting the relationship between drainage area and coefficients from the logarithmic and exponential models. The spatial variation of coefficients have been attempted to relate with the drainage area. Table 3. 3 shows the coefficients a_1, a_2 and d_1, d_2 from the logarithmic models in Eq. (3.1) and in Eq. (3.4), respectively, and the coefficients c_1, c_2 and f_1, f_2 from the exponential models in Eq. (3.3) and Eq. (3.6), respectively, of each station. The spatial variation of coefficients have been attempted to relate with the drainage area.

The drainage area (A) and the coefficients are plotted to identify the relationships. The relationships between drainage area and these coefficients can be fitted by the straight-line equation(Fig. 3.1 and 3.2). The straight-line equations of the coefficients can be written as follow

$$a_1 = j_1 + j_2A \tag{3.7}$$

$$a_2 = j_3 + j_4A \tag{3.8}$$

$$d_1 = k_1 + k_2A \tag{3.9}$$

$$d_2 = k_3 + k_4A \tag{3.10}$$

$$c_1 = l_1 + l_2A \tag{3.11}$$

$$c_2 = l_3 + l_4A \tag{3.12}$$

$$f_1 = m_1 + m_2A \tag{3.13}$$

$$f_2 = m_3 + m_4A \tag{3.14}$$

The straight-line coefficients (j_1 to j_4, k_1 to k_4, l_1 to l_4 and m_1 to m_4) are determined using the regression analysis.

To obtain the regional model, the stream flow data as given in Table 3.1 was used for model development and verification (validation). To surmise, stream flow records for Zungeru and Agaie station were used for validation of the regionalised models based on the conventional FDCs. The coefficients of equations (3.1 – 3.6) were re-parametised as a function of drainage for effective regionalisation. The coefficients (j_1 to j_4, k_1 to k_4, l_1 to l_4 and m_1 to m_4) are determined using the regression analysis. The discharges (Q) corresponding to percent of time (D) at the interval increasing 1 % each time up to 100 %, for each station were computed and compared between the results from the logarithmic and exponential models to find the best fitted model. Based on the re-parameterisation, equation (3.15) – (3.18) were obtained; i.e., for the respective logarithmic and exponential schema.

$$Q = (j_1 + j_2A) + (j_3 + j_4A)\ln D \qquad 3.15$$

$$Q = (I_1 + I_2A)\exp\big((I_3 + I_4A)(D)\big) \qquad 3.16$$

$$\frac{Q}{\bar{Q}}\ (k_1 + k_2A) + (k_3 + k_4)\ln(D) \qquad 3.17$$

$$\frac{Q}{\bar{Q}} = (m_1 + m_2A)\exp\big((m_3 + m_4A)(D)\big) \qquad 3.18$$

where values for the constants k_1 to k_4, l_1 to l_4, m_1 to m_4 and n_1 to n_4 are presented and discussed in chapter 4. The estimation of each sub-basin's representative average flow (Q) in Equations 3.20 and 3.21 was performed by analysing the relationship between mean annual flow and drainage area, written as

$$Q = aA^b \qquad 3.19$$

where A is drainage area in km^2, a and b are constants.

The regionalised models based on the re-parameterisation procedures, were obtained as in equations (3.20) and (3.21)

$$Q = (I_1 + I_2A) + (I_3 + I_4A)\ln(D)\ x\ aA^b \qquad 3.20$$

$$Q = (m_1 + m_2A)\ exp((m_3 + (m_4A)(D))\ x\ aA^b \qquad 3.21$$

where the constants l_1 to l_4, m_1 to m_4, and a, b, respectively.

To ascertain the adequacy or otherwise of the regional models, model calibration and validation were done for both the Analytical and Stastical approaches. The accuracy of regional models was examined by using the measured discharges. In order to do this, measured and simulated results were compared in terms of root mean square relative error (E_R)

$$E_R = {}^{100}\sqrt{\frac{\sum_{D=1}^{100}\left(Q_D - Q_{Dm}\right)^2}{\sum_{D=1}^{100} Q_{Dm}^2}} \qquad 3.22$$

where D is the percent of time between 1 to 100 %, Q_{Dc} is the computed discharge at any percent of time Q_{Dm} is the measured discharge at any percent of time. Using the coefficients J_1 to J_4, K_1 to K_4, l_1 to l_4 and m_1 to m_4, the constants a, b for the drainage area of each station were determined at interval of 1 %, for each step or percentage, of time (D).

3.2.3.2 Stastical Approach

Here, five gauged stations were chosen for the development of the regional flow duration curve, whereas the remaining two stations were used for validation of the regionalised model. Each FDC for the five calibration catchments was constructed using monthly flows; the discharges of exceedance percentage $Q_{P\%}$ (p=1,5,10,20,30,40,50,60,70,80,90,95,99) for each catchment were calculated using each FDC. (Observed discharges are shown table 4.1) In this case, the regional flow duration curve comprised a family of regression equations relating the flows of various exceedance percentages $Q_{P\%}$ with the catchment area. This approach is generally expressed in the the form as equation (3.23).

$$Q = a_1 A^{a_2} \qquad 3.23$$

where A is catchment area, a_1 and a_2 are respectively correlation coefficients. The efficiency of model prediction was computed according to equation (3.24)

$$CE = 1 - \frac{\sum_{i=1}^{N}\left(Q_{pi}^{s} - Q_{pi}^{o}\right)^2}{\sum_{i=1}^{N}\left(\overline{Q_p^o} - Q_{pi}^{o}\right)^2} \qquad 3.24$$

where CE is the coefficient of efficiency at the *p*th exceedance percentage, N=5 Q^o is the discharge of the exceedance percentage P% at the *i*th catchment in the observed flow duration curve, Q^s is the discharge of the exceedance percentage p% at the *i*th catchment in the simulated flow duration curve $\overline{Q_p^o}$ is the mean value of all five catchments. Stream flow data for the remaining stations, Zungeru and Agaie were then used respectively for the validation (verification) of the model.

3.2.4 Reliability Assessment

Reliability Indices (i.e., flow durability) considering the fact that the slope of the flow duration curve is a Quantitative index or measure of the variability, the flow available at fifty (50) percent of the time (q_{50}) and the flow available ninety (90) percent of the time (q_{90})were used. The choice of q_{90} was predicated on the fact that it can be used as the prime power as well as the q_{50} can be used as an index of the power potential with storage. Thus in this study, the two indices combined, indicate the probable flow variability and power potential. Model performance considering extreme flow scenarios was raised by employing the coefficient of correlation statistic R as in equation (3.25.

$$R = \frac{(Q_o - \overline{Q_o})(Q_s - \overline{Q_s})}{\left[\frac{1}{n}\sum_{i=1}^{n}(Q_o - \overline{Q_o})^2\right]^{\frac{1}{2}}\left[\frac{1}{n}\sum_{o}^{n}(Q_s - \overline{Q_s})^2\right]^{\frac{1}{2}}} \qquad 3.25$$

where n = size of data points for the observed and predicted discharge.

Q_o = observed discharge, Q_s = predicted discharge

$\bar{Q}_o$ = mean observed discharge, $\bar{Q}_s$ = means predicted discharge

In this regard, specifically for extreme value, two correlation indices R_{max} (%) and R_{min} (%) were used where

$$R_{max}\,(\%) = \frac{Qs}{\max\,(Qo)} \times 100 \tag{3.18}$$

and

$$R_{min}\,(\%) = \frac{Qs}{\mathrm{Min}\,(Qo)} \times 100 \tag{3.19}$$

It suffices to note that max (Q_o) in the maximum of the observed discharge or flow and Q_s in the predicted flow corresponding to such maximum. Similarly, min (Q_o) is the minimum of the observed flow while Q_s is the predicted minimum corresponding to that minimum. Here, for this scheme, as used in this study, R_{max} = 100% means, that the observed peak in perfectly reproduced by model! R_{max} > 100% indicates that the model underestimated, the peak value while R_{max} > 100% connotes over estimation.

CHAPTER FOUR

4.0 RESULTS AND DISCUSSION

4.1 General Characteristic (FDC)

Table 4.1 below is the summary of the observed discharges of exceedance percentage from fig 4.1 - fig 4.5 as discussed accordingly,

Table: 4.1 summary of discharges of exceedance percentage ($Q_{p\%}$)

$Q_{P(\%)}$/m^3/s	Kaduna	Shiroro	Kachia	Izom	Baro
1	790	1450	710	1100	950
5	680	1080	560	940	740
10	580	950	460	720	610
20	410	575	330	480	420
30	260	300	170	300	280
40	160	150	90	190	180
50	90	90	50	100	100
60	60	60	30	60	43
70	40	50	18	50	29
80	30	48	10	25	20
90	20	25	7	10	7
95	18	20	5	8	5
99	10	5	2	5	

Flow Duration Curve (FDC) as observed in Figure 4.1 showed a gentle slope as depicted by the fall from the upper region of the graph dam to the lower end. This is an indication that there is enough water to sustain power generation and irrigation activities all through the various seasons. Between 0.00 and 20.00 percent time, the flow of water along River Niger at Baro station showed that the mean monthly flow was observed to be high while between 20.00 and 60.00 percent time, the mean monthly flow was observed to be a transition belt between the high flow rate and low flow rates.

Similarly, as shown in figure 4.2, though the slope of the somewhat gentle, towards the tail, the fall became sudden. Within the hydrological context, the physical, application of this is likelihood of prolong period of the dry season; this is not without its attendant shortcomings. Considering this, it suffuses to note that there may be low storage potential thus reducing its probable capacity to sustain both power general and irrigation. This scenario might be attributable to long- term change in flow require resulting activities from deleterious effects of anthropogenic and hydro- climatic variability. Fig 4.3 shows the FDC for Kaduna station. The up tail of the Flow duration curves for measuring exceedance percentages at Kaduna was observed to have a high flow rate between 0.00 and 40.00 percent time. This is a strong indication that it can sustain the generation of electricity for the people within the community. A transition period of between 40.00 and 85.00 percent time was observed which characterises the mid-range flow while the dry condition was observed to be between 85.00 and 100.00 percent time.

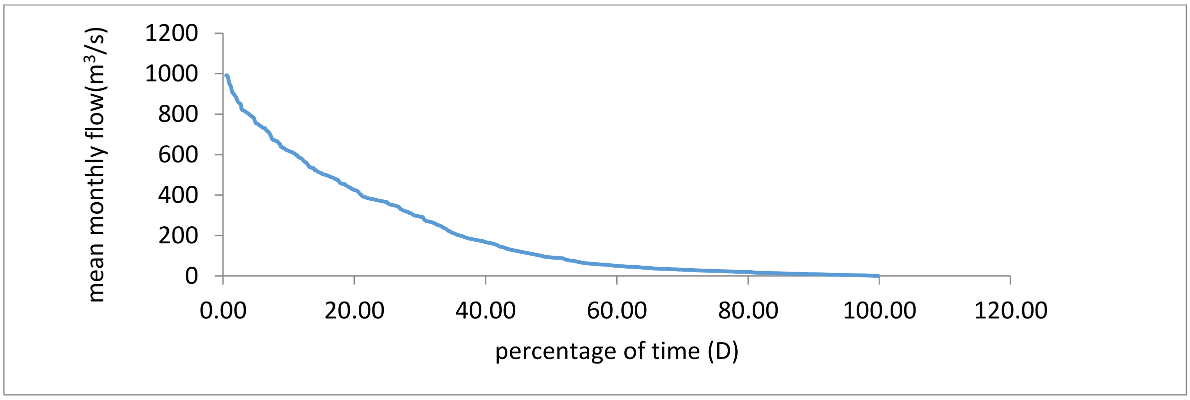


Fig 4.1 Flow duration curves for measuring exceedance percentages at Baro

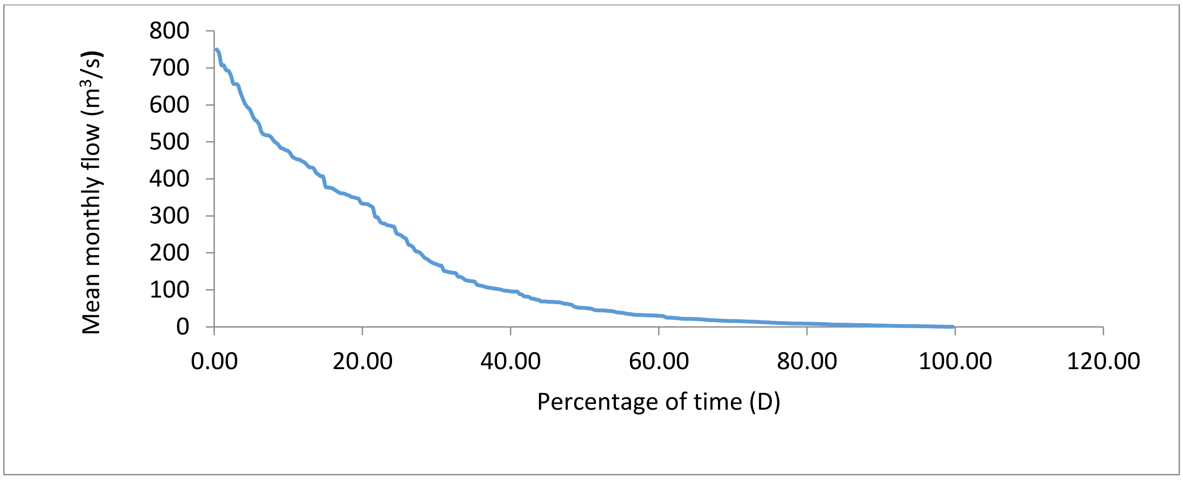


Fig 4.2 Flow duration curves for measuring exceedance percentages at Kachia

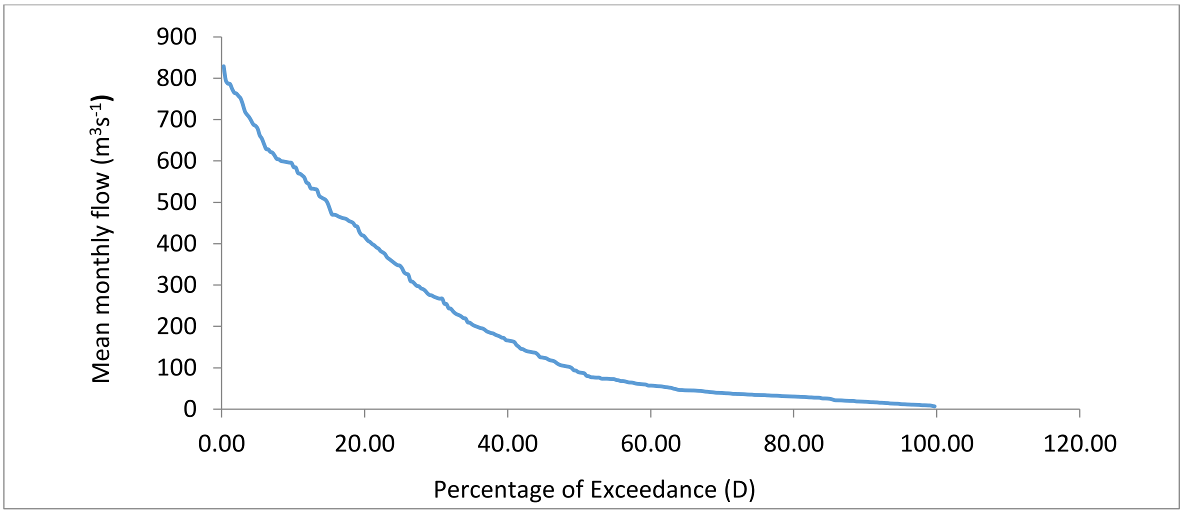


Fig 4.3 Flow duration curve showing exceedance percentages at Kaduna Station

Just like in the proceeding station, Figure 4.4 shows the FDC curve for Izom river flow station. It was observed that a high period of flow occurred between 0.00 and 20.00 percent time, between 20.00 and 40.00 percent time it was observed that the moist condition started occurring which also is known as the transition period from the high flow rates to the dry condition which started occurring from 60.00 to 100.00 percent time. This is in accordance with the works of Holmes, M. G. R., Young, A. R., Gustard, A., and Grew, R. (2002).

However, the slope depicted a sharp fall over a short period of time and becoming stable. This connotes variability in the flow regime. It was observed that the high flow period for this station ranged between 0.00 to 10.00 percent times which was observed to be of the shortest duration when compared with other stations. The moist condition for the study area ranged between 10.00 and 20.00 percent time which is a strong indication that the river will support irrigation and hydropower activities, at 1.00 percent time, the discharge at Shiroro station was observed to be the highest, closely followed by that of Izom. This pattern was observed all through to 99.00 percent time. Though towards the 70th percent time, all the five stations were observed to have low flow rates.

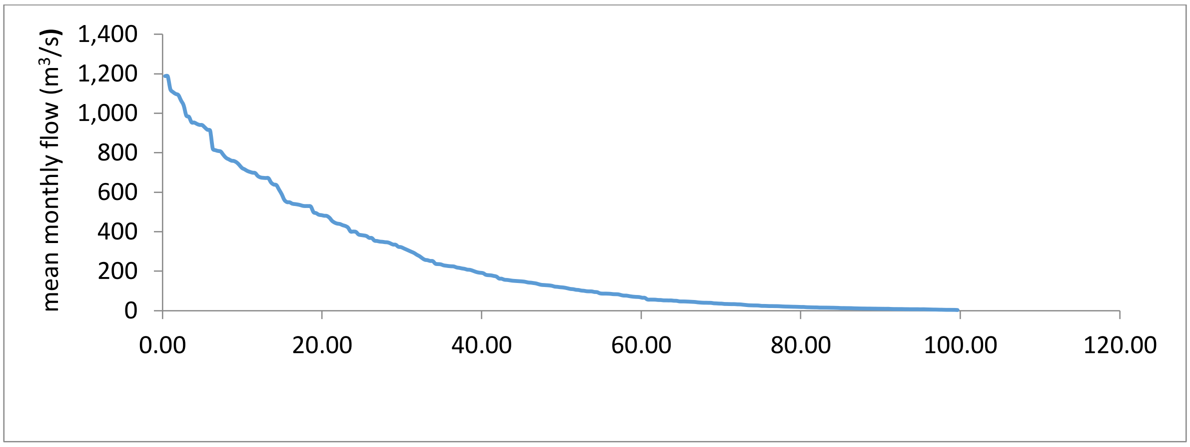


Fig 4.4 Flow duration showing percentages of exceedance for Izom Station

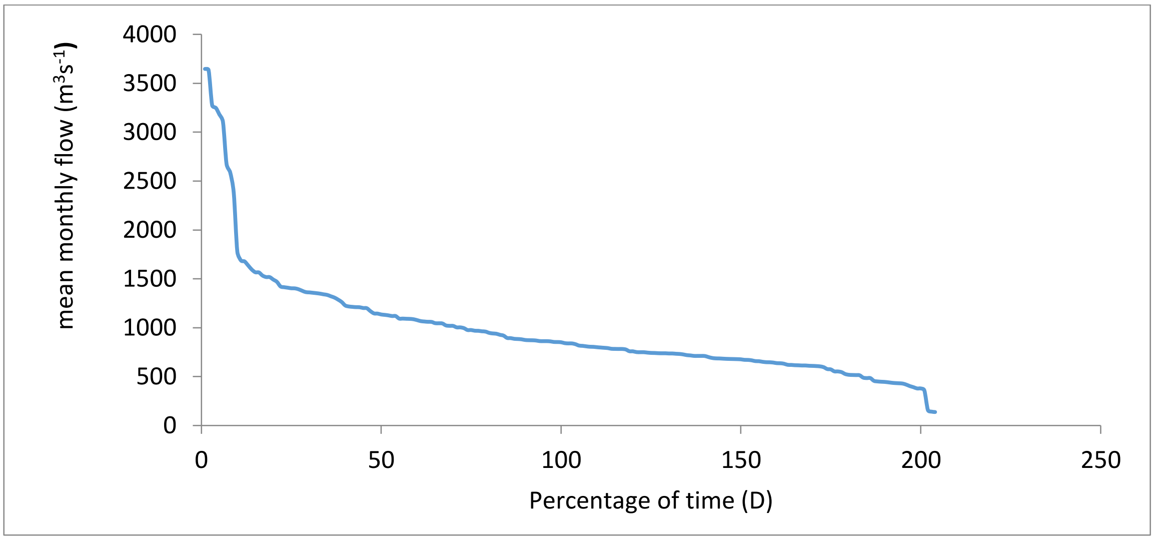


Fig 4.5 Flow duration curves for measuring exceedance percentages at Zungeru

Based on the data quality test ((Appendix B), that there was a seeming consistency among the flow data for the stations considered. Considering that the integrity of the data was robust, it connotes in a simple term that FDCs developed using these stream flow data can be transferred to the other ungauged station, otherwise known as regionalisation. Results of the correlation deficient used as a statistic indicator in the fitting of distribution function in as presented in Table 4.2 below.

Table: 4.2: Regression Coefficient (R^2) of the three mathematical models for the regular FDC and Dimensionless FDC

S/N	Location	Logarithmic Eq.(3.123),(3.4)	Power Eq.(3.2),(3.5)	Exponential Eq.(3.3),(3.6)
1	Kaduna	0.94	0.75	0.99
2	Shiroro	0.96	0.76	0.96
3	Kachia	0.96	0.74	0.99
4	Izom	0.96	0.71	0.98
5	Baro	0.97	0.70	0.98
	Average	0.96	0.73	0.98

From Table 4.2, in term of correlation coefficient (R^2), each mathematical model for the flow duration curves for the two parameters: Q and Q/Q, gives the same values of R^2. The average R^2 for all stations of the logarithmic, power and exponential models were 0.96, 0.73, and 0.98, respectively as shown in Table 4.2. The accuracy of the exponential model is as high as that of the logarithmic model. Though the power model had the least value of R^2, the low value of R^2 could be attributable to the inability of the power function to capture the flow variability. No model gives the best fit for all stations; however, the logarithmic and the exponential models seemed to fit very well to the FDC. Therefore, both the logarithmic and exponential models are considered in the regionalization of FDC models in the next section as can be clearly seen from Figures 4.6 to 4.8 for the dimensioned flow duration curve as well as Figures 4.9 to 4.11.

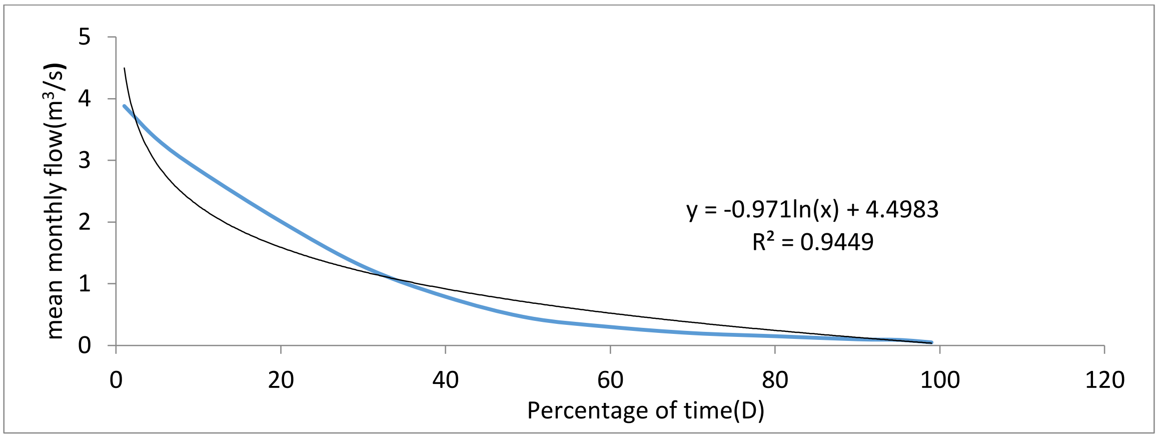


Figure 4.6: Dimensioned Flow duration curves and fitted lines from logarithmic model

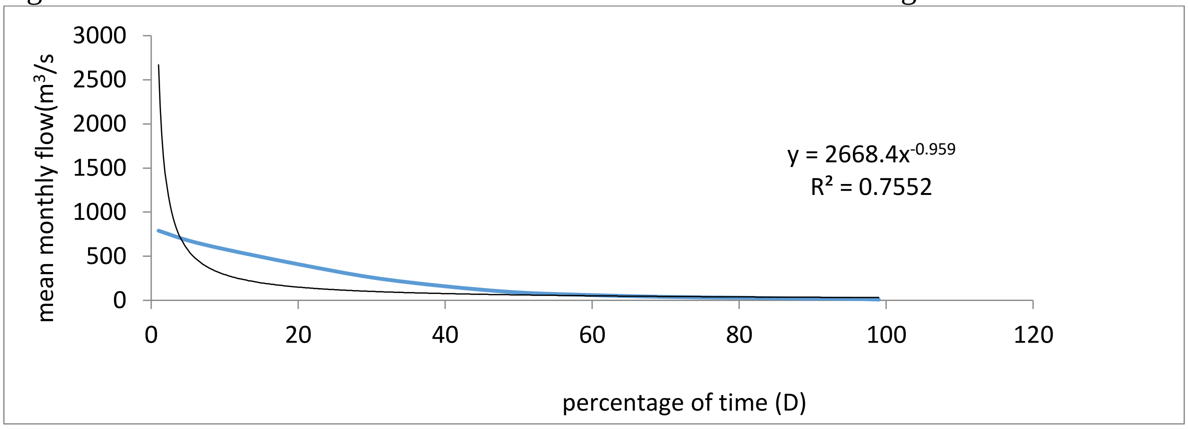


Figure 4.7: Dimensioned Flow duration curves and the fitted lines from power model

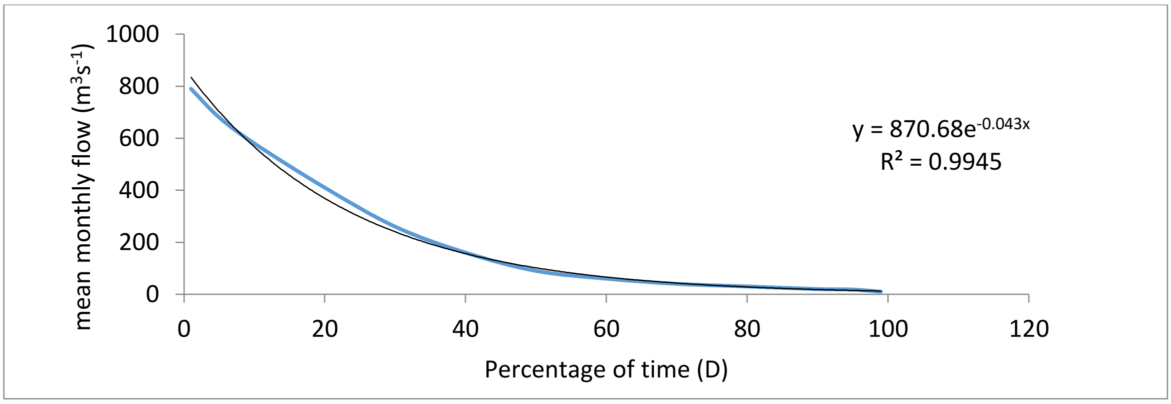


Figure 4.8: Dimensioned Flow duration curves and the fitted lines from exponential model

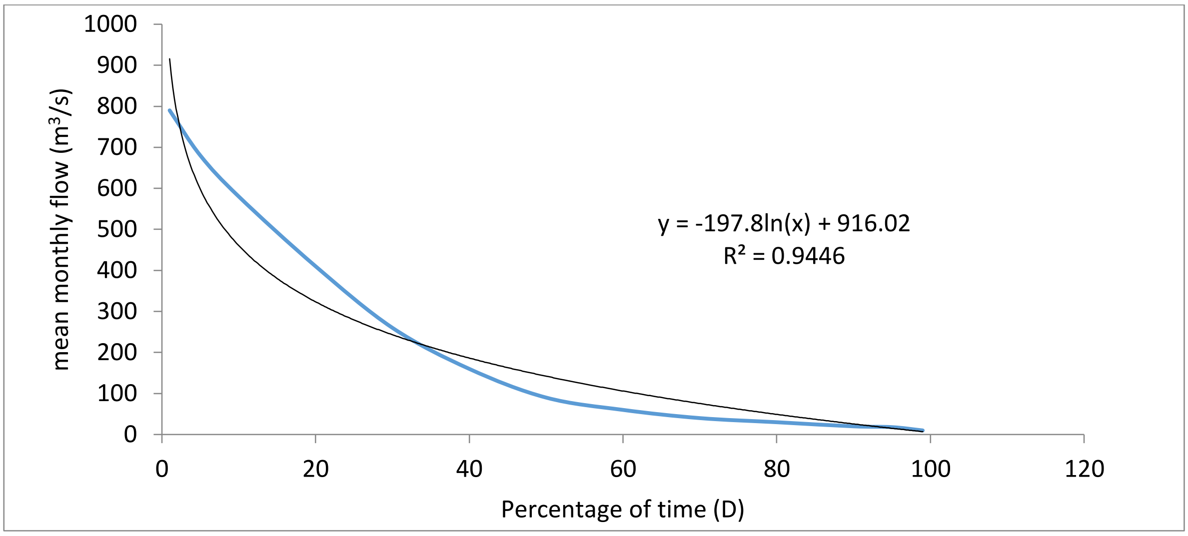


Figure 4.9: Dimensionless Flow duration curves and the fitted lines from logarithm model

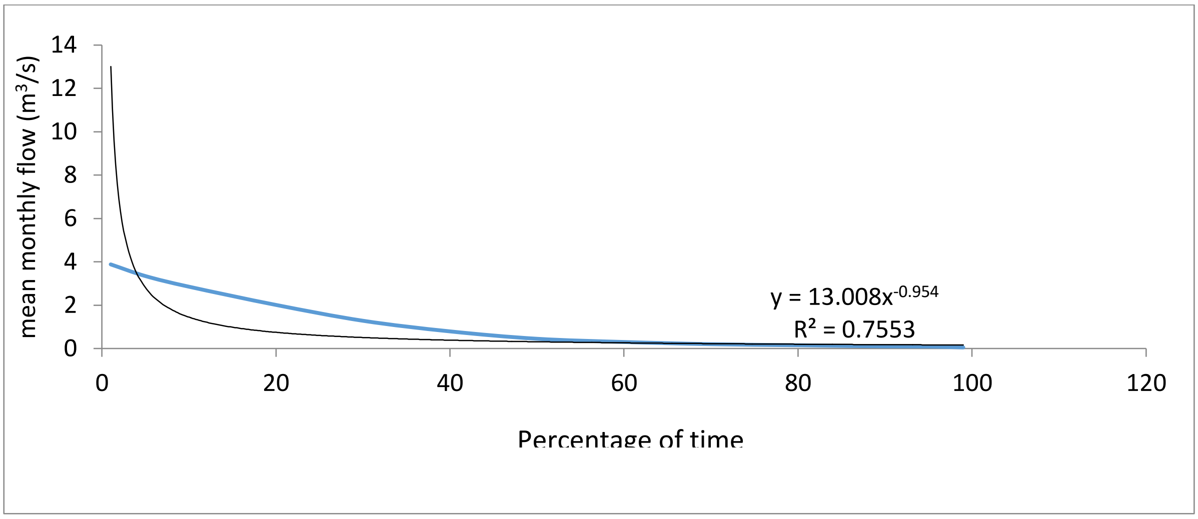


Figure 4.10: Dimensionless Flow duration curves and the fitted lines from power model

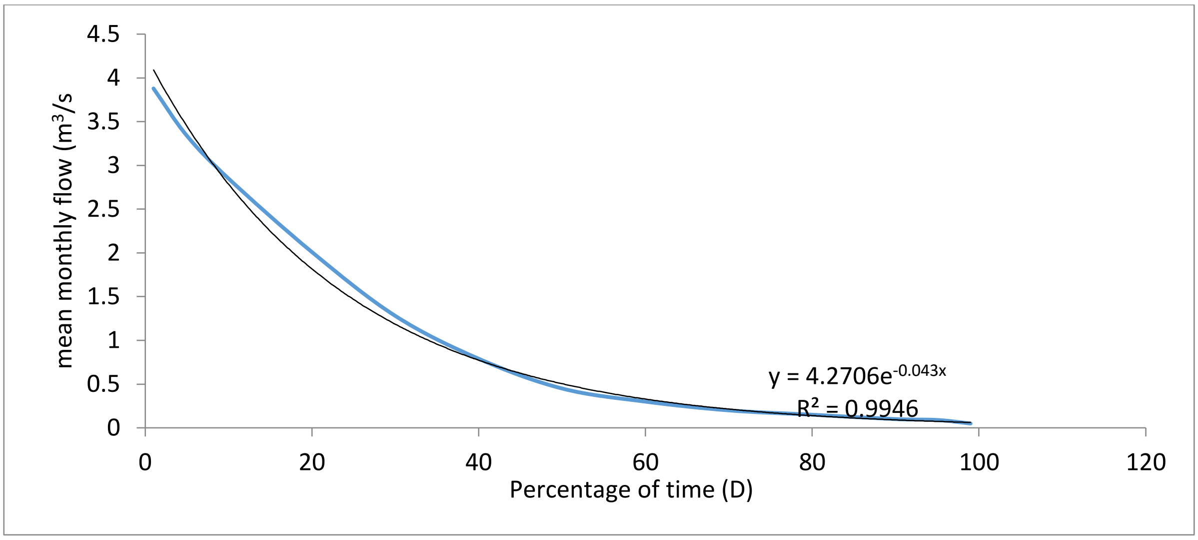


Figure 4.11: Dimensionless Flow duration curves and the fitted lines from exponential model

4.2 Regionalised FDCs

Table 4.3 below shows the coefficients a_1, a_2 and d_1, d_2 from the logarithmic models in Equations (3.1) and (3.4) and the coefficients c_1, c_2 and f_1, f_2 from the exponential models in Equations (3.3) and (3.6), respectively, for each of the study locations. The coefficients were regressed against the drainage area; the determination of the regionalised parameters is as shown in Figures 4.12 – 4.14. These parameters are then employed in the regionalisation framework

Table 4.3: Determined coefficient values for logarithmic and exponential models

		Eq.(3.1)		Eq.(3.4)		Eq.(3.3)		Eq.(3.6)	
S/N	Location	a_1	a_2	d_1	d_2	c_1	c_2	f_1	f_2

1	Kaduna	916.02	-197.8	4.4983	-0.971	870.68	-0.043	4.2706	-0.043
2	Shiroro	1558.2	-349	5.2739	-1.18	1381.6	-0.049	4.5955	-0.048
3	Kachia	783.89	-175	5.2132	-1.163	835.42	-0.056	5.273	-0.054
4	Izom	123.9	-273.8	4.983	-1.10	1356.8	-0.053	5.3873	-0.052
5	Baro	1047	-230.9	4.9108	-1.082	1254.4	-0.056	5.549	-0.054

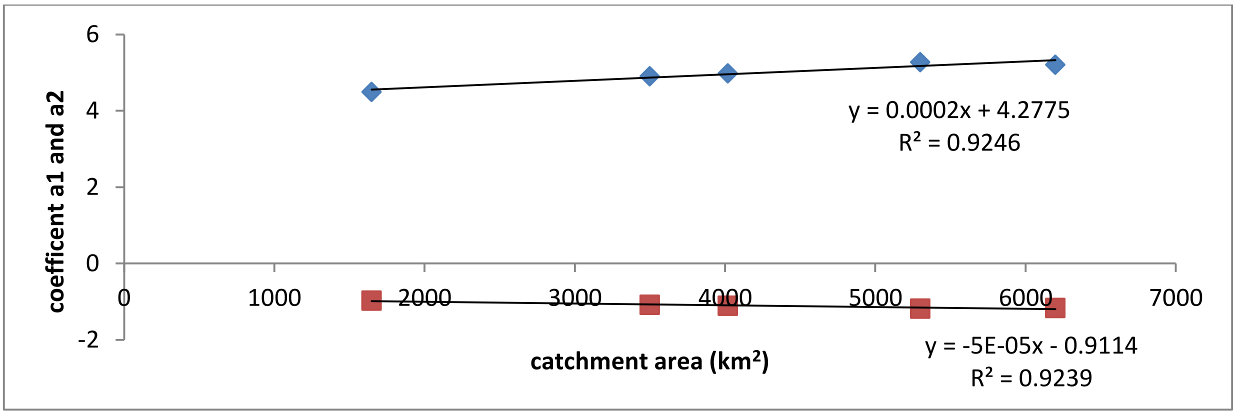


Figure 4.12: Relationship between coefficients a_1, a_2 and drainage area at sub-basin level.

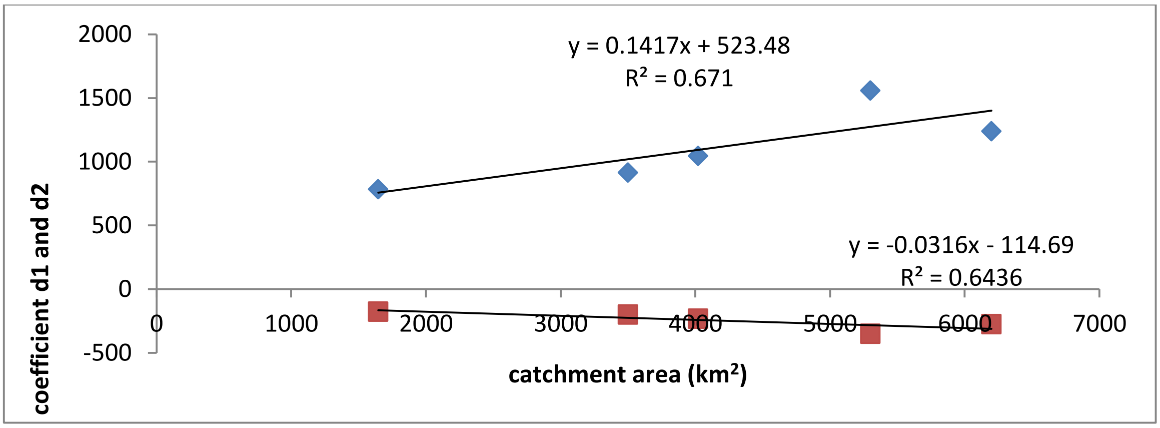


Figure 4.13: Relationship between coefficients of d_1, d_2 and drainage area at sub-basin level.

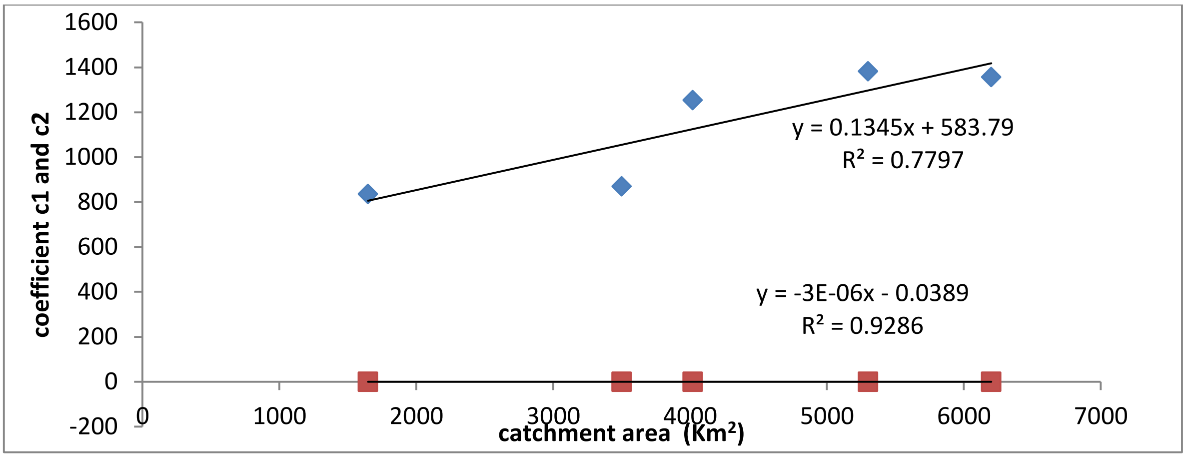


Figure 4.14: Relationship between coefficients c_1, c_2 and drainage area

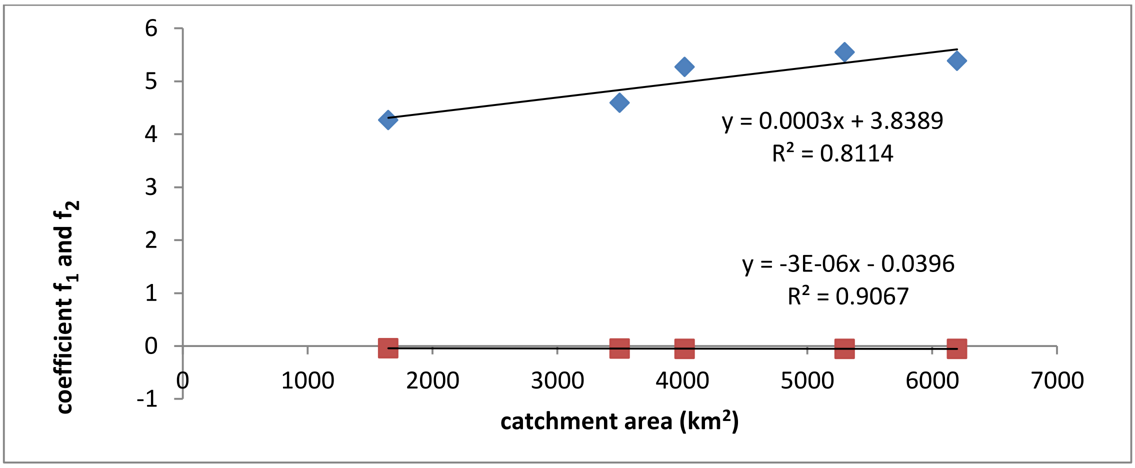


Figure 4.15 Relationship between coefficients of f1, f_2 and drainage area

Table 4.4: Regionalised parameter values

Coefficients	Basins			Model
$a_1 = j_1 + j_2A$	j_1	j_2	R^2	Dimensioned Logarithmic
	523.48	0.1417	0.671	
$a_2 = j_3 + j_4 A$	j_3	J_4	R^2	
	-114.69	-0.0316	0.6436	
$d_1 = k + k_2A$	K_1	K_2	R^2	

	4.2775	0.0002	0.9246	Dimensionless Logarithmic
$d_2 = k_3 + k_4A$	K_3	K_4	R^2	
	-0.9114	-0.00005	0.9239	
$C_1 = l_1 + l_2A$	L_1	L_2	R^2	Dimension exponential
	583.79	0.1345	0.7797	
$C_2 = l_3 + l_4A$	L_3	L_4	R^2	
	-0.0389	-0.000003	0.9286	
$F_1 = m_1 + m_2A$	M_1	M_2	R^2	Dimensionless exponential
	3.8389	0.0003	0.8114	
$F_2 = m_3 + m_4A$	M_3	M_4	R^2	
	-0.0390	-0.000003	0.9067	

The calculated basin values were inserted into equations (3.7) and (3.8) for the dimensioned Logarithmic and exponential models and (3.12) to (3.13) for the dimensionless Logarithmic and exponential models in order to compute the discharges (Q) corresponding to percent of time (D) at intervals increasing 1 % each time up to 100 %, for each station these were computed and compared between the results from the Logarithmic and exponential models to find the best fitted model. The correlation in this regard is as clearly shown by fig: 4.16. Appendix (A) gives the details of the simulated stream flow values. Table 4.6 show the error pattern of the predicted.

Table 4.5a: Coefficients *a*, *b* and *Q* for each station

Station	Drainage Area (km^2)	**a**	**b**	$\bar{Q}=aA^b$ (m^3/sec**)**
Kaduna	1647	4.1849	0.4795	145.91
Shiroro	3,500			209.44
Kachia	4020			223.83
Izom	6200			275.51
Baro	5300			255.55

For the dimensionless logarithmic and exponential models, using equation (3.12) and (3.13), the computed discharges are also presented. Tables: 4.7 and 4.8**:** details the E_R relative errors for the different models during the calibration and validation (verification) phases.

Table 4.7: Mean square relative errors (E_R) in the calibration phase for logarithmic and exponential models

E_R of Model Calibration				
Station	**FDC Model**		**Dimensionless Model**	
	Logarithmic, Eq(3.7)	Exponential, Eq(3.8)	Logarithmic, Eq(3.12)	Exponential Eq(3.13)
Kaduna	25.43	8.81	31.45	27.74
Shiroro	36.75	17.95	35.15	31.58
Kachia	20.88	49.49	49.20	69.19
Izom	56.34	16.88	26.70	27.63
Baro	24.69	27.45	32.64	35.52
Average	32.81	24.11	35.02	38.33

Table: 4.8: Comparison of Root Mean Square Relative errors (*ER*) during verification for the logarithmic and exponential models

ER of Model Calibration

station	FDC Model		Dimensionless Model	
	Logarithmic, Eq(3.7)	Exponential, Eq(3.8)	Logarithmic, Eq(3.12)	Exponential Eq(3.13)
Zungeru	77.97	78.11	79.02	76.82
Agaie	32.22	18.99	47.65	48.18
Average	55.09	48.55	63.33	62.50

The average errors for all cases during calibration were 32.81%, 24.11 %, 35.02 %, 38.33 %, and 55.09 %, 48.55 %, 63.33 %, 62.50 % for validation respectively. It can be seen that the exponential model using FDC parameter gives reasonably well estimations of the FDC for the stations considered. However, in view of the short length of data used for the study, the model that gives the smallest root mean square relative error (E_R) value can be used to predict flow at the ungauged site within sub-basin. The contrasts between these values (E_R and R^2) should not be related since E_R is used to measure the prediction error of the proposed models whereas the values of R^2 indicate how strong a linear correlation existed between the coefficients(a_1 , a_2 , e_1 , e_2 , f_1 , f_2 , j_1 , j_2) and the drainage area (A).The physical implication of the results here is that a representative catchment characteristics may not be sufficient to reflect the hydrologic variation of a particular catchment.

Table 4.9: Model forecast performance of extreme flow value for calibration stations

S/No	Station	Logarithmic		Exponential	
		R_{max} (%)	R_{min}(%)	R_{max}(%)	R_{min}(%)
1.	Kaduna	95.81	65.8	95.57	52.65

2.	Shiroro	70.31	56.7	76.41	79.30
3.	Kachia	76.98	13.53	72.26	81.00
4.	Izom	63.73	86.6	60.84	95.60
5.	Baro	67.08	79.67	64.61	11.85
	Average	74.0	60.0	73.0	64.00

Table 4.10: Validation Stations

S/No	Station	Logarithm		Exponential	
		R_{max}%	R_{min}%	R_{max}%	R_{min}%
1.	Zungeru	23.7	5.02	24.12	7.96
2.	Agaie	73.98	6.95	76.83	11.47
	Average				

(A) STATISTICAL APPROACH

Table 4.11 below shows the discharge of the various exceedance percentages for the five study stations; the tables show a seemingly good agreement indicating the effectiveness of using catchment area. Thus, the flow duration curve of the ungauged basin can be calculated using equation 3.23 and the coefficient value from the same Table 4.11

Table: 4.11: The regression coefficient and coefficient of efficiency in the study region

P%	a_1	a_2	CE

1	17.622	0.4865	0.858
5	15.141	0.4785	0.857
10	11.81	0.4857	0.774
20	18.931	0.3808	0.705
30	6.3087	0.45	0.583
40	1.3308	0.5731	0.883
50	0.9536	0.543	0.984
60	0.4644	0.5654	0.993
70	0.0396	0.8236	0.995
80	0.0028	1.0973	0.994
90	0.0045	0.9574	0.997
95	0.0012	1.0919	0.998
99	0.0006	1.078	0.999

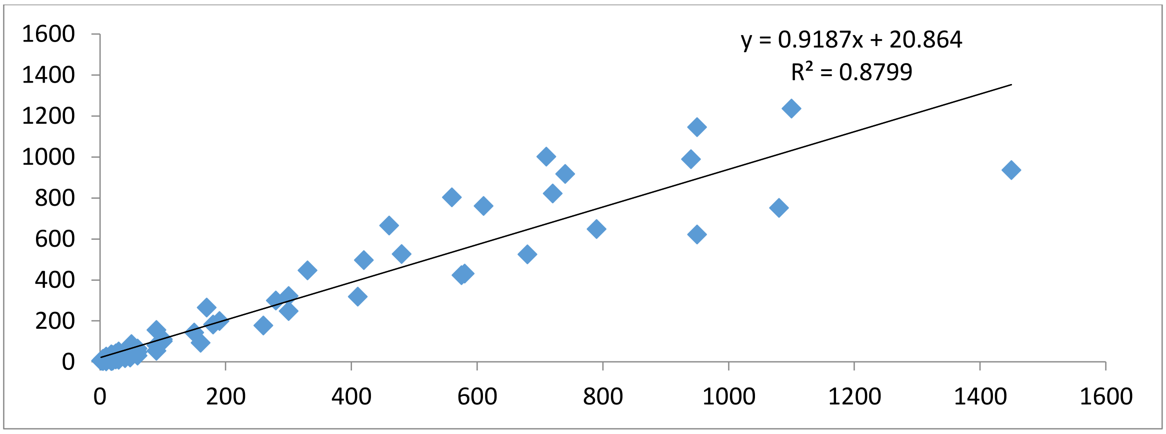


Figure: 4.16: Scatter plot of the observed discharge against the simulated discharge

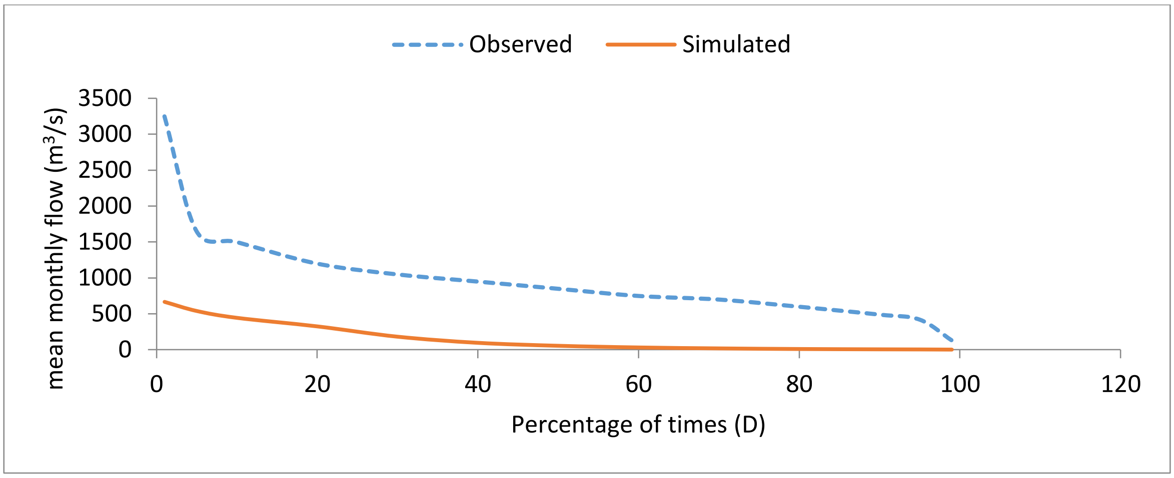


Figure 4.17a: The simulated results of flow duration curve for the Zungeru verification station

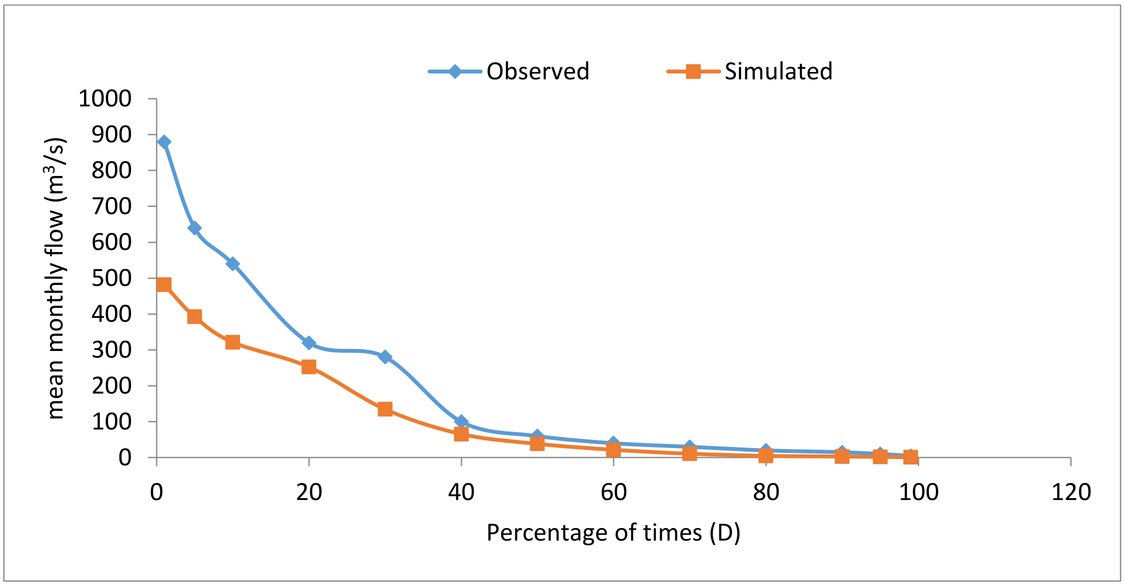


Figure 4.17b: The simulated results of flow duration curve for the Agaie verification station

The regional flow duration curve was then validated at the two stations as shown in figures 4.17a and 4.17b respectively in which the synthetic flow duration curves were not very close to the actual flow duration curves. Also from figure 4.17, it is evident that the regionalised models could not reproduce low flow pattern in the both the validation stations. However, the degree of correlation for high flow prediction is high. The only viable explanation for this is that models generally replicate a system's characteristics well only if fed sufficient information. The case here is so different; the data base mobilised for the study is barely adequate. Despite this short coming, the overall performance of the regionalised models is adequate.

(B) Reliability and Stream Flow Variability

Table 4.12: Stream Flow Variability (Low Flow Index)

S/No	Station	Q_{20}/Q_{50}	Q_{50}/Q_{90}
1.	Kaduna	4.56	4.5
2.	Shiroro	6.39	3.6
3.	Kachia	6.6	7.14
4.	Izom	4.8	10
5.	Baro	4.2	14.29

Table 4.13: Stream Flow Variability at Validation Stations.

Station	**Q_{20}/Q_{50}**	**Q_{50}/Q_{90}**
Zungeru	1.41	1.73
Agaie	5.33	4

CHAPTER FIVE

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

1. This study attempted to critically look at the question of stream flow generation at ungauged station using prior information of other gauging station based on the assumption of hydro-meteorological similarity.
2. Based on the findings of the study it is imperative to note that the regionalisation of flow duration curves is an effective approach for stream flow generation and extension, especially in the face of data scarcity.
3. In addition, it is clearly evident that the use of catchment characteristics as input parameters, to large extent for model development whether conceptional or statistical model in essential. Results obtained by incorporating the drainage area in the overall regionalised model indicate that it is representative enough but however, considering the length of data used and the attendant problem of flow variability it suffices to note that an ensemble of catchment characteristics maybe needed.

It is also important to take into consideration the ability of the models to reproduce the flow signature; in this case the prediction of extreme values is critical. Both the logarithmic and exponential function employed in the FDCs portrayed different characteristics in the calibration and validation stages. The exponentially regionalised models overwhelmingly perform better than the logarithmic where as in the validation phase, it was abysmal for both models this could be attributable to the short length of data used.

4. Considering the results obtained in the overall objective generalisations may not be possible though however, the parameters obtained for the models in the regionalisation procedure were largely optimised. Thus, effective conclusion on the suitability of using the drainage area by a representative parameter depicting the overall behaviour of a hydrologic response unit should be done with cautious optimism; especially taking into cognisance the implications geologic characteristics of drainage basin in affecting low flows.

5. Generally, in all contexts, since there in a dearth of long-term gauging station records, the regional analysis helped to improve the reliability of the flow duration curves.

5 .2 Recommendations

Based on the findings of the study, the following are preferred for consideration

1. Considering the fact that monthly stream flows exhibit short memory, it is imperative to develop flow duration curves using a smaller temporal resolution.
2. In view of the fact that variability of stream flow is the result of variability in rainfall pattern as modified by basin characteristics it pertinent that varying combination of catchment characteristics like slope, drainage density and land use factor e.t.c should be employed
3. Since the slope of the flow duration curve is a quantitative measure of variability several indexes of slope should be considered in the development of flow duration curves as well as in the regionalisation frame work.
4. When the extreme end of the flow duration curve is important, base-flow measurement should be made at the unmeasured site(s) and correlated with concurrent discharge at a stream gauging stations.

5. The reliability of the record for predicting the behaviour of the stream/river depends upon the accuracy and consistency of the record and upon how well the period of record represent the long-term flow of the stream/river: thus in addition to consistency test for data quality, it is important that heterogeneity and Seasonality analyses the done to enhance data integrity.

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APPENDIX A: Summary of Simulated Discharges in (m^3/s)

Q(P%)/$M^3/_S$	Kaduna	Shiroro	Kachia	Izom	Baro
1	648.58	935.90	1001.14	1236.06	1,145.25
5	524.01	751.61	803.11	988.13	916.69
10	431.12	621.73	665.00	820.76	760.56
20	317.75	423.40	446.34	526.40	495.88
30	176.79	248.18	264.14	321.01	299.13
40	92.81	142.96	154.77	198.40	181.34
50	53.21	80.13	86.39	109.30	100.38
60	30.59	46.85	50.67	64.73	59.24
70	17.66	32.85	36.82	52.61	46.24
80	9.48	21.68	25.24	40.60	34.18
90	5.41	11.13	12.70	19.23	16.55
95	3.90	8.89	10.34	16.60	13.99
99	1.76	3.97	4.61	7.35	6.21

SUMMARY OF COMPUTED DISCHARGES IN (m^3/s) from Equation 15

$$Q = (J_1+J_2A) + (J_2+J_4A)\ \ln(D)$$

% (D)	KADUNA	SHIRORO	KACHIA	IZOM	BARO
1	756.86	1019.43	1093.11	1402.02	1274.49
10	372.94	500.68	536.53	686.81	624.77
20	257.37	344.52	368.98	471.52	429.18
30	189.76	253.17	270.97	345.57	314.77
40	141.79	188.36	201.43	256.22	233.60
50	104.59	138.09	147.49	186.91	170.63
60	74.19	97.02	103.42	130.28	119.19
70	48.49	62.29	66.16	82.39	75.69
80	26.22	32.20	33.88	40.92	38.01
90	6.58	5.67	5.41	4.33	4.78
95	-2.43	-6.51	-7.66	-12.46	-10.48
99	-9.31	-15.80	-17.63	-25.27	-22.11

Appendix A

SUMMARY OF COMPUTED DISCHARGES IN (M^3) FROM EQUATION 16

$$Q = (L_1+L_2A) \exp ((L_3+L_4A (D))$$

% D	KADUNA	SHIRORO	KACHIA	IZOM	BARO
1	770.77	1107.94	1068.61	1338.47	1227.50
5	646.79	1350.00	871.55	1063.46	985.88
10	519.48	643.46	675.51	797.74	749.59
20	335.09	392.63	405.81	448.89	433.34
30	216.16	239.57	243.78	252.59	250.52
40	139.43	146.18	146.45	142.14	144.75
50	89.94	89.20	87.98	79.98	83.72
60	58.02	54.43	52.85	45.01	48.40
70	37.43	33.21	31.75	25.32	27.98
80	24.14	20.26	19.07	14.25	16.18
90	15.57	12.36	11.46	8.02	9.35
95	12.51	9.66	8.88	6.02	7.11
99	10.53	9.93	7.24	4.78	71.06

SUMMARY OF COMPUTED DISCHARGES IN (m^3) FROM EQUATION 19

$$Q= (K_1+K_2A) + (K_3+K_4A) \ln(D) \times aA^b$$

% D	KADUNA	SHIRORO	KACHIA	IZOM	BARO
1	672.19	1042.49	1137.39	1520.13	1364.00
5	438.83	676.28	736.66	978.54	880.15
10	338.33	518.57	564.07	745.29	671.77
20	237.83	360.85	391.49	512.04	463.39
30	179.04	268.59	290.53	375.60	341.50
40	137.32	203.14	218.90	278.79	255.01
50	104.97	152.36	163.34	203.70	187.93
60	78.53	110.88	117.95	142.35	133.12
70	56.18	75.80	79.57	90.48	86.78
80	36.82	45.42	46.32	45.54	46.63
90	19.74	18.62	16.99	5.90	11.22
95	11.90	6.32	3.53	12.29	-5.03
99	5.92	3.07	6.74	26.17	-17.43

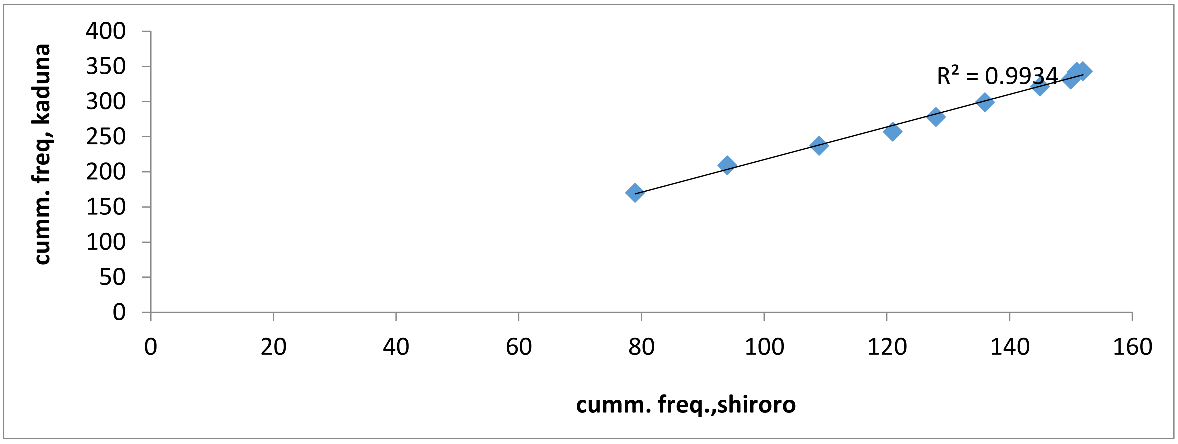
SUMMARY OF COMPUTED DISCHARGES IN (m^3) FROM EQUATION 20

$$Q = (M_1 + M_2A) \exp((M_3 + (M_4A)(D)) \times aA^b$$

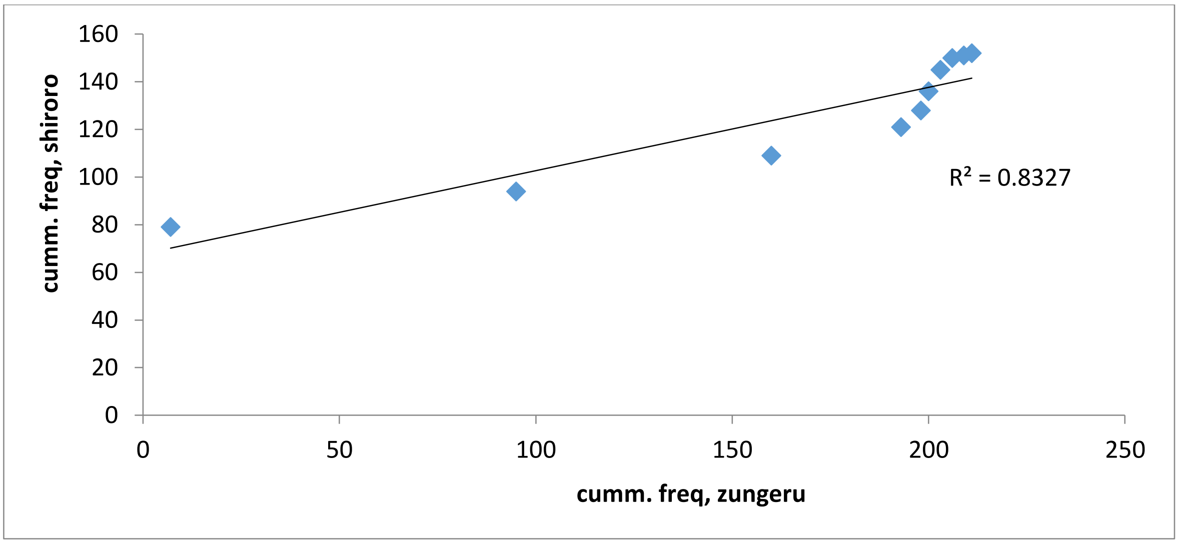
% (D)	KADUNA	SHIRORO	KACHIA	IZOM	BARO
1	604.69	973.90	1072.35	1481.33	1312.45
5	506.00	797.03	872.15	1173.68	1051.17
10	404.98	620.42	673.62	877.34	796.44
20	259.42	375.93	401.84	490.24	457.22
30	166.17	227.79	239.72	273.94	262.47
40	106.44	138.02	143.00	153.07	150.68
50	68.18	83.63	85.31	85.53	86.50
60	43.68	50.67	50.89	47.79	49.66
70	27.98	30.70	30.36	26.71	28.50
80	17.92	18.60	18.11	14.92	16.37
90	11.48	11.27	10.80	8.34	9.39
95	9.19	8.77	8.34	6.23	7.12
99	7.69	7.18	6.79	4.94	5.70

APPENDIX B: Consistency Test

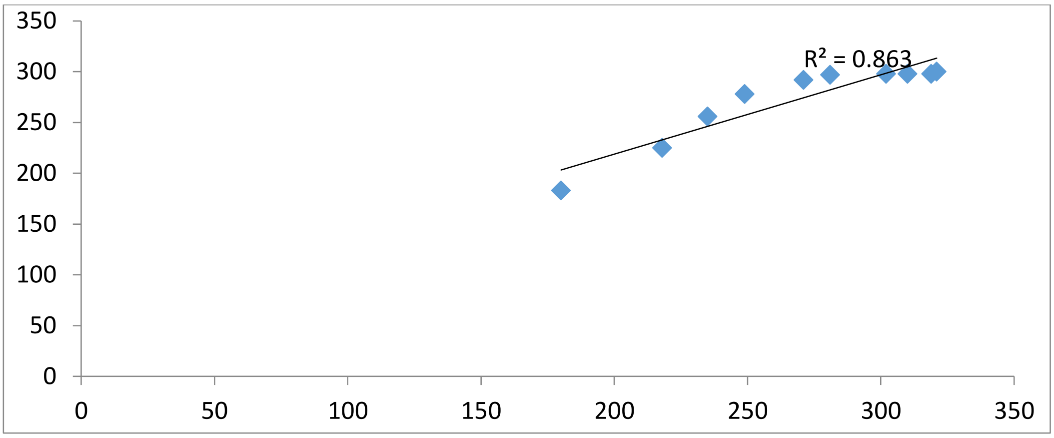
(i) Shiroro Station vs Kaduna Station



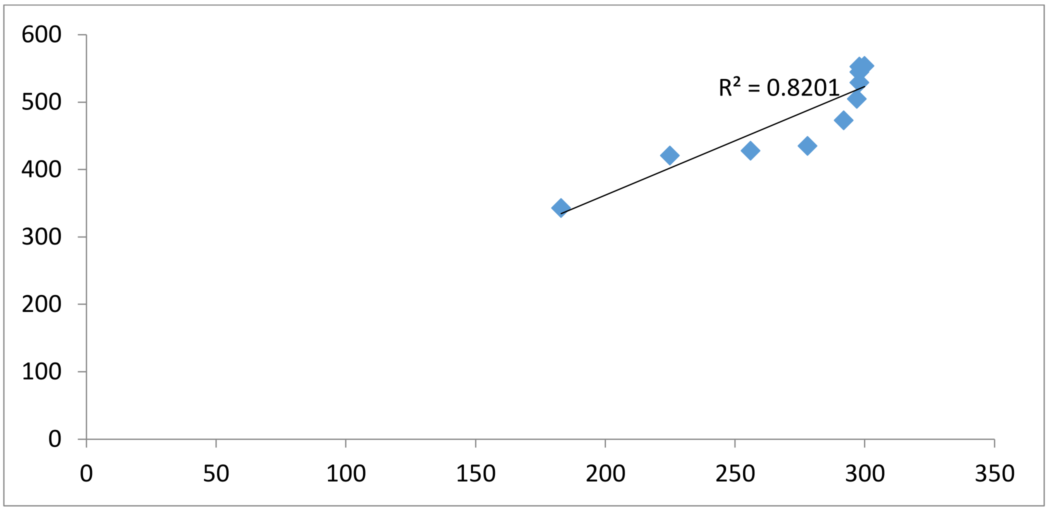
(ii) Consistency test between Zungeru and Shiroro station



(iii) Consistency test between Kachia and Izom station


R² = 0.863
350
300
250
200
150
100
50
0
0
50
100
150
200
250
300
350

(iv) (iv) Consistency test between Izom and Baro



APPENDIX C: Computational Analysis for both the regular FDCs and the regionalised models

Kaduna station

From equation (3.15)

$Q = (J_1 + J_2A) + (J_3 + J_4A) \ln D$

$= (523.48 + 0.1417(1647) + (-114.69 + (-0.0316 \times 1647) \ln (D)$

$Q = 756.8599 – 166.7352 \ln (D)$

If D = 1% then, Q = 756.86

5%	-	488.51
10%	-	372.94
20%	-	257.37
30%	-	189.76
40%	-	141.79
50%	-	104.59
60%	-	74.19
70%	-	48.49
80%	-	26.22
90%	-	6.58
95%	-	-2.43
99%	-	-9.31

Shiroro

$Q = (J_1 + J_2A) + (J_3 + J_4A) \text{ In } (D)$

$= 523.48 + (0.1417 \times 3500) + (-114.69 + (-0.0316 \times 3500) \text{ In } (D)$

$Q = 1019.43 – 225.29 \text{ In } (D)$

1%	-	1019
5%	-	656.48
10%	-	500.68
20%	-	344.52
30%	-	253.17
40%	-	188.36
50%	-	138.09
60%	-	97.02
70%	-	62.29
80%	-	32.20
90%	-	5.67
95%	-	-6.51
99%	-	-15.80

Kachia

$Q = (J_1 + J_2A) + (J_3 + J_4A) \text{ In } (D)$

$= (523.48 + 0.1417 \times 4020) + (-114.69 + (-0.0316 \times 4020) \text{ In } (D)$

$Q = 1093.114 – 241.722 \text{ In } (D)$

1%	-	1093.114
5%	-	704.08
10%	-	536.53
20%	-	368.98
30%	-	270.97
40%	-	201.43
50%	-	147.49
60%	-	103.42
70%	-	66.16
80%	-	33.88
90%	-	5.41
95%	-	-7.66
99%	-	-17.63

Izom

$Q = (J_1 + J_2A) + (J_3 + J_4A) \text{ In } (D)$

$= (523.48 + 0.1417 \text{ x } 6200) + (-114.69 + (-0.0316 \text{ x } 6200) \text{ In } (D)$

$Q = 1402.02 – 310.61 \text{ In } (D)$

1%	-	1402.02
5%	-	902.11
10%	-	686.81
20%	-	471.52
30%	-	345.57
40%	-	256.22
50%	-	186.91
60%	-	130.28
70%	-	82.39
80%	-	40.92
90%	-	4.33
95%	-	-12.46
99%	-	-25.27

Baro

$Q = (J_1 + J_2A) + (J_3 + J_4A)\ \ln(D)$

$= (523.48 + 0.1417 \times 5300) + (-114.69 + (-0.0316 \times 5300)\ \ln(D)$

$Q = 1274.49 - 282.17\ \ln(D)$

1%	-	1274.49
5%	-	820.35
10%	-	624.77
20%	-	429.18
30%	-	314.77
40%	-	233.60
50%	-	170.63
60%	-	119.19
70%	-	75.69
80%	-	38.01
90%	-	4.78
95%	-	-10.48
99%	-	-22.11

Kaduna

From equation (3.16)

$Q = (L_1 + L_2A) \exp [(L_3 + L_4A) (D)]$

$= (583.79 + 0.1345x1647 \exp [(-0.0389+ (-0.000003 \times 1647) (D)]$

$Q = 805.3115 \exp [(-0.043841)(D)]$

If D = 1% then, Q = 770.77

5%	-	646.79
10%	-	519.48
20%	-	335.09
30%	-	216.16
40%	-	139.43
50%	-	89.94
60%	-	58.02
70%	-	37.43
80%	-	24.14
90%	-	15.57
95%	-	12.51
99%	-	10.53

Shiroro

$Q = (L_1 + L_2A) \exp [(L_3 + L_4A) (D)]$

$= (583.79 + 0.1345 \times 3500) \exp [(-0.0389+ (-0.0000003.3500) (D)]$

$Q = 1054.54 \exp (-0.0494 \times D)$

1%	-	1107.94
5%	-	1,350.00
10%	-	643.46
20%	-	392.63
30%	-	239.57
40%	-	146.18
50%	-	89.20
60%	-	54.43
70%	-	33.21
80%	-	20.26
90%	-	12.36
95%	-	9.66
99%	-	7.93

Kachia

$Q = (J_1 + J_2A) + (J_3 + J_4A)\ \text{In}\ (D)$

$= (523.48 + 0.1417 \times 4020) + (-114.69 + (-0.0316 \times 4020)\ \text{In}\ (D)$

$Q = 1093.114 – 241.722\ \text{In}\ (D)$

1%	-	1068.61
5%	-	871.55
10%	-	675.51
20%	-	405.81
30%	-	243.78
40%	-	146.45
50%	-	87.98
60%	-	52.85
70%	-	31.75
80%	-	19.07
90%	-	11.46
95%	-	8.88
99%	-	7.24

Izom

$Q = (L_1 + L_2A) \exp [(L_3 + L_4A) (D)]$

$= (583.79 + 0.1345 \times 6200) \exp [(-0.0389 + (-0.000003 \times 6200) (D)]$

$Q = 1417.69 \exp (-0.0575 \times D)$

1%	-	1338.47
5%	-	1063.46
10%	-	797.74
20%	-	448.89
30%	-	252.59
40%	-	142.14
50%	-	79.98
60%	-	45.01
70%	-	25.32
80%	-	14.25
90%	-	8.02
95%	-	6.02
99%	-	4.78

Baro

$Q = (L_1 + L_2A) \exp [(L_3 + L_4A) (D)]$

$= (583.79 + 0.1345 \times 5300) \exp [(-0.0389 + (-0.000003 \times 5300) (D)]$

$Q = 1296.64 \exp (-0.0548 \times D)$

1%	-	1227.50
5%	-	985.88
10%	-	749.59
20%	-	433.34
30%	-	250.52
40%	-	144.75
50%	-	83.72
60%	-	48.40
70%	-	27.98
80%	-	16.18
90%	-	9.35
95%	-	7.11
99%	-	-71.06

Kaduna

From equation (3.20)

$Q = (M_1 + M_2A) \exp [(M_3 + (M_4A) (D)] \times aA^b$

$= (3.8389+0.0003x1647\exp[(-0.0396 + (-0.000003 \times 1647) (D)] \times 145.91$

$Q = 4.333\exp (-0.044541xD) \times 145.91$

If D = 1% then, Q = 604.69

5%	-	506.00
10%	-	404.98
20%	-	259.42
30%	-	166.17
40%	-	106.44
50%	-	68.18
60%	-	43.68
70%	-	27.98
80%	-	17.92
90%	-	11.48
95%	-	9.19
99%	-	7.69

Shiroro

$Q = (M_1 + M_2A) \exp [(M_3 + (M_4A) (D)] \times aA^b$

$= (3.8389 + 0.0003 \times 3500)\exp[(-0.0396+(-0.0000003.3500)(D)]\times 209.44$

$Q = 4.8889 \exp (-0.0501 \times D) \times 209.44$

$= 1023.931216 \exp (-0.0501\times D)$

1%	-	973.90
5%	-	797.03
10%	-	620.42
20%	-	375.93
30%	-	227.79
40%	-	138.02
50%	-	83.63
60%	-	50.67
70%	-	30.70
80%	-	18.60
90%	-	11.27
95%	-	8.77
99%	-	7.18

Kachia

$Q = (M_1 + M_2A) \exp [(M_3 +(M_4A) (D)] \times aA^b$

$= (3.8389 + 0.0003 \times 4020)\exp[(-0.0396 + (-0.000003\times4020)(D)]\times223.83$

$Q = 5.0449\exp (-0.05166\times D) \times 223.83$

$Q = 1129.199967\exp(-0.05166\times D)$

1%	-	1072.35
5%	-	872.15
10%	-	673.62
20%	-	401.84
30%	-	239.72
40%	-	143.00
50%	-	85.31
60%	-	50.89
70%	-	30.36
80%	-	18.11
90%	-	10.80
95%	-	8.34
99%	-	6.79

Izom

$Q = (M_1 + M_2A) \exp [(M_3 + (M_4A) (D)] \times aA^b$

$= (3.8389 + 0.0003 \times 6200)\exp[(-0.0389+(-0.000003 \times 6200)(D)]\times 275.51$

$Q = 5.6989 \exp (-0.0582 \times D) \times 275.51$

$Q = 1570.103939 \exp (-0.0582 \times D)$

1%	-	1481.33
5%	-	1173.68
10%	-	877.34
20%	-	490.24
30%	-	273.94
40%	-	153.07
50%	-	85.33
60%	-	47.79
70%	-	26.71
80%	-	14.92
90%	-	8.34
95%	-	6.23
99%	-	4.94

Baro

$Q = (M_1 + M_2A) \exp [(M_3 + (M_4A)(D)] \times aA^b$

$= (3.8389 + 0.0003 \times 5300)\exp[(-0.0396+(-0.000003 \times 5300)(D)]xaA^b$

$Q = 5.4289 \exp (-0.0555 \times D) \times 255.55$

$Q = 1387.355395 \exp (-0.0555 \times D)$

1%	-	1312.45
5%	-	1051.17
10%	-	796.44
20%	-	457.22
30%	-	262.47
40%	-	150.68
50%	-	86.50
60%	-	49.66
70%	-	28.50
80%	-	16.37
90%	-	9.39
95%	-	7.12
99%	-	5.70

Kaduna

From Equation (3.19)

$Q = (K_1 + K_2A) \exp (K_3 + K_4A) \ln (D) \times aA^b$

$= (4.2775+0.0002x1647+ (-0.9114 + (-0.000005 \times 1647) \ln (D) \times 145.91$

$Q = 4.6069-0.99373 \ln (D) \times 145.91$

1%	-	672.19
5%	-	438.83
10%	-	338.33
20%	-	237.83
30%	-	179.04
40%	-	137.32
50%	-	104.97
60%	-	78.53
70%	-	56.18
80%	-	36.82
90%	-	19.74
95%	-	11.90
99%	-	5.92

Shiroro

$Q = (K_1 + K_2A) +(K_3 + K_4A)\ \text{In}\ (D) \times aA^b$

$= (4.2775 + 0.0002 \times 3500)+(-0.9114+(-0.000005.3500)\ \text{In}\ (D)\times 209.44$

$Q = 4.9775-1.0864\ \text{In}\ (D) \times 209.44$

1%	-	1042.49
5%	-	676.28
10%	-	518.57
20%	-	360.85
30%	-	268.59
40%	-	203.14
50%	-	152.36
60%	-	110.88
70%	-	75.80
80%	-	45.42
90%	-	18.62
95%	-	6.32
99%	-	3.07

Kachia

$Q = (K_1 + K_2A) + (K_3 + K_4A) \text{ In } (D) \times aA^b$

$= (4.2775 + 0.0002 \times 4020)+(-0.9114 + (-0.00005x4020)In(D)x223.83$

$Q = 5.815-1.1124 \text{ In } (D) \times 223.83$

1%	-	1137.39
5%	-	736.66
10%	-	564.07
20%	-	391.49
30%	-	290.53
40%	-	218.90
50%	-	163.34
60%	-	117.95
70%	-	79.57
80%	-	46.32
90%	-	16.99
95%	-	3.53
99%	-	-6.74

Izom

$Q = (K_1 + K_2A) + (K_3 + K_4A)In(D) \times aA^b$

$= (4.2775 + 0.0002 \times 6200)+(-0.00005 \times 6200)In(D) \times 275.51$

$Q = 5.5175-1.2214 \ In \ (D) \times 275.51$

1%	-	1520.13
5%	-	978.54
10%	-	745.29
20%	-	512.04
30%	-	375.60
40%	-	278.79
50%	-	203.70
60%	-	142.35
70%	-	90.48
80%	-	45.54
90%	-	5.90
95%	-	-12.29
99%	-	-26.17

Baro

$Q = (K_1 + K_2A) + (K_3 + K_4A)In(D) \times aA^b$

$= (4.2775 + 0.0002x5300) + (-0.9114+(-0.00005 \times 5300) In(D) \times 255.55$

$Q = 5.3375-1.1764 \ In \ (D) \times 255.55$

1%	-	1364.00
5%	-	880.15
10%	-	671.77
20%	-	463.39
30%	-	341.50
40%	-	255.01
50%	-	187.93
60%	-	133.12
70%	-	86.78
80%	-	46.63
90%	-	11.22
95%	-	-5.03
99%	-	-17.43

Validation Stations

From Equation (15)

Zungeru

$Q = (J_1 + J_2A) + (J_3 + J_4A) \text{ In } (D)$

$= (523.48+0.1417x1750+(-114.69 + (-0.0316 \text{ x } 1750) \text{ In}(D)$

$Q = 771.455 – 169.99 \text{ In } D$

1%	-	771.46
5%	-	497.87
10%	-	380.04
20%	-	262.21
30%	-	193.29
40%	-	144.38
50%	-	106.45
60%	-	75.46
70%	-	49.25
80%	-	26.55
90%	-	6.53
95%	-	-2.66
99%	-	-9.67

Shiroro

$Q = (J_1 + J_2A) + (J_3 + J_4A) \text{ In } (D)$

$= 523.48 + (0.1417 \times 900) + (-114.69 + (-0.0316 \times 900) \text{ In } (D)$

$Q = 651.01 - 143.13 \text{ In } (D)$

1%	-	651.01
5%	-	420.65
10%	-	321.44
20%	-	222.23
30%	-	164.20
40%	-	123.02
50%	-	91.08
60%	-	64.99
70%	-	42.92
80%	-	23.81
90%	-	6.95
95%	-	-0.79
99%	-	-6.69

Zungeru

Equation (3.16)

$Q = (L_1 + L_2A)\ \exp[(L_3 + L_4A)\ (D)$

$= (583.79 + 0.1345 \times 1750)\exp[(-0.0389 + (-0.000003 \times 1750)(D)$

$Q = 819.16\exp[(-0.04415)(D)]$

1%	-	783.79
5%	-	656.90
10%	-	526.78
20%	-	338.76
30%	-	217.85
40%	-	140.09
50%	-	90.09
60%	-	57.93
70%	-	37.25
80%	-	23.96
90%	-	15.41
95%	-	12.35
99%	-	10.35

Agaie

$Q = (L_1 + L_2A) \exp[(L_3 + L_4A)(D)$

$= (583.79 + 0.1345 \times 900)\exp[(-0.0389+(-0.000003 \times 900)(D)]$

$Q = 704.84\exp[(-0.0416)(D)]$

1%	-	676.72
5%	-	572.48
10%	-	464.97
20%	-	306.73
30%	-	202.34
40%	-	133.48
50%	-	88.06
60%	-	58.09
70%	-	38.32
80%	-	25.28
90%	-	16.68
95%	-	13.54
99%	-	11.47

Zungeru

Equation (19)

$Q = (K_1 + K_2A) + (K_3 + K_4A)\text{In}(D) \times aA^b$

$= (4.2775 + 0.0002x1750) + (-0.9114+(-0.00005 \times 1750) \text{In}(D) \times 150.21$

$Q = 4.6275-0.9989 \text{ In } (D) \times 150.21$

1%	-	695.10
5%	-	453.61
10%	-	349.61
20%	-	245.60
30%	-	184.76
40%	-	141.60
50%	-	108.12
60%	-	80.76
70%	-	57.63
80%	-	37.60
90%	-	19.92
95%	-	11.81
99%	-	5.62

Agaie

$Q = (K_1 + K_2A) + (K_3 + K_4A)In(D) \times aA^b$

$= (4.2775 + 0.0002x900) + (-0.9114+(-0.00005 \times 900)\ In(D) \times 109.21$

$Q = 4.4575-0.9564\ In\ (D) \times 109.21$

1%	-	486.80
5%	-	318.70
10%	-	246.30
20%	-	173.90
30%	-	131.55
40%	-	101.51
50%	-	78.20
60%	-	59.16
70%	-	43.05
80%	-	29.11
90%	-	16.81
95%	-	11.16
99%	-	6.85

Zungeru

Equation (3.20)

$Q = (M_1 + M_2A) \exp [(M_3 + (M_4A) (D)] \times aA^b$

$= (3.8389 + 0.0003x1750) \exp [(-0.0396+(-0.00003 \times 1750)(D) \times 150.21$

$Q = 4.3639 \exp (-0.04485xD) \times 150.21$

$Q = 655.501419 \exp (-0.04485 \times D)$

1%	-	626.75
5%	-	523.82
10%	-	418.59
20%	-	267.31
30%	-	170.70
40%	-	109.01
50%	-	69.61
60%	-	44.45
70%	-	28.39
80%	-	18.13
90%	-	11.58
95%	-	9.25
99%	-	7.73

Agaie

$Q = (M_1 + M_2A) \exp [(M_3 + (M_4A) (D) \times aA^b$

$= (3.8389 + 0.0003x900) \exp [(-0.0396x900)(D) \times 109.21$

$Q = 4.1089 \exp (-0.0423 \times D) \times 109.21$

$Q = 448.732969 \exp (-0.0423 \times D)$

1%	-	430.15
5%	-	363.19
10%	-	293.96
20%	-	192.56
30%	-	126.14
40%	-	82.63
50%	-	54.13
60%	-	35.46
70%	-	23.23
80%	-	15.22
90%	-	9.97
95%	-	8.07
99%	-	6.81

APPENDIX D

Monthly and Annual Flows of River Niger at Baro from 1948 to 2000

Cubic meters per second

Year	*Jan.*	*Feb.*	*March*	*April*	*May*	*June*	*July*	*Aug.*	*Sept.*	*Oct.*	*Nov.*	*Dec.*	*Annual*
1948	24.6	13.6	9.8	7.43	47.1	198	372	640	825	459	276	99.2	248
1949	49.1	32	23.7	16.3	15.6	25	93.8	391	907	383	251	93.5	190
1950	33.2	40	25.4	22.7	25.1	39.8	132	318	785	562	244	91.4	193
1951	59.5	26.4	33.5	20.4	39.5	184	488	739	683	853	743	188	338
1952	101	39.1	36.4	22.9	30	76.4	272	582	814	811	352	133	272
1953	37.6	20.3	23.4	22.7	34.5	291	448	743	928	704	365	180	316
1954	95.4	58.1	37.2	59.2	72.8	185	378	512	851	672	533	270	310
1955	126	65.7	71	49	88.3	242	523	731	1,080	732	378	196	357
1956	103	59.7	44.5	41.5	67.9	84.9	326	303	751	511	217	128	220
1957	61.2	33.5	27.8	13.3	30.3	153	499	609	990	857	423	172	322
1958	95.9	52.7	34.6	56.8	127	359	295	215	801	651	472	253	284
1959	109	55.4	29.8	21.3	48.9	158	411	519	866	510	351	136	268
1960	68.9	36.5	19.3	14.8	31.3	147	356	754	902	598	332	129	282
1961	63	35.7	14.5	9.35	31.7	44.9	267	489	481	373	176	67.9	171
1962	31.8	10.2	4.13	4.42	42.2	96.3	198	793	1,150	620	383	193	294
1963	93.1	61.3	31	29	46	48.8	238	404	632	753	253	91.7	223
1964	43.9	20.2	6.23	4.3	20.7	144	309	535	793	565	269	166	240
1965	79.5	53.4	26.7	19.8	37.1	231	500	292	668	447	212	78.4	220
1966	37.0	20.6	14.5	6.76	21.2	115	267	485	429	537	295	91.4	193
1967	43.4	21.9	14.6	6.54	24.8	49.9	182	371	819	894	417	161	250
1968	75	52.2	28.5	23.3	41.4	339	290	603	952	633	351	185	298
1969	87.4	45	32.4	24.1	27.1	139	614	763	943	991	502	162	361
1970	88.2	49.5	35.1	30.4	23.1	47.3	162	326	606	365	206	101	170
1671	44.5	23.4	10.7	10.5	27	110	211	697	808	454	180	123	225
1972	53.3	27.4	10.3	38.9	61.9	248	394	544	666	588	223	129	249
1973	55.1	27.6	8.92	6.31	14.7	82.7	168	375	551	342	221	58.5	159
1974	26.8	12.7	3.87	12.8	8.03	35.3	349	537	978	621	203	77.3	239
1975	36.2	13.5	4.36	2.25	28.7	48.3	183	437	879	638	321	120	226
1976	57.1	30.0	20.8	15.7	36	90.6	145	172	393	779	612	184	211
1977	88.8	48.1	33.0	25.5	45.1	65.2	143	299	583	368	166	56	160

1978	25.5	12.0	8.7	4.85	15.1	190	310	480	713	585	315	113	231
1979	60.4	31.9	13.9	9.32	15.4	166	671	818	664	596	339	122	292
1980	60	33.3	12.9	3.79	4.72	36.9	127	381	502	295	263	111	153
1981	46.3	21	8.48	17.4	68.5	103	348	713	734	559	182	62.9	239
1982	40.7	22.2	8.94	12.2	51.2	103	271	365	454	347	125	56.0	155
1983	25.6	12.0	8.7	9.09	22.5	155	197	430	523	386	173	57.0	167
1984	26.1	12.3	8.9	7.80	20.7	63.6	110	505	295	302	131	28.1	126
1985	11.8	4.2	3.4	3.66	2.03	7.42	131	457	617	258	105	40.6	137
1986	10	2.75	0.853	0.772	0.866	15.6	74	386	576	373	203	45.8	141
1987	20.6	9.2	6.8	6.83	19.4	80.7	184	390	475	430	144	57.0	152
1988	26.1	12.3	8.9	5.75	17.9	11.2	44	387	516	248	113	13.8	117
1989	4.8	0.2	0.7	1.79	12.4	35.6	71	352	442	441	143	30.6	128
1990	12.7	8.7	5.0	3.5	3.5	35.9	57.7	225	423	280	116	45.2	101
1991	20.3	9.0	6.6	5.50	17.5	89.8	129	374	329	463	173	44.3	138
1992	19.8	8.7	6.5	3.61	14.9	89.8	201	376	425	311	173	29.2	138
1993	12.4	4.5	3.6	3.21	14.4	35.9	93	455	421	378	234	56.4	143
1994	25.8	12.1	8.8	3.03	14.1	62.8	109	267	787	729	497	102	218
1995	48.3	25.0	17.4	7.42	20.2	22.4	123	371	488	494	167	33.3	151
1996	14.4	5.7	4.4	3.74	15.1	76.6	113	321	719	888	640	158	247
1997	75.8	40.7	28.0	12.4	27.1	88.0	293	476	804	402	162	33.1	204
1998	14.3	5.6	4.4	2.07	12.8	89.6	211	570	621	495	150	25.3	183
1999	10.5	3.4	2.9	2.61	13.5	7.7	29	133	629	615	261	51.5	147
2000	23.4	10.8	7.8	4.03	15.5	88.6	532	191	317	672	234	42.9	178
Average	**49.1**	**26.3**	**17.0**	**14.6**	**30.5**	**103**	**257**	**464**	**679**	**544**	**282**	**103**	**215**

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Monthly and Annual Flows of River Chanchaga at Agaie

Year	*Jan.*	*Feb.*	*March*	*April*	*May*	*June*	*July*	*Aug.*	*Sept.*	*Oct.*	*Nov.*	*Dec.*	*Annual*
1975	39.3	17.2	10.75	6.33	10.4	32.1	172	566	905	886	652	117	336
1976	53.6	19.0	9.10	4.32	11.3	87.3	171	491	626	820	636	181	304
1977	84.3	30.2	14.00	6.15	8.27	24.3	72.7	255	541	400	143	47	151
1978	31.0	16.7	9.77	6.41	24.0	65.9	173	371	774	612	322	98	246
1979	45.1	16.1	7.26	4.34	5.9	62.6	3.1	699	650	522	320	100	267
1980	54.7	21.4	8.03	3.97	3.74	27.0	78.8	293	555	266	190	95	152
1981	38.1	20.0	7.08	4.90	30.9	104	282	528	650	541	425	70	264
1982	32.1	14.1	7.12	4.86	15.4	48.8	171	374	513	309	234	63	174
1983	28.7	12.0	5.99	4.02	5.86	69.2	165	316	555	306	147	55.0	163
1984	22.0	10.7	5.76	3.70	7.40	35.9	161	404	358	283	166	48	147
1985	19.1	8.60	4.89	2.49	1.15	2.29	36.5	463	650	414	178	49	180
1986	18.8	7.74	4.11	2.40	2.0	22.5	95.7	306	614	380	208	54	169
1987	14.3	9.40	4.20	2.22	1.7	51.4	263	443	479	447	51.1	62	180
1988	20.0	10.0	5.17	2.44	1.07	3.15	114	374	587	207	94.8	28.3	142
1989	13.0	7.16	3.91	1.52	1.6	27.4	95.2	262	465	362	183	59	146
1990	13.1	8.4	4.1	2.4	2.4	3.3	65.1	259	488	323	133	50.7	113
1991	18.2	7.3	3.5	1.8	1.8	7.2	54.2	198	317	316	130	42.6	92
1992	13.3	6.1	2.8	1.6	3.2	14.5	55.9	247	411	322	125	44.9	104
1993	16.6	11.1	10	5.7	2.2	61.2	37.8	332	387	258	123	41.6	108
1994	13.1	11.9	11.2	4.3	2.2	51.8	149	413	750	833	478	117	237
1995	49.6	24	12.2	7.8	11.2	30.2	104	179	275	253	122	38	92
1996	56.5	29.8	16	9.9	10.9	30.5	136	442	658	576	186	79.5	186
1997	36.9	19.7	10.1	5.8	5.4	31.2	107	217	562	540	259	85.2	157
1998	37	17	8	3.6	3.8	34.7	142	424	547	548	171	59.7	167
1999	25	10.9	5.4	3.6	3.2	12.4	54.9	168	460	533	234	75.9	133
2000	34.4	16.9	8.4	5.2	5.6	37.2	152	387	850	905	350	124	240
Average	**52.6**	**25.4**	**13.4**	**8.76**	**12.3**	**57.4**	**180**	**414**	**682**	**595**	**299**	**112**	**232**

Monthly and Annual Flows of River Kaduna from 1979 to 2006

Year	*Jan.*	*Feb.*	*March*	*April*	*May*	*June*	*July*	*Aug.*	*Sept.*	*Oct.*	*Nov.*	*Dec.*	*Annual*
1979	45.7	23.1	19.8	6.87	18.6	108	298	453	766	388	125	60.3	193
1980	331	14.8	7.69	9.02	33.2	117	419	569	628	267	186	73	196
1981	41	24.7	28.5	34	33.9	37	229	599	763	348	162	70.5	198
1982	52	35.8	21.4	13.1	30	45.2	200	271	533	547	198	73.5	168
1983	45	31.6	28.9	29.5	76.1	142	397	604	641	697	531	146	281
1984	77.3	45.5	36.4	30.6	36.9	73.8	243	598	685	585	221	94.1	227
1985	64.6	31	29.7	19.9	27.4	172	347	595	688	469	198	105	229
1986	61.3	42.1	35.2	49.5	60.9	155	407	599	678	622	379	151	270
1987	80.6	443	48.6	46.3	69.9	205	413	598	737	620	237	125	269
1988	68	46.2	44	46.9	39.2	64.4	167	308	509	367	139	77	156
1989	39.6	21.6	20.2	15.6	29.4	87.7	276	471	712	629	244	104	221
1990	55.2	34.2	18	39.4	99.3	275	286	195	605	461	219	132	202
1991	57	30.6	18.2	13.6	32.7	72.6	351	325	787	327	188	73.6	190
1992	34.3	16.9	9.51	19.9	51.4	124	270	720	828	511	219	89.8	241
1993	45.4	25.4	12.2	11.3	34.1	35.1	183	363	484	297	123	53.4	139
1994	28.4	14.8	11.9	20	42.1	72.8	254	459	774	560	268	111	218
1995	53	45.3	32.5	31	55.4	62.2	192	404	662	516	180	68.1	192
1996	32.7	17.8	10.1	10.9	16.8	103	173	498	506	429	164	121	174
1997	65.8	40.5	30	18.4	36.6	136	421	267	565	309	137	56.4	174
1998	16.5	11.8	8.84	11.40	25.8	86.3	232	461	375	391	184	63.7	156
1999	30.5	17.3	13.3	15.5	32.8	55.8	255	534	751	706	227	88.4	227
2000	43.3	26	16.3	21.3	38.2	209	200	530	571	382	210	105	196
2001	59.3	37.7	35.8	31.6	27.8	103	441	757	786	654	355	121	284
2002	67.2	38.7	36.7	41.4	39.7	73.6	167	399	613	280	139	76.4	164
2003	44.3	33.4	21.3	25.9	38.7	54.4	167	455	598	341	115	76.2	164
2004	34.4	20.8	12.2	27.8	59.9	178	292	470	585	359	138	79.7	188
2005	36.4	18.4	9.33	10.8	13.8	44.3	93.2	450	545	303	173	57.1	146
2006	32.1	15.2	10.9	10.1	9.52	27.7	290	467	795	464	145	56.8	194

Cubic meters per second

Monthly and Annual Flows of River Kaduna at Zungeru from 1984 to 2000

Month	*1982*	*1983*	*1984*	*1985*	*1986*	*1987*	*1988*	*1989*	*1990*	*1991*	*1992*	*1993*	*1994*	*1995*	*1996*	*1997*	*1998*	*1999*	*2000*
Jan.			N0	518	1043	690	783	869	863	615	1018	842	676	1517	947	1135	853	1403	157
Feb.			378	513	631	711	577	613	686	745	805	1059	782	1637	1023	1335	886	1348	137
Mar.			417	484	647	679	447	607	573	526	710	1003	831	1422	969	1046	884	1063	141
Apr.			436	441	670	683	809	777	874	616	759	881	839	1060	784	1093	895	1119	1353
May			515	428	678	596	431	484	1146	716	731	604	749	1126	782	711	860	938	1131
June			794	391	814	791	359	636	963	1227	871	402	637	1086	609	872	1144	968	804
July			699	450	719	657	489	758	433	1567	552	378	613	976	736	726	1341	854	620
Aug.			800	680	681	733	893	1077	545	2379	456	445	1209	1308	862	553	1402	1595	1378
Sept.			797	1518	750	1019	2675	1393	1199	1678	817	960	3106	1357	1287	1119	2590	3182	1321
Oct.			671	1361	741	738	1777	1409	666	1364	1091	736	3275	1414	1169	919	3636	3250	1217
Nov.			516	711	749	643	941	658	739	862	976	651	1469	1201	1003	742	1533	1688	646
Dec.			619	1093	839	685	1210	995	1067	851	927	610	1263	1090	1045	738	1213	1565	1492
Total			6642	8588	8962	8625	8775	10640	9454	13146	9641	8571	15449	15194	11216	10989	17237	18973	1433
AVG			553.5	715.7	746.8	718.7	731.3	886.7	787.8	1095.5	803.4	714.3	1287.4	12662	934.7	915.	436.	581.	194.

Monthly and Annual Flows of River Gurara Kachia from 1975 to 2000

Cubic meters per second

Year	*Jan.*	*Feb.*	*March*	*April*	*May*	*June*	*July*	*Aug.*	*Sept.*	*Oct.*	*Nov.*	*Dec.*	*Annual*
1975	29.5	15.8	8.47	21.6	42.5	88.9	238	504	773	498	145	66.8	203
1976	39.1	21.7	12.7	12.5	41.4	66.7	178	374	517	616	412	109	200
1977	62.3	34.4	18	11.5	16.7	51.6	146	274	561	270	97.9	44.7	132
1978	21.2	13.9	9.03	23.8	32.3	152	247	456	654	453	203	73.5	195
1979	43.4	21.1	15.6	25.2	24.9	101	439	693	656	476	205	76.2	231
1980	45.2	32.2	15.9	10.3	24.8	55.1	111	361	470	200	135	64.6	127
1981	34.8	19.8	12.2	15.8	42.5	51.7	357	694	634	279	106	49.7	191
1982	24.4	14.4	8.55	20.7	53	67.6	253	428	518	272	127	47.2	153
1983	23.2	13.2	5.63	5.99	15.2	115	183	406	518	285	103	44.5	143
1984	20.1	8.18	5.49	5.86	22	51.6	123	481	355	215	73.5	29.7	116
1985	10.2	4.06	2.18	2.6	5.09	16.4	170	603	680	332	95.1	35.8	163
1986	14.6	5.98	3.05	3.63	8.75	16.7	97.3	361	556	274	124	38.8	125
1987	17.8	8.99	3.73	2.41	5.39	62.5	174	366	484	322	82.3	44.5	131
1988	15.2	5.15	2.67	1.99	2.18	13.6	76.4	362	513	169	60.8	22.4	104
1989	5.73	3.44	2.18	1.18	4.52	30.8	95.4	327	460	332	81.8	31	115
1990	9.7	3.05	1.25	1.48	15.7	31	101	335	407	220	80.7	33.3	103
1991	14.6	7.4	3.9	2.12	17.2	68.9	136	349	379	350	103	38	122
1992	10.1	4.7	1.49	0.594	6.43	68.9	186	351	448	222	103	30.3	119
1993	9.14	3.62	1.67	7.61	9.1	31	111	431	445	279	146	44.2	127
1994	8.7	2.5	0.8	0.6	1.6	49.9	122	242	707	574	331	67.6	176
1995	19.2	4.5	2.8	7.7	18.1	21.5	132	346	493	376	98.7	32.4	129
1996	10.4	4.9	6	12.3	18.3	59.6	125	296	658	708	432	96.2	202
1997	31.2	8.2	1.7	0.4	9.1	67.6	250	452	719	299	95.5	32.3	164
1998	6.4	2.1	0.3	0.3	13.9	68.7	193	546	588	377	86.9	28.3	159
1999	21.6	9.67	4.93	5.13	8.31	11.2	66.1	107	594	478	165	50.8	127
2000	11.1	2.1	0.1	0.9	7.468	417	166	370	526	146	146	37.3	146
Average	**35.2**	**20.0**	**14.4**	**16.0**	**29.2**	**81.6**	**229**	**439**	**599**	**412**	**179**	**70**	**177**

Monthly and Annual Flows of River Gurara at Izom from

Cubic meters per second

Year	*Jan.*	*Feb.*	*March*	*April*	*May*	*June*	*July*	*Aug.*	*Sept.*	*Oct.*	*Nov.*	*Dec.*	*Annual*
1981	34.0	23.0	15.5	13.0	11.0	37.6	162	157	535	649	236	76.6	189
1982	52.5	32.2	24.4	17.4	33.5	94.8	324	527	760	709	916	203	361
1983	86.7	46.8	24.5	13.4	15.8	50.1	230	346	639	751	256	101	243
1984	65.7	32.0	21.7	11.8	18.8	126.2	386	738	988	912	355	151.1	371
1985	106	82.9	70.6	72.0	34.3	109	224	350	703	672	540	292	307
1986	131	69.8	37.1	26.2	35.8	176	347	484	1,121	982	481	266	396
1987	130	85.6	45.6	27.2	20.9	74.2	208	304	561	433	212	105	199
1988	46.3	19.4	10.1	4.6	6.1	44.9	217	486	1,040	1,065	499	194	357
1989	110	56.7	26.7	21.2	39.9	121	138	192	636	774	399	256	260
1990	122	47.3	23.4	7.9	10.7	97.7	225	316	681	429	276	113	218
1991	56.1	20.1	8.3	4.4	7.0	153	349	953	1,098	673	283	119	365
1992	52.4	23.4	11.1	5.0	4.2	5.6	207	531	789	383	194	88.3	222
1993	40.3	17.0	13.5	23.2	50.4	86.6	304	550	928	671	368	179	317
1994	94.0	51.9	22.5	9.6	14.2	16.5	182	446	716	940	353	143	284
1995	69.8	26.8	10.7	5.2	8.5	98.7	236	421	820	808	298	148	286
1996	79.3	33.2	15.0	6.5	8.8	56.3	335	699	1,189	1,187	439	129	406
1997	36.1	24.0	15.5	11.0	8.4	54.3	150	322	613	813	381	225	259
1998	151	44.1	12.1	7.0	10.7	83.5	163	441	953	1,107	369	149	329
1999	76.5	47.3	19.8	10.3	20.1	142	192	494	809	590	256	143	268
2000	65.7	27.8	15.2	9.6	7.8	54.1	251	530	941	1,091	538	180	362
2001	86.3	41.3	18.8	9.73	8.25	17.7	113	235	697	401	214	102	182
2002	39.7	15.7	7.49	4.36	9.87	33.3	151	723	758	480	173	118	246
2003	42.4	16.7	7.51	7.11	12.8	129	341	455	673	530	401	142	270
2004	51.9	14.6	7.45	3.37	5.76	55.8	134	471	542	378	219	83.5	190
2005	30.5	13.2	7.46	4.14	40.4	29.1	226	549	946	767	334	98.0	300