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# THE APPLICATION OF SYNTHETIC UNIT HYDROGRAPH TO EVALUATE FLOOD DISCHARGE AT THE LOWER NIGER RIVER

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**Abstract**
The evaluation of the flood discharge at the lower Niger river was carried out by employing the tool of the Geographical Information System (GIS). The flood discharge research work at the catchment could serve the purpose of planning, and early warning strategy to both individuals and government structures with a view of effectively curbing the occurrence of floods, averting loss of lives and properties and checking the outbreak of epidemics. The analyzed watershed attribute parameters, rainfall data and hydrological models were used in forecasting the peak runoff for return periods of 25yr, 50yr, 75yr, and 100yr using the SCS methods. And from the determined hydrograph the return periods of 25yr has the lowest peak runoff with 2883.89$m^3$/s while the 100yr return period has the highest peak runoff of 3616.19$m^3$/s respectively.

**Key words**
bifurcation ratio, catchment, discharge , flood, GIS, formula, Lower, Morphometric Parameters, Niger, peak flow, Stream Flow, River, Slope, stream , soil conservation service, Synthetic unit hydrograph,

## 1.0 Introduction

A **sy**nthetic unit hydrograph is a method that retains all the features of the unit hydrograph, but does not require rainfall-runoff data (Victor). A synthetic unit hydrograph is derived from theory and experience, and its purpose is to simulate basin diffusion by estimating the basin lag based on a certain formula or procedure. The use of synthetic hydrograph also provides an advantage of geomorphological data generated from the Digital Elevation Model (DEM) through the correlation of various hydrologic phenomena with the physiographic characteristics of drainage basins such as size, shape, slope of drainage area, drainage density, size and length of the tributaries.

The challenges of procuring hydraulic gauge equipment, installation and management in developing countries have prompted the desire for the use of the synthetic unit hydrograph. Some techniques have evolved that allow generation of synthetic unit hydrograph. These include Snyder's method, Soil Conservation Service (SCS) Method, Gray's Method and Clark's Instantaneous Unit Hydrograph Methods. The peak discharges of stream flow from rainfall can be obtained from the designed storm hydrographs developed from unit hydrographs generated from established methods. Salami (2009) described hydrograph as a continuous graph showing the properties of stream flow with respect to time, normally evaluated by means of a continuous strip recorder that indicates stages versus time and is then transformed to a discharge hydrograph by application of a rating curve. He also observed that with an adjustment and well measured rating curve, the daily gauge readings may be converted directly to runoff volume. He also emphasized that catchment properties such as area, slope, orientation, shape, altitude and also stream influence runoff to a large or small degree. Aside the advantage of resources saving, the method also provides both soil and hydrological information of the catchment.

## 2.0 Material and methods

The evaluation of the of flood discharge at the lower Niger river carried out by employing the tool of the Geographical Information System (GIS) and 40 years (1971-2010) rainfall data of the catchment. The remotely sensed data is geometrically rectified and the digitization of dendritic drainage pattern is carried out in Arc GIS 10.1 software. The software was used for the delineation of the Niger South Catchment, of the lower Niger River as shown in fig 2.1. Consequently, information in table 2.2 were derived by-products of the Digital Elevation Model (DEM)

## 2.1 Literature Review

### 2.1.1 Peak Flow Formula

Numerous methods are available for predicting peak flood (the maximum flood discharge) required for design application in small and rural watersheds. Some of the methods are empirical or by correlating the flow rate with simple drainage basin characteristics such as slope, length, or area whereas some are based on rational analysis of the rainfall-runoff process Mustafa (2012).

**a. Rational Formula**

The rational formula for estimating peak runoff rate was introduced by Emil and the formula is

$Q_p = 0.00278CiA$ (2.1)

Where $Q_p$ = the peak flow rate = ($m^3/s$)

C= runoff coefficient assumed to be dimensionless

i = average rainfall intensity (mm/hr), lasting for a critical period of time

A= size of the drainage area (ha)

C = the net rain intensity (mm/hr) at $t=t_c$ ($t_c$ is the time of concentration)

The formula is valid for small urban basin and agricultural land between 100-200ha in area.

Typical C value for storm of 5-10 years in return periods are provided in table 2.4

The use of rational formula is based on the concept that application of steady, uniform rainfall intensity will cause runoff to reach its maximum rate when all parts of the watershed are contributing to the outflow at the point of design. This condition is met after the elapsed time $t_c$, the time of concentration. At this time, the net rain rate matches the runoff rate. The following steps are in determining peak flow rate.

Estimate the time of concentration of the drainage area

Estimate the runoff coefficient, C

Select a return period, T, and find the intensity of rain that will be equalled or exceeded, on the average, once every T years. This design storm must have duration equal to $t_c$. The desired intensity is read from a locally derived

Intensity Duration Curve (IDC) using rainfall duration equals to the time of concentration. If the IDC are not available for the catchment and a maximum precipitation of P cm occurs during a storm period of $t_r$ hours, then the design intensity $i=i_c$ can be obtained from

$$I_c = t_r = \frac{P}{t_r}\left(\frac{t_{r+1}}{t_{c+1}}\right) \quad (2.2)$$

If $t_c$ is not known, $i_c$ can be approximated from $i_c = p/t_r$. Recurrence intervals are normally of the order of 10-25years.

Table 2.1: Typical C coefficients for 5-10 year frequency

| Description of Area | Runoff Coefficients |
|---|---|
| Business | |
| Down town areas | 0.7-0.95 |
| Neighbourhood areas | 0.5-0.70 |
| Residential | |
| Single family areas | 0.3-0.5 |
| Multiunit, detached | 0.4-0.6 |
| Multiunit, attached | 0.6-0.75 |
| Residential (suburban) | 0.25-0.40 |
| Apartment dwelling areas | 0.50-0.70 |
| Industrial | |
| Light areas | 0.50-0.80 |
| Heavy areas | 0.60-0.90 |
| Parks, cemeteries | 0.10-0.25 |
| Play grounds | 0.20-0.35 |
| Rail road yard areas | 0.20-0.40 |
| Unimproved areas | 0.10-0.30 |
| Street | |
| Asphaltic | 0.70-0.95 |
| Concrete | 0.80-0.95 |
| Bricks | 0.70-0.85 |
| Walks | 0.75-0.95 |
| Roofs | 0.75-0.95 |
| Lawns; sandy soil: | |
| Flat, 2% | 0.05-0.10 |
| Average, 2-7% | 0.10-0.15 |
| Steep, 7% | 0.15-0.20 |
| Lawns; heavy soil: | |
| Flat, 2% | 0.12-0.17 |
| Average, 2-7% | 0.18-0.22 |
| Steep, 7% | 0.25-0.35 |

Source: Mustafa 2012

**b. Estimation of Time of Concentration**

Observed values of time of concentration are rarely available. Designers normally make estimates of $t_c$ using empirical formula such as the one developed by Kirpich, (1940) for small agricultural basins. (A<50ha)

$$t_c = \frac{L^{1.15}}{3080H^{0.38}} \quad (2.3)$$

where $t_c$ = time of concentration (hr), L= maximum travelling distance in the basin (m) H= difference in elevation over the above distance (m).

The time of concentration may also be derived by dividing the travelling distant, L, by the velocity of flow, V, i.e $t_c$ = L/V

$$t_c = \sum \frac{L_i}{V_i} \quad (2.4)$$

where $L_i$ and $V_i$ respectively represent the travelling distance and the velocity of flow in the individual reaches. The velocity of flow in the drainage canals may be estimated using Maning's formula (Mustafa, 2012).

**c. Empirical Formula**

A multitude of peak flow formulae relating the discharge to the drainage area and other basin characteristics have been proposed and applied. Chow and Gray have listed 35 such formulae and compared many others. Example of such is

$Q_P = CA^m$ (2.5)

Where m and C, are regression constants, A= drainage area, $Q_p$ = the peak discharge associated with a given return period; m is usually between 0.5 and 1.2

$Q = 175\sqrt{A}$ (2.6)

**a. Curve Number Method**

The curve number method is applicable to basins larger than the areas considered by the rational method. The method is applicable to several thousand hectares of land and complete hydrograph of stream flow can be generated to obtain the design storm. The method developed by the United States Soil Conservation Service (USSCS) involves the following steps:

i. Based on catchment characteristics, rainfall is converted into surface runoff using curve number graph.
ii. Discharge is converted into a basin hydrograph using USSCS dimensionless unit hydrograph.
iii. Determination of design discharge $Q_{design}$ as $Q_{design}$ = q x A where q is the drainage coefficient taken as the peak of the hydrograph and A is the basin area

**2.1.2 Design Flood from Flood Frequency Analysis**

The design of any hydraulic structure is based on hydrologic events which are random in nature due to the uncertainties in their occurrences. Since no complete information or long years of observed records are available for the planning or designing hydraulic structures, the concept of probability is normally utilized by hydrologists to forecast future events with some degree of accuracy. It is frequently required to estimate the maximum possible discharge of a particular river in order to size spillway, reservoir capacity, bridge etc. The probability of an event being equalled or exceeded is employed by hydrologists in designing water resources structures since it is not possible to make the exact prediction of such event due to its randomness in occurrence. This section deals with probability techniques for estimating the magnitude and the frequency of the hydrologic event for the safety design of flood control structures.

In the selection of data for probability analysis, the data series must be relevant to the problem in question, adequate, and homogeneous (Mustafa, 2012). Most flood studies are concern with peak flows and as such observed peaks are selected for such studies. Similarly, the length of record must be adequate enough and homogeneous in nature.

Sometimes records at a station may not be homogenous owing to changes in the hydrologic characteristics of a catchment. Such changes may introduce inconsistencies, which have to be adjusted to the current conditions.

A data series of maximum annual flood is selected if the analysis is concerned with probabilities less than 0.5. However, for more frequent events, partial duration is preferred.

- **Probability of Event**

The average return period or recurrence interval T is defined as the time which, on average, elapses between two events which equal or exceed a particular event. Mathematically it can be represented as

$T = \frac{1}{P(F)} = \frac{1}{1-P(F^1)}$ (2.7)

Where $P(F^1)$ is the probability that F will not occur in any year. The reciprocal of T is also the P(F) i.e

$P(F) = \frac{1}{T}$ (2.8)

and

$P(F^1) = 1 - P(F) = 1 - \frac{1}{T}$ (2.9)

The probability that F will not occur for n successive years is

$P_1(F^1)\,xP_2(F^1).....P_n(F^1) = P(F_1)^n$ or

$P(F_1)^n = \left(1 - \frac{1}{T}\right)^n$ (2.10)

The probability R called Risk that F will occur at least once in n successive years is given as

$$R = 1- \left[1 - \frac{1}{T}\right]^{n} = 1-(P(F^1))^n \qquad (2.11)$$

## 2.2 Unit Hydrograph Methods

Two different methods of unit hydrographs are described and can be used to synthesize the peak runoff. The methods include: Snyder's and Soil Conservation Service (SCS).

### a. Snyder's Method

Ramirez (2000) reported that the hydrograph characteristics are the effective rainfall duration, $t_r$, the peak direct runoff rate $Q_p$, and the basin lag time, $t_p$. from the given relationships, five characteristics of a required unit hydrograph for a given effective rainfall duration may be calculated. The five characteristics are the peak discharge per unit of watershed area, $q'_p$, the basin lag time, $t'_l$, the base time, $t_b$, and the widths, w ( in time units) of the unit hydrograph at 50 and 75 percent of the peak discharge. The unit hydrograph parameters are estimated in accordance to Ramirez (2002) and Arora (2004).

**i. Lag time, $t_p$**

$$t_p = C_t (LL_c)^{0.3} \qquad (2.12)$$

Where $t_p$ is the lag time (hr) and $C_t$ is a coefficient representing variations of watershed slope and storage. (Values of $C_t$ range from 1.0 to 2.2, Arora (2004)). Equation (2.23) gives the lag time $t_p$ for the watershed.

**ii. Unit-hydrograph duration, $t_r$ (storm duration)**

$$t_r = \frac{t_p}{5.5} \qquad (2.13)$$



From equation (2.24) the duration of the storm can be obtained. However if other storm durations are intended to be generated for the watershed, the new unit hydrograph storm duration ($t'_r$), the corresponding basin lag time ($t'_p$) can be obtained using equation (2.14)

$$t'_p = \frac{t'_r - t_r}{4} \qquad (2.14)$$

***iii.* Peak discharge, $Q'_p$**

The peak discharge ($Q'_p$) can be obtained using Equation (2.15)

$$Q_p = \frac{2.78 * C_p * A}{t_p} \qquad (2.15)$$

Where, $C_p$ is the coefficient accounting for flood wave and storage conditions. (Values of $C_p$ range from 0.3 to 0.93 Arora (2004).

**iv. Base time (days)**

The base time in days can be obtained from Equation (2.16)

$$t_b = 3 + 3\left\{\frac{t_p}{24}\right\} \qquad (2.16)$$

The time width $W_{50}$ and $W_{75}$ of the hydrograph at 50% and 75% of the height of the peak flow ordinate can be obtained using Equations (2.17) and (2.18) respectively in accordance to U.S Army Corps of Engineer (Arora, 2004). The unit of the time width is hr. also the peak discharge per area (cumec/km$^2$) is given by Equation (2.19)

$$W_{50} = \frac{5.9}{(q'_p)^{1.08}} \qquad (2.17)$$

$$W_{75} = \frac{3.4}{(q'_p)^{1.08}} \qquad (2.18)$$

$$q'_p = \frac{Q_p}{A} \quad (2.19)$$

**b. Soil Conservation Service (SCS) Method**

The peak discharge, the time to peak and the lag time can be determined in accordance to SCS (1972), Viessman et al (1989) and Ogunlela and Kasali (2002).

Peak discharge:

The peak discharge can be obtained through the equation (2.20), (Ramirez, 2002).

$$q_p = \frac{2.08A}{t_p} \quad (2.20)$$

where,

$q_p$= peak discharge ($m^3$/s/cm)

A = watershed area ($km^2$)

$t_p$= time to peak(hr)

**Time to peak and lag time**

$$t_p = \frac{t_r}{2} + t_L \quad (2.21)$$

or

$$t_p = \frac{t_c + 0.133t_c}{1.7} \quad (2.22)$$

where;

$t_c$= time of concentration (hr)

$t_r$= storm duration (hr)

$t_L$ = lag time (hr)

$t_c$ = time of concentration (hr) (Kirpich's equation)

$$= 0.06628\left\{\frac{L^{0.77}}{S^{0.385}}\right\} \quad (2.23)$$

where;

L = length of channel (stream) in km

S = Slope of channel (m/m)

$t_L$= Lag time (hr)

$$t_L = 0.6t_c \quad (2.24)$$

$$t_r = \frac{t_L}{5.5} \quad (2.25)$$

The estimated values for both the peak discharge $q_p$ and time to peak $t_p$ are applied to the dimensionless hydrograph ratios to obtain points for the unit hydrograph. The coordinates for the SCS dimensionless unit hydrograph are given in Table 2.2.

Table 2.2: Coordinates of SCS dimensionless unit hydrograph

| $t/t_p$ | $q/q_p$ |
|---|---|
| 0 | 0 |
| 0.1 | 0.015 |
| 0.2 | 0.075 |
| 0.3 | 0.160 |
| 0.4 | 0.280 |
| 0.5 | 0.430 |
| 0.6 | 0.600 |
| 0.7 | 0.770 |

| | |
|---|---|
| 0.8 | 0.890 |
| 0.9 | 0.970 |
| 1.0 | 1.000 |
| 1.1 | 0.980 |
| 1.2 | 0.920 |
| 1.3 | 0.840 |
| 1.4 | 0.750 |
| 1.5 | 0.660 |
| 1.6 | 0.560 |
| 1.8 | 0.420 |
| 2.0 | 0.320 |
| 2.2 | 0.240 |
| 2.4 | 0.180 |
| 2.6 | 0.130 |
| 2.8 | 0.098 |
| 3.0 | 0.075 |
| 3.5 | 0.036 |
| 4.0 | 0.018 |
| 4.5 | 0.009 |
| 5.0 | 0.004 |

**Source: Viessman et al (1989)**

**i. Estimation of design storm (runoff) hydrograph**

The estimated synthetic unit hydrograph from SCS method is used to develop the runoff hydrographs due to annual peak daily rainfall event over the sub-basin. The design runoff hydrographs for selected rainfall of recurrence intervals of 25 year, 50 year, 75 year and 100 year would be developed through hydrograph convolution. Hydrograph convolution involves multiplying the unit hydrograph ordinates by incremental rainfall excess, adding and lagging in a sequence.

**ii. Estimation of Rainfall Excess**

The term rainfall excess or net rain is used to denote a simple numerical subtraction of losses from the precipitation volume. This differentiates it from surface runoff, which refers to part of flow in the receiving water body that was generated by rainfall excess. The unit of rainfall excess is the depth of water on the surface from the excess rain generated during a time interval, while the unit for surface runoff is volume/time. A time lag between the maximum rain excess and the peak of the surface runoff is typical for all but the very few small drainage areas. The time lag (called the peak time) is due to overland and channel flow routing. There are several estimation procedures available for estimation of rainfall excess. The definition of rainfall excess exclude the use of formulas that are based on proportionality between the rainfall and runoff such the well known rational formula presented in d above section. The most common runoff determination procedure is the SCS Runoff Curve Method. (William and Adel, 2000).

The SCS developed a method for estimating rainfall excess that does not require computation of infiltration and surface storage separately. Both runoff characteristics are included into just one watershed characteristic. The method has evolved from analysis of numerous storms under a variety of soil and cover conditions.(William and Adel, 2000).

In the SCS method the excess rain volume, Q, depends on the volume of precipitation, P, and the volume of the total storage, S, which includes both the initial abstraction, $I_a$, and the total infiltration F. the relation between rainfall excess and total rainfall (on twenty-four hour basis) is then (McCuen and Bondelid, 1983)

$$Q = \frac{(P-0.2S)^2}{(P+0.8S)} \qquad (2.26)$$

$$P = \frac{P^*}{24} * P_T \qquad (2.27)$$

where,
P= accumulated rainfall (mm)
$P_T$ = rainfall recurrence interval of the sub-basin (mm)

$P^*$= precipitation ratio, given in Table 2.6 below
S= volume of total storage (mm)
Note that equation (2.26) contains only one unknown, the storage parameter S. This parameter in (mm) can be obtained from

$$S = \frac{25400}{CN} - 254 \tag{2.28}$$

Where;
CN is runoff curve number and can be obtained in Table 2.3 below.
CN value of 75 was adopted based on the soil type and land use of the study area.

Table 2.3 SCS Type II: Useful in estimation of excess rainfall

| \ **Precipitation $p_x/p_{24}$** | **Time (hr)** |
|---|---|
| 0.000 | 0 |
| 0.011 | 1 |
| 0.022 | 2 |
| 0.035 | 3 |
| 0.045 | 4 |
| 0.063 | 5 |
| 0.080 | 6 |
| 0.098 | 7 |
| 0.120 | 8 |
| 0.147 | 9 |
| 0.181 | 10 |
| 0.235 | 11 |
| 0.663 | 12 |
| 0.772 | 13 |
| 0.820 | 14 |
| 0.854 | 15 |
| 0.881 | 16 |
| 0.902 | 17 |
| 0.921 | 18 |
| 0.937 | 19 |
| 0.953 | 20 |
| 0.965 | 21 |
| 0.978 | 22 |
| 0.989 | 23 |
| 1.000 | 24 |

**Source: Viessman et al (1989)**

Table2.4 2.7: Runoff Curve Numbers (CN) for hydrologic soil-cover

| **Land use or crop** | **Treatment or practice** | **Hydrologic Condition** | **Hydrologic Soil Group** | | | |
|---|---|---|---|---|---|---|
| | | | **A** | **B** | **C** | **D** |

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| | | | | | | |
|---|---|---|---|---|---|---|
| Fallow | Straight row | - | 77 | 86 | 91 | 94 |
| Row crops | Straight row | Poor | 72 | 81 | 88 | 91 |
| | Straight row | Good | 67 | 78 | 85 | 89 |
| | Contoured | Poor | 70 | 79 | 84 | 88 |
| | Contoured | Good | 65 | 75 | 82 | 86 |
| | Terraced | Poor | 66 | 74 | 80 | 82 |
| | Terraced | Good | 62 | 71 | 78 | 81 |
| Small grain | Straight row | Poor | 65 | 76 | 84 | 88 |
| | Straight row | Good | 63 | 75 | 83 | 87 |
| | Contoured | Poor | 63 | 74 | 82 | 85 |
| | Contoured | Good | 61 | 73 | 81 | 84 |
| | Terraced | Poor | 61 | 72 | 79 | 82 |
| | Terraced | Good | 59 | 70 | 78 | 81 |
| Close-seeded legumes or rotation meadow | | | | | | |
| | Straight row | Poor | 66 | 77 | 85 | 89 |
| | Straight row | Good | 58 | 72 | 81 | 85 |
| | Contoured | Poor | 64 | 75 | 83 | 85 |
| | Contoured | Good | 55 | 69 | 78 | 83 |
| | Terraced | Poor | 63 | 73 | 80 | 83 |
| | Terraced | Good | 51 | 67 | 76 | 80 |
| Pasture or range | | Poor | 68 | 79 | 86 | 89 |
| | | Fair | 49 | 69 | 79 | 84 |
| | | Good | 39 | 61 | 74 | 80 |
| | Contoured | Poor | 47 | 67 | 81 | 88 |
| | Contoured | | 25 | 59 | 75 | 83 |
| | Contoured | | 6 | 35 | 70 | 79 |
| Meadow(permanent) | | Good | 30 | 58 | 71 | 78 |
| Woods(farm woodlots) | | | 45 | 66 | 77 | 83 |
| Farmsteads | | | 36 | 60 | 73 | 79 |
| Roads and right-of-way | | | 25 | 55 | 70 | 77 |
| Hard surface | | | | | | |
| | | | 59 | 74 | 82 | 86 |
| | | | 74 | 84 | 90 | 92 |



| Soil Group | Description | Final Infiltration Rate (mm/h) |
|---|---|---|
| A | Lowest runoff potential, includes deep sands with very little silt and clay,also deep, rapidly permeable loss | 8 – 12 |

| | | |
|---|---|---|
| B | Moderately low runoff potential, mostly sandy soils less deep than, A and loss less deep or less aggregated than A, but the group as a whole above- infiltration after thorough wetting | 4 – 8 |
| C | Moderately high runoff potential, comprises shallow soils and silts containing considerable clay and colloids , though less than those of group D, the group has average infiltration after presaturation | 1- 4 |
| D | Highest runoff potential, includes mostly clays of high swelling percent, but the group also incudes some shallow soils with nearly impermeablesub-horizons near the surface | 0 – 1 |

**Source: SCS 1972**

**iii. Hydrograph Convolution (Runoff hydrograph development)**

The discrete convolution equation allows the computation of direct runoff $Q_n$.
Let R=incremental rainfall excess (cm)
U= unit hydrograph ordinate ($m^3$/s/cm)
The equations of the ordinates are given in the form of equation below,
Generally:

$$Q_n = R_1U_n + R_2U_{n-1} + R_3U_{n-2} \quad (2.29)$$

Hydrological forecasting
Gumbell's Extreme value type I: The probability of occurrence of a magnitude being equal to or greater than any value $Q_T$ is expressed as

$$P = 1 - e^{-e^{-y}} \quad (2.30)$$

Where e= base of Napierian logarithm
y= reduced variate

$$y = -In\left[-In(1 - \frac{1}{T})\right] \quad (2.31)$$

$$P = \frac{1}{T} \quad (2.32)$$

The event R of the return period T year is defined as

$$R = R_{av} + \sigma(0.78y - 0.45) \quad (2.33)$$

Where
R= Peak annual daily rainfall with magnitude with return period T
$R_{av}$=average value of peak annual daily rainfall
N= number of years of records
σ= standard deviation

$$R = \frac{\sum R_{max}}{N} \quad (2.34)$$

$$\sigma = \sqrt{\frac{N}{N-1}\left(\frac{\sum R^2{}_{max}}{N} - R_{av}{}^2\right)} \quad (2.35)$$

**2.3 Study Area**

The Niger Delta receives the lower Niger river in the Niger South catchment in Nigeria, between the longitude 6$^{o}$E and latitude 8$^{o}$36'N North West, longitude7°37.8'E and latitude7$^{o}$37.2'Nnortheast, longitude5°26.4'E and latitude5$^{0}$ 6'N southwest, longitude7°0.6'E and latitude4$^{o}$25.8'Nsoutheast. The Lower Niger River and the Niger Delta hydrographical region of the Niger River Basin is approximately the Hydrological area 5 (HA5) in Nigeria. States in the hydrological area include Delta, Rivers, Bayelsa, parts of Edo, Anambra and Kogi States.
Vegetation in the project area can be classified into four types. Namely: the Guinea Savannah, Tropical Rainforest, Fresh water Swamp and the Salt water Swamp.

The catchment is part of the rain forest belt, stretches all the way across the southern parts of the country, from east to west and covers the greater parts of Ogun, Ondo, Southern Edo, Delta, Imo, Akwa-Ibom and southern Cross River states.

The climate of the Niger South Catchment is characterized by a long rainy season from March- April through October-November. The precipitation increases from the north of the catchment (with an average of 1,500 mm around Lokoja) to the coastal area of the Niger Delta where mean annual rainfall averages around 4,000 mm, making it one of the wettest areas in Africa**.**

The soils of the Niger South Catchment fall into three zones- (a) interior zone of laterite soils (parts of Kogi State), (b) zone of alluvial soils (parts of Kogi, Edo, Delta, Anambra, Bayelsa, and Rivers States, and (c) southern belt of forest soils (parts of Edo, Delta, Anambra, Bayelsa, and Rivers States).The soils are all of fluviatile origin, except for the Coastal Barrier Islands that consist of marine sand overlain with an organic surface layer.
For many communities in the Niger South Catchment, erosion and the associated flooding constitute serious environmental hazards. Erosion caused by water is predominant in the Catchment. Different types of erosions, such as sheet, rill, and gully, are pervasive in Anambra and Edo States, and to a lesser extent in Kogi State.

**3.1 Data Analysis**

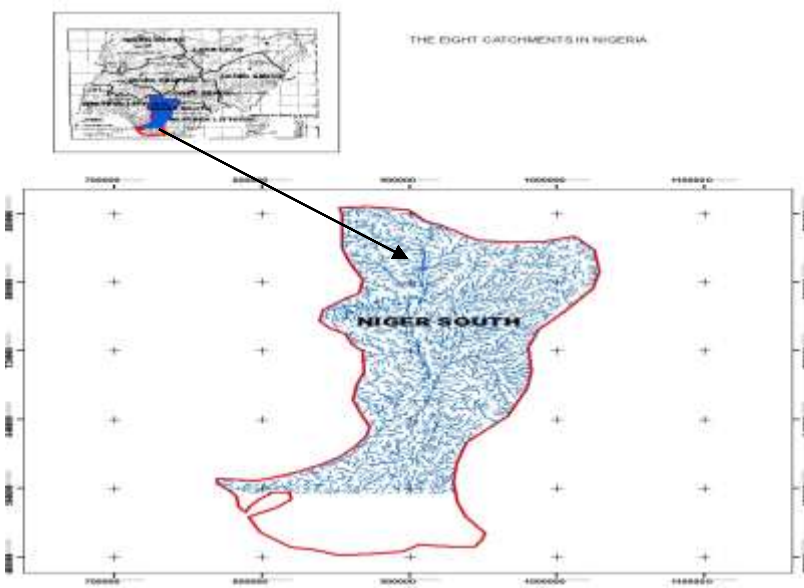


Figure 3.1: Digital Map of Niger Delta showing all the streams

- **Morphometric parameter**

The remotely sensed data was geometrically rectified and the digitization of dendritic drainage pattern was carried out in Arc GIS 10.1 software as indicated in figure 3.1 above. The data obtained were collated and presented in Table 3.1 and 3.2 for linear aspect of the drainage network and Aerial aspects respectively. The table contains information on morphometric parameters such as stream order (Ns), stream length (Ls), drainage density ($D_d$), stream frequency (F), texture ratio($R_f$), comprising the area properties of the drainage basin are computed, forming the basis of analysis of the drainage basin.

Table 3.1: Linear Aspect of the Drainage Network of the Study Area

| **Catchment** | **Stream order U** | **Stream no Nu** | **Stream Length km Lu** | **Stream mean Length km Lu** | **Cumulative Stream mean Length km Lu** | **Log NU** | **Log Lu** |
|---|---|---|---|---|---|---|---|
| Niger-South Hydrological Area V | 1 | 2556 | 74.3 | 2.9 | 2.9 | 3.4 | 6.9 |
| | 2 | 1228 | 39.3 | 3.2 | 6.1 | 3.1 | 6.6 |
| | 3 | 630 | 19.6 | 3.1 | 9.2 | 2.8 | 6.3 |
| | 4 | 321 | 8.7 | 2.7 | 11.9 | 2.5 | 5.9 |
| | 5 | 118 | 3.4 | 2.9 | 14.8 | 2.1 | 5.5 |
| | 6 | 55 | 1.5 | 2.8 | 17.6 | 1.7 | 5.2 |
| **Total (Σ)** | | **4908** | **146.8** | **17.6** | **62.6** | **15.6** | **36.4** |
| **Bifurcation Ratio** | | | | | | | Mean Bifurcation Ratio |
| 2**.08** | | **1.95** | **1.96** | **2.72** | | **2.15** | **2.172** |

Table 3.2: Aerial Aspects of the Study Area

| Morphometric Parameters | Symbol/Formula | Result |
|---|---|---|
| Area (km$^2$) | A | 496.8 |
| Area (km$^2$) | лr$^2$ | 498.8 |
| Perimeter (km) | Pb | 1470.7 |
| Circumference (km) | Pc=2лr | 79.2 |
| Basin Length (km) | $L_b$ | 387.2 |
| Axial width (km) | $W_b$ | 125.2 |
| Slope (S) | $S = \Delta E/L$ | 1.29 |
| Drainage density (km/km$^2$) | $Dd= \Sigma Lu/A$ | 0.3 |
| Constant channel maintenance(C) | 1/Dd | 3.3 |
| Overland flow $L_O$ | 1/2Dd | 1.7 |
| Infiltration number | DdxFs | 2.97 |
| Elongation ratio | $Re= 2R/ L_b$ | 0.07 |
| Circularity ratio | Rc = A/лr$^2$ | 0.9 |
| Compactness coefficient Cc | Pb/Pc | 18.6 |
| Form Factor ratio | $F = \frac{A}{L_b^2}$ | 0.003 |
| Drainage Texture | $Dt=\Sigma Nu/P$ | 3.3 |
| Drainage frequency | $Fs =\Sigma\ Nu/A$ | 9.9 |
| Main channel | | 194.9 |
| Total stream length (km) | | 146.80 |



## 3.2 Estimation of Peak Runoff

The synthetic unit hydrograph from SCS method was used to develop the runoff hydrographs due to annual peak daily rainfall event over the watershed. The design runoff hydrographs for selected rainfall of recurrence intervals of 25 year, 50 year, 75 year and 100 year are developed through hydrograph convolution. Hydrograph convolution involves multiplying the unit hydrograph ordinates by incremental rainfall excess, adding and lagging in a sequence.

### 3.2.1 Application of SCS method to obtain unit hydrograph ordinates

The method of soil conservation service (SCS) for constructing synthetic unit hydrograph was based on a dimensionless hydrograph, which relates ratios of time to ratios of flow, it involved determination of slope of the catchment, S, time of concentration, $t_c$, the time to peak, $t_p$ and the peak flow $Q_p$, in accordance to Viessman et al, 1989) and Ramirez (2000). The equations adopted are equations (2.20) to (2.25) and the parameters used for the analysis are the catchment area (A), length (L), and catchment slope (S) obtained from Tables 3.1 and 3.2, while the obtained parameters are peak discharge ($Q_p$), time of concentration ($t_c$), lag time ($t_L$) and time to peak ($t_p$). They are presented in Table 3.3.

Table 3.3: Parameters for the generation of unit hydrograph

| **L(km)** | **A(km$^2$)** | **$S_c$(slope)** | **$t_c$(hr)** | **$t_L$(hr)** | **$t_p$(hr)** | **$Q_p$(m$^3$/s/cm)** |
|---|---|---|---|---|---|---|
| 146.80 | 496.80 | 1.29 | 3.49 | 2.09 | 2.28 | 452.63 |

The time and the corresponding flow ordinates for the sub-catchments are determined based on the relationship between time and flow presented in Table 2.1. The ordinates and corresponding time for the sub-catchments are presented in Tables 3.4. The corresponding unit hydrograph are presented in Figures 3.2 for the watershed.

Table 3.4: Unit Hydrograph Ordinates for the watershed

| **$t/t_p$** | 0.0 | 0.5 | 1.0 | 1.5 | 2.0 | 2.5 | 3.0 | 3.5 | 4.0 | 4.5 | 5.0 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| **t(hr)** | 0 | 1.14 | 2.28 | 3.42 | 4.57 | 5.71 | 6.85 | 7.99 | 9.13 | 10.27 | 11.41 |
| **$q/q_p$** | 0.00 | 0.43 | 1.00 | 0.66 | 0.32 | 0.16 | 0.08 | 0.04 | 0.02 | 0.01 | 0.00 |
| **q(m$^3$/s/cm)** | 0.00 | 194.63 | 452.63 | 298.73 | 144.84 | 70.16 | 33.95 | 16.29 | 8.15 | 4.07 | 1.81 |

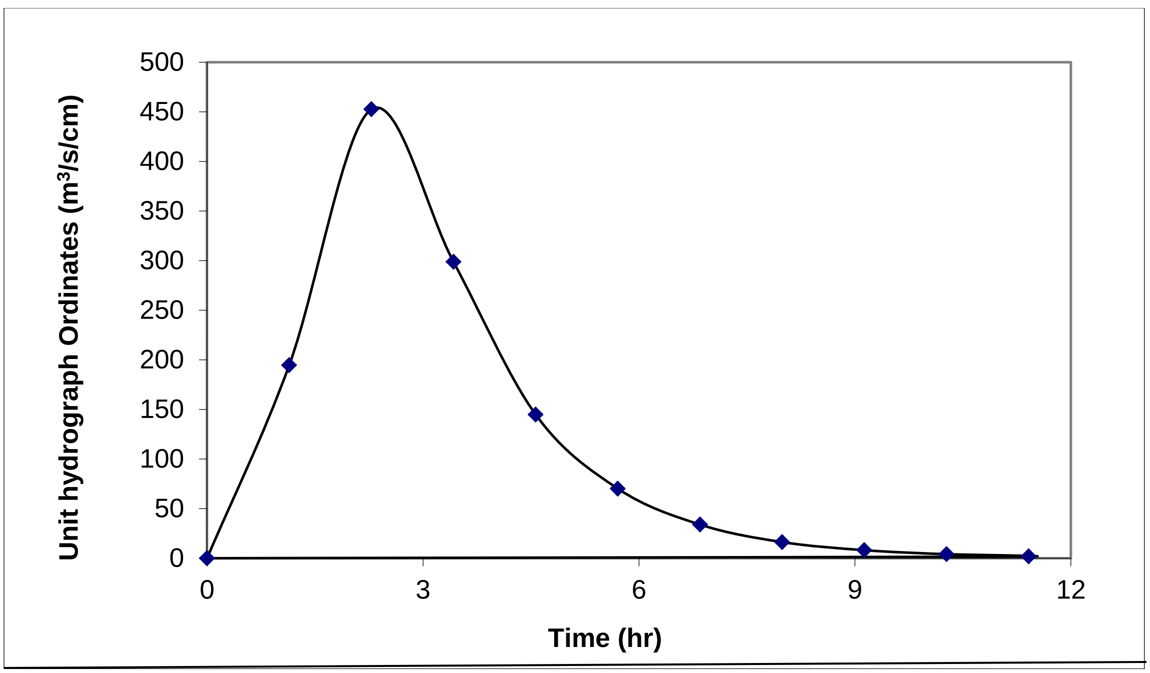


Figure 3.2 Unit hydrograph ordinates versus time for the watershed

### 3.2.2 Determination of rainfall depth of different return periods

To analyze the rainfall data for recurrence intervals of 25 year, 50 year, 75 year and 100 year for the study area. Gumbel's Extreme value type I distribution system was adopted based on equation (2.33). The Gumbel model developed for peak annual daily rainfall for Warri is presented in equation (3.1) with the mean value of 117.19 mm and standard deviation of 25.46 mm.

$$R_T = 117.19 + 25.46(0.78y - 045) \qquad (3.1)$$

In order to determine the rainfall depth of different return periods such as 25yr, 50yr, 75yr and 100yr, equation (2.31) was adopted to estimate the reduced variate (y) while equation (3.1) was adopted to determine the corresponding rainfall value as presented in Table 3.5.

Table 3.5 Corresponding rainfall depth for different return period

| Recurrence Interval | Rainfall(mm) |
|---|---|
| 25-year | 169.27 |
| 50-year | 183.17 |
| 75-year | 191.12 |
| 100-year | 197.07 |

**a. Estimation of Rainfall Excess for different return periods**

Equations (2.26) – (2.28) were used to estimate the rainfall excess, other parameters used include the rainfall depth for different return period and curve number (CN) selected from Table 2.6 based on soil distribution and land use of the study area. Tables 3.8 -3.11 presents the results of excess rainfall for different return period. Curve Number, CN = 75, S=84.67 and Ia= 16.93.

Table 3.6: Estimated rainfall excess for 25yr, 24-hr storm P=169.27mm

| Time (hr) | Precipitation Ratio (P*/24) | Precipitation P (mm) | Cumulative Rainfall Excess $Q_d$ (mm) | Incremental Rainfall Excess (mm) | | |
|---|---|---|---|---|---|---|
| 25 yr, 24-hr storm; $P_T$ = | | | 169.27 | Mm | | |
| 0 | 0.0000 | 0.0000 | 0.0000 | 0.0000 | | |
| 3 | 0.0350 | 5.9245 | 0.0000 | 0.0000 | Runoff Coeff | |
| 6 | 0.0800 | 13.5416 | 0.0000 | 0.0000 | C= | 0.58 |
| 9 | 0.1470 | 24.8827 | 0.0000 | 0.0000 | | |
| 12 | 0.6630 | 112.2260 | 50.4597 | 50.4597 | | |
| 15 | 0.8540 | 144.5566 | 76.7238 | 26.2641 | | |
| 18 | 0.9210 | 155.8977 | 86.3525 | 9.6286 | | |
| 21 | 0.9650 | 163.3456 | 92.7672 | 6.4147 | | |
| 24 | 1.0000 | 169.2700 | 97.9162 | 5.1490 | | |

Table 3.7: Estimated rainfall excess for 50yr, 24-hr storm P=183.17mm

| Time (hr) | Precipitation Ratio (P*/24) | Precipitation P (mm) | Cumulative Rainfall Excess $Q_d$ (mm) | Incremental Rainfall Excess (mm) | |
|---|---|---|---|---|---|
| 50 yr, 24-hr storm; $P_T$ = | | | 183.17 | Mm | |
| 0 | 0.0000 | 0.0000 | 0.0000 | 0.0000 | |
| 3 | 0.0350 | 6.4110 | 0.0000 | 0.0000 | Runoff Coeff |
| 6 | 0.0800 | 14.6536 | 0.0000 | 0.0000 | C=0.60 |
| 9 | 0.1470 | 26.9260 | 0.0000 | 0.0000 | |
| 12 | 0.6630 | 121.4417 | 57.7349 | 57.7349 | |
| 15 | 0.8540 | 156.4272 | 86.8062 | 29.0714 | |
| 18 | 0.9210 | 168.6996 | 97.4187 | 10.6125 | |

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| | | | | | |
|---|---|---|---|---|---|
| 21 | 0.9650 | 176.7591 | 104.4788 | 7.0600 | |
| 24 | 1.0000 | 183.1700 | 110.1405 | 5.6618 | |

Table 3.8: Estimated rainfall excess for 75yr, 24-hr storm P=191.12mm

| Time (hr) | Precipitation Ratio (P*/24) | Precipitation P (mm) | Cumulative Rainfall Excess $Q_d$ (mm) | Incremental Rainfall Excess (mm) | |
|---|---|---|---|---|---|
| 75 yr, 24-hr storm; $P_T$ = | | | 191.12 | mm | |
| 0 | 0.0000 | 0.0000 | 0.0000 | 0.0000 | |
| 3 | 0.0350 | 6.6892 | 0.0000 | 0.0000 | Runoff Coeff |
| 6 | 0.0800 | 15.2896 | 0.0000 | 0.0000 | C=0.61 |
| 9 | 0.1470 | 28.0946 | 0.0000 | 0.0000 | |
| 12 | 0.6630 | 126.7126 | 61.9786 | 61.9786 | |
| 15 | 0.8540 | 163.2165 | 92.6555 | 30.6769 | |
| 18 | 0.9210 | 176.0215 | 103.8299 | 11.1745 | |
| 21 | 0.9650 | 184.4308 | 111.2585 | 7.4286 | |
| 24 | 1.0000 | 191.1200 | 117.2131 | 5.9546 | |

Table 3.9: Estimated rainfall excess for 100yr, 24-hr storm P=197.07mm

| Time (hr) | Precipitation Ratio (P*/24) | Precipitation P (mm) | Cumulative Rainfall Excess $Q_d$ (mm) | Incremental Rainfall Excess (mm) | |
|---|---|---|---|---|---|
| 100 yr, 24-hr storm; $P_T$ = | | | 197.07 | mm | |
| 0 | 0.0000 | 0.0000 | 0.0000 | 0.0000 | |
| 3 | 0.0350 | 6.8975 | 0.0000 | 0.0000 | Runoff Coeff |
| 6 | 0.0800 | 15.7656 | 0.0000 | 0.0000 | C=0.62 |
| 9 | 0.1470 | 28.9693 | 0.0000 | 0.0000 | |
| 12 | 0.6630 | 130.6574 | 65.1904 | 65.1904 | |
| 15 | 0.8540 | 168.2978 | 97.0685 | 31.8782 | |
| 18 | 0.9210 | 181.5015 | 108.6633 | 11.5947 | |
| 21 | 0.9650 | 190.1726 | 116.3674 | 7.7041 | |
| 24 | 1.0000 | 197.0700 | 122.5408 | 6.1735 | |

### 3.3 Hydrograph Convolution (Runoff hydrograph development)

The convolution equations are obtained by computing the direct runoff Qn using equation (2.29), ordinates obtained for the watershed from Tables 3.4 and incremental rainfall excess from Tables 3.6 -3.9. The results of the peak runoff hydrograph are presented in Tables 3.10-3.13 for the four return periods 25yr, 50yr, 75yr and 100yr return period respectively. The summary of the peak storm runoff for various return periods are also presented in Table 3.14. In order for pictorial illustration, the relationships of the synthetic unit hydrograph and the storm runoff hydrograph was also presented in Figure 3.3 – 3.6 respectively for 25 yr, 50 yr, 75 yr and 100yr. The combine hydrographs for various return periods are also presented in Figure 3.10.

Table 3.10: Peak runoff hydrograph for 25yr return period

| **Time** (hr) | **UH ordinate Un ($m^3/s$)** | **$P_1U_n$** | **$P_2U_n$** | **$P_3U_n$** | **$P_4U_n$** | **$P_5U_n$** | **Storm Hydrograph $Q_n$ ($m^3/s$)** |
|---|---|---|---|---|---|---|---|
| 0.00 | 0.00 | 0.00 | | | | | 0.00 |
| 1.14 | 194.63 | 982.10 | 0.00 | | | | 982.10 |
| 2.28 | 452.63 | 2283.96 | 511.29 | 0.00 | | | 2795.25 |
| 3.42 | 298.73 | 1507.41 | 1189.05 | 187.43 | 0.00 | | 2883.89 |
| 4.57 | 144.84 | 730.87 | 784.77 | 435.88 | 124.95 | 0.00 | 2076.47 |
| 5.71 | 70.16 | 354.01 | 380.50 | 287.68 | 290.59 | 100.23 | 1413.01 |
| 6.85 | 33.95 | 171.30 | 184.30 | 139.48 | 191.79 | 233.10 | 919.97 |
| 7.99 | 16.29 | 82.22 | 89.18 | 67.56 | 92.99 | 153.85 | 485.80 |
| 9.13 | 8.15 | 41.11 | 42.81 | 32.69 | 45.04 | 74.59 | 236.24 |
| 10.27 | 4.07 | 20.56 | 21.40 | 15.69 | 21.79 | 36.13 | 115.58 |
| 11.41 | 1.81 | 9.14 | 10.70 | 7.85 | 10.46 | 17.48 | 55.63 |
| 12.56 | 0.00 | 0.00 | 4.76 | 3.92 | 5.23 | 8.39 | 22.30 |
| 13.70 | | | 0.00 | 1.74 | 2.62 | 4.20 | 8.55 |
| 14.84 | | | | 0.00 | 1.16 | 2.10 | 3.26 |
| 15.98 | | | | | 0.00 | 0.93 | 0.93 |
| 17.12 | | | | | | 0.00 | 0.00 |

Table 3.11: Peak runoff hydrograph for 50yr return period

| **Time** (hr) | **UH ordinate Un ($m^3/s$)** | **$P_1U_n$** | **$P_2U_n$** | **$P_3U_n$** | **$P_4U_n$** | **$P_5U_n$** | **Storm Hydrograph $Q_n$ ($m^3/s$)** |
|---|---|---|---|---|---|---|---|
| 0.00 | 0.00 | 0.00 | | | | | 0.00 |
| 1.14 | 194.63 | 1123.79 | 0.00 | | | | 1123.79 |
| 2.28 | 452.63 | 2613.47 | 565.79 | 0.00 | | | 3179.26 |
| 3.42 | 298.73 | 1724.89 | 1315.79 | 206.70 | 0.00 | | 3247.37 |
| 4.57 | 144.84 | 836.31 | 868.42 | 480.69 | 137.60 | 0.00 | 2323.02 |
| 5.71 | 70.16 | 405.09 | 421.05 | 317.26 | 320.01 | 110.16 | 1573.56 |
| 6.85 | 33.95 | 196.01 | 203.95 | 153.82 | 211.20 | 256.19 | 1021.17 |
| 7.99 | 16.29 | 94.08 | 98.68 | 74.51 | 102.40 | 169.08 | 538.76 |
| 9.13 | 8.15 | 47.04 | 47.37 | 36.05 | 49.60 | 81.98 | 262.04 |
| 10.27 | 4.07 | 23.52 | 23.68 | 17.30 | 24.00 | 39.71 | 128.22 |
| 11.41 | 1.81 | 10.45 | 11.84 | 8.65 | 11.52 | 19.21 | 61.68 |
| 12.56 | 0.00 | 0.00 | 5.26 | 4.33 | 5.76 | 9.22 | 24.57 |
| 13.70 | | | 0.00 | 1.92 | 2.88 | 4.61 | 9.41 |
| 14.84 | | | | 0.00 | 1.28 | 2.31 | 3.59 |
| 15.98 | | | | | 0.00 | 1.02 | 1.02 |
| 17.12 | | | | | | 0.00 | 0.00 |



Table 3.12: Peak runoff hydrograph for 75yr return period

| **Time** (hr) | **UH ordinate Un ($m^3/s$)** | **$P_1U_n$** | **$P_2U_n$** | **$P_3U_n$** | **$P_4U_n$** | **$P_5U_n$** | **Storm Hydrograph $Q_n$ ($m^3/s$)** |
|---|---|---|---|---|---|---|---|
| 0.00 | 0.00 | 0.00 | | | | | 0.00 |
| 1.14 | 194.63 | 1206.32 | 0.00 | | | | 1206.32 |
| 2.28 | 452.63 | 2805.38 | 597.12 | 0.00 | | | 3402.51 |
| 3.42 | 298.73 | 1851.55 | 1388.66 | 217.60 | 0.00 | | 3457.81 |
| 4.57 | 144.84 | 897.72 | 916.52 | 506.04 | 144.61 | 0.00 | 2464.89 |
| 5.71 | 70.16 | 434.83 | 444.37 | 333.98 | 336.30 | 116.00 | 1665.49 |
| 6.85 | 33.95 | 210.40 | 215.24 | 161.93 | 221.96 | 269.77 | 1079.30 |
| 7.99 | 16.29 | 100.99 | 104.15 | 78.44 | 107.62 | 178.05 | 569.24 |
| 9.13 | 8.15 | 50.50 | 49.99 | 37.95 | 52.13 | 86.33 | 276.89 |
| 10.27 | 4.07 | 25.25 | 25.00 | 18.22 | 25.22 | 41.81 | 135.50 |
| 11.41 | 1.81 | 11.22 | 12.50 | 9.11 | 12.11 | 20.23 | 65.17 |
| 12.56 | 0.00 | 0.00 | 5.55 | 4.55 | 6.05 | 9.71 | 25.87 |
| 13.70 | | | 0.00 | 2.02 | 3.03 | 4.86 | 9.91 |
| 14.84 | | | | 0.00 | 1.35 | 2.43 | 3.77 |
| 15.98 | | | | | 0.00 | 1.08 | 1.08 |
| 17.12 | | | | | | 0.00 | 0.00 |

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Table 3.13: Peak runoff hydrograph for 100yr return period

| **Time** (hr) | **UH ordinate Un ($m^3/s$)** | **$P_1U_n$** | **$P_2U_n$** | **$P_3U_n$** | **$P_4U_n$** | **$P_5U_n$** | **Storm Hydrograph $Q_n$ ($m^3/s$)** |
|---|---|---|---|---|---|---|---|
| 0.00 | 0.00 | 0.00 | | | | | 0.00 |
| 1.14 | 194.63 | 1268.79 | 0.00 | | | | 1268.79 |
| 2.28 | 452.63 | 2950.68 | 620.48 | 0.00 | | | 3571.16 |
| 3.42 | 298.73 | 1947.45 | 1442.98 | 225.77 | 0.00 | | 3616.19 |
| 4.57 | 144.84 | 944.22 | 952.36 | 525.05 | 150.06 | 0.00 | 2571.69 |
| 5.71 | 70.16 | 457.35 | 461.75 | 346.53 | 348.98 | 120.28 | 1734.90 |
| 6.85 | 33.95 | 221.30 | 223.66 | 168.02 | 230.32 | 279.72 | 1123.02 |
| 7.99 | 16.29 | 106.22 | 108.22 | 81.38 | 111.67 | 184.62 | 592.12 |
| 9.13 | 8.15 | 53.11 | 51.95 | 39.38 | 54.09 | 89.51 | 288.04 |
| 10.27 | 4.07 | 26.56 | 25.97 | 18.90 | 26.17 | 43.36 | 140.96 |
| 11.41 | 1.81 | 11.80 | 12.99 | 9.45 | 12.56 | 20.98 | 67.78 |
| 12.56 | 0.00 | 0.00 | 5.77 | 4.73 | 6.28 | 10.07 | 26.85 |
| 13.70 | 0.00 | 0.00 | 0.00 | 2.10 | 3.14 | 5.04 | 10.28 |
| 14.84 | 0.00 | 0.00 | 0.00 | 0.00 | 1.40 | 2.52 | 3.91 |
| 15.98 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 1.12 | 1.12 |
| 17.12 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |



Table 3.14: Peak storm runoff hydrograph for various return period

| Time (hr) | Synthetic UH ordinate ($m^3/s/cm$) | Peak Storm runoff hydrographs ($m^3/s$) | | | |
|---|---|---|---|---|---|
| | | 25-yr,24-hr | 50-yr,24-hr | 75-yr,24-hr | 100-yr,24-hr |
| 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 1.14 | 194.63 | 982.10 | 1123.79 | 1206.32 | 1268.79 |
| 2.28 | 452.63 | 2795.25 | 3179.26 | 3402.51 | 3571.16 |
| 3.42 | 298.73 | 2883.89 | 3247.37 | 3457.81 | 3616.19 |
| 4.57 | 144.84 | 2076.47 | 2323.02 | 2464.89 | 2571.69 |
| 5.71 | 70.16 | 1413.01 | 1573.56 | 1665.49 | 1734.90 |
| 6.85 | 33.95 | 919.97 | 1021.17 | 1079.30 | 1123.02 |
| 7.99 | 16.29 | 485.80 | 538.76 | 569.24 | 592.12 |
| 9.13 | 8.15 | 236.24 | 262.04 | 276.89 | 288.04 |
| 10.27 | 4.07 | 115.58 | 128.22 | 135.50 | 140.96 |
| 11.41 | 1.81 | 55.63 | 61.68 | 65.17 | 67.78 |
| 12.56 | 0.00 | 22.30 | 24.57 | 25.87 | 26.85 |
| 13.70 | | 8.55 | 9.41 | 9.91 | 10.28 |
| 14.84 | | 3.26 | 3.59 | 3.77 | 3.91 |
| 15.98 | | 0.93 | 1.02 | 1.08 | 1.12 |
| 17.12 | | 0.00 | 0.00 | 0.00 | 0.00 |
| Max | | 2883.89 | 3247.37 | 3457.81 | 3616.19 |
| Mean | | 749.94 | 843.59 | 897.73 | 938.55 |

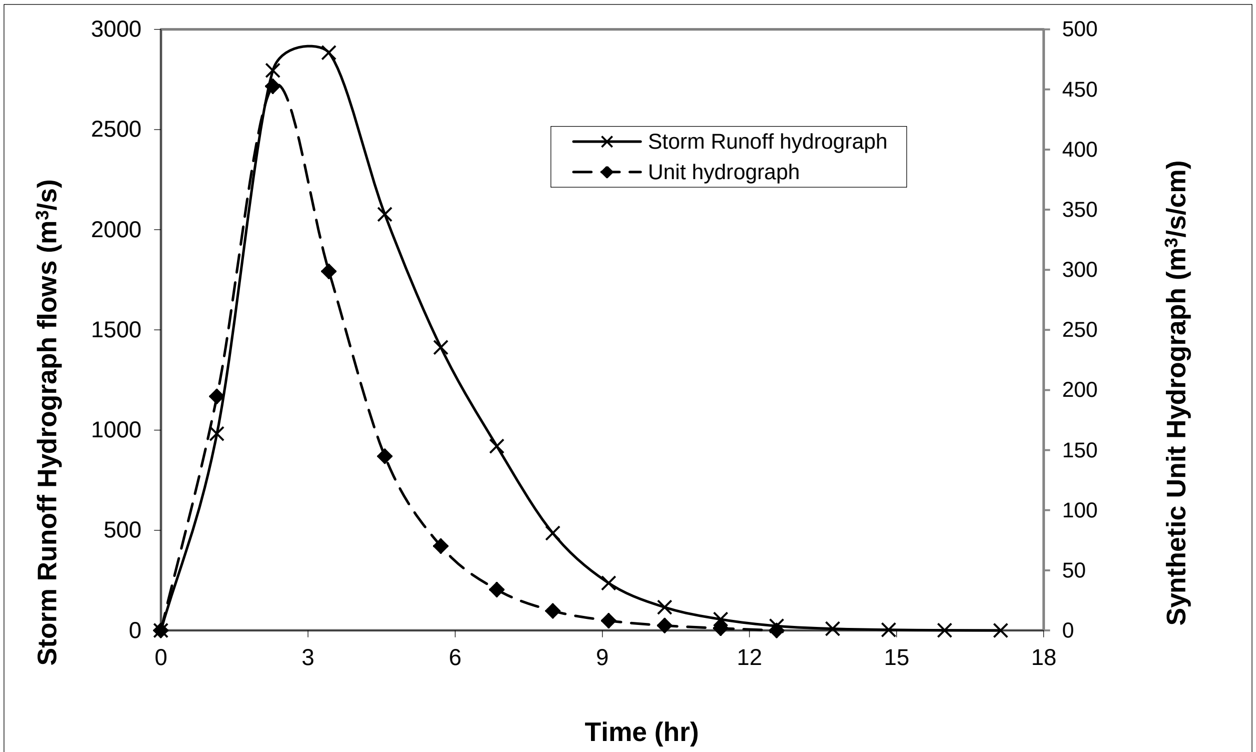


Figure 3.3 Synthetic Unit and 25-yr, 24-hr Storm Runoff Hydrographs

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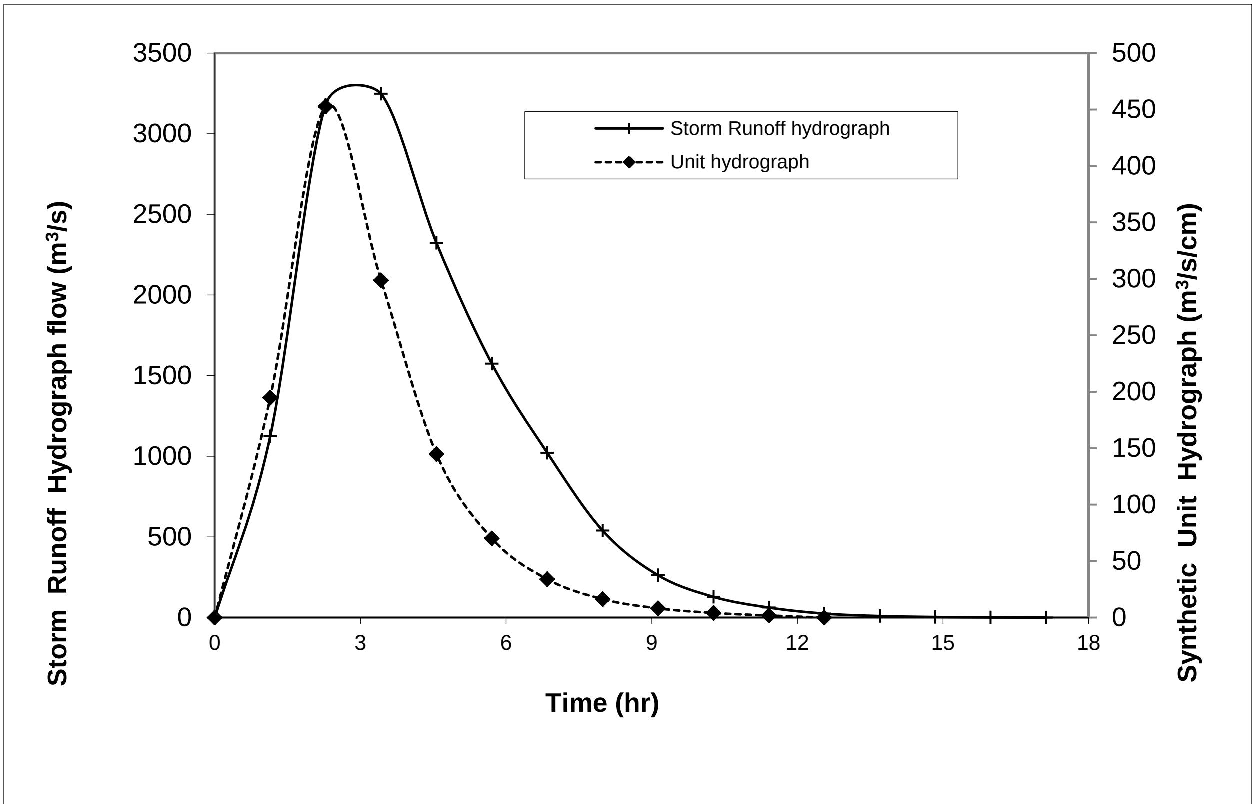


Figure 3.4 Synthetic Unit and 50-yr, 24-hr Storm Runoff Hydrographs

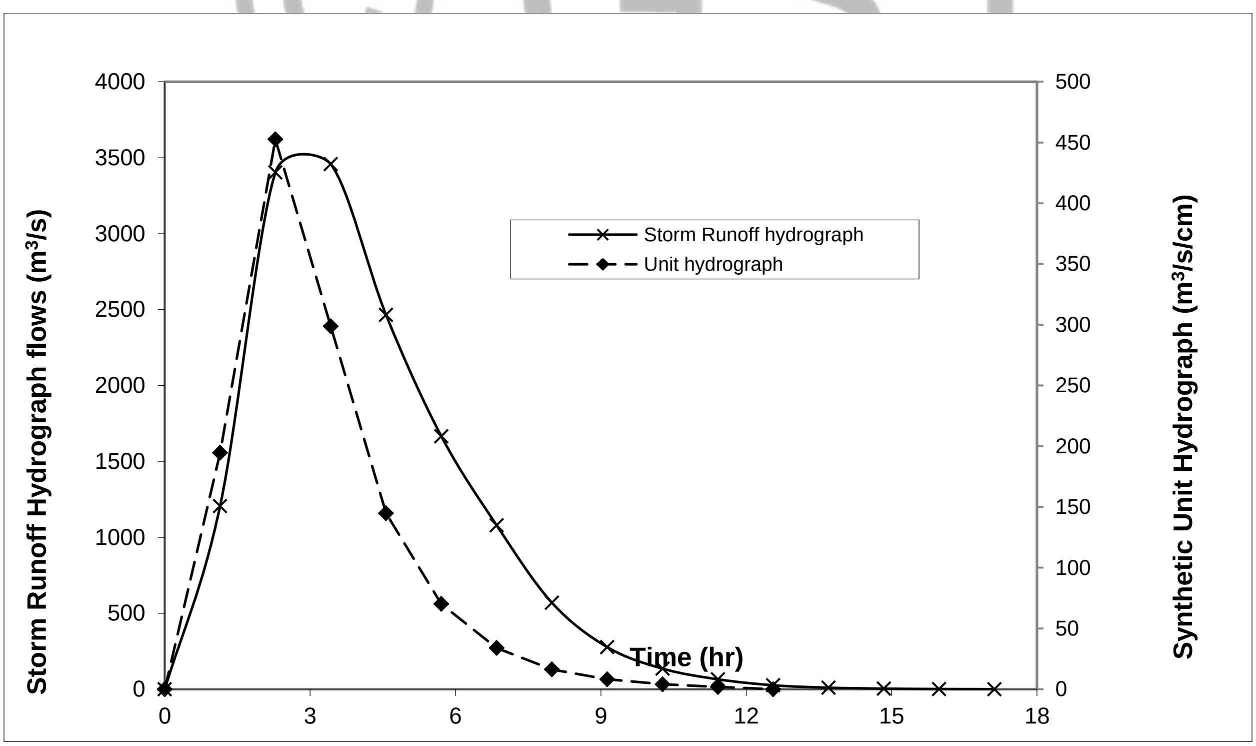


Figure 3.5 Synthetic Unit and 75-yr, 24-hr Storm Runoff Hydrographs

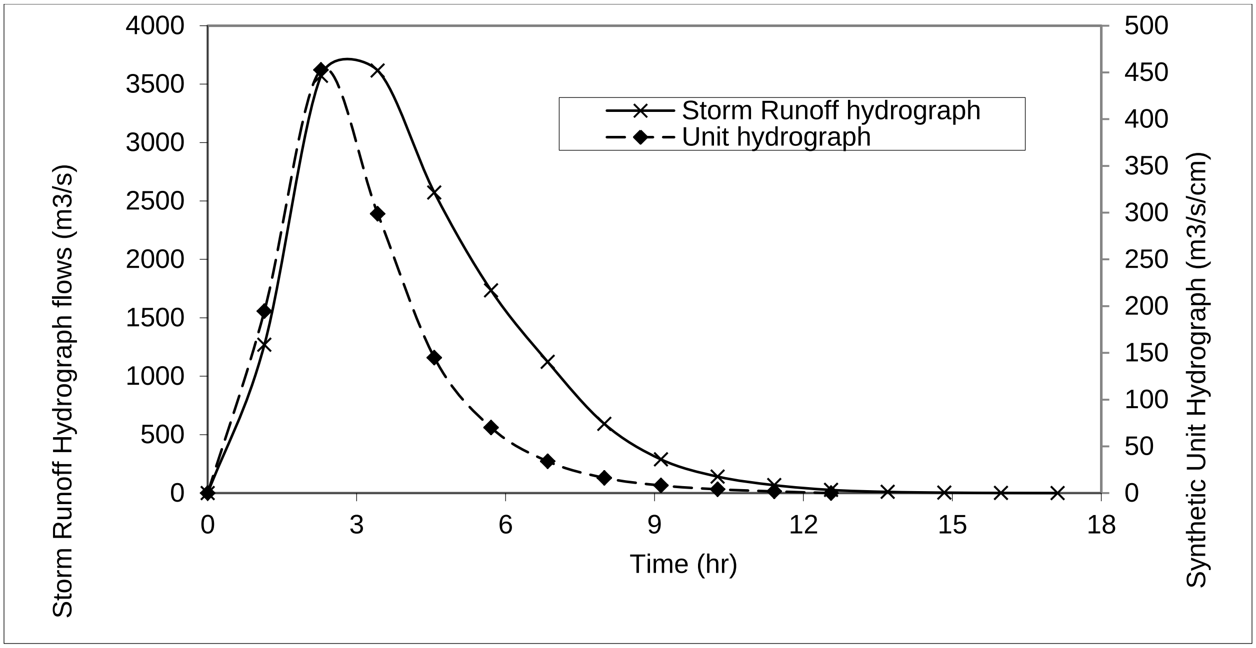


Figure 3.6 Synthetic Unit and 100-yr, 24-hr Storm Runoff Hydrographs

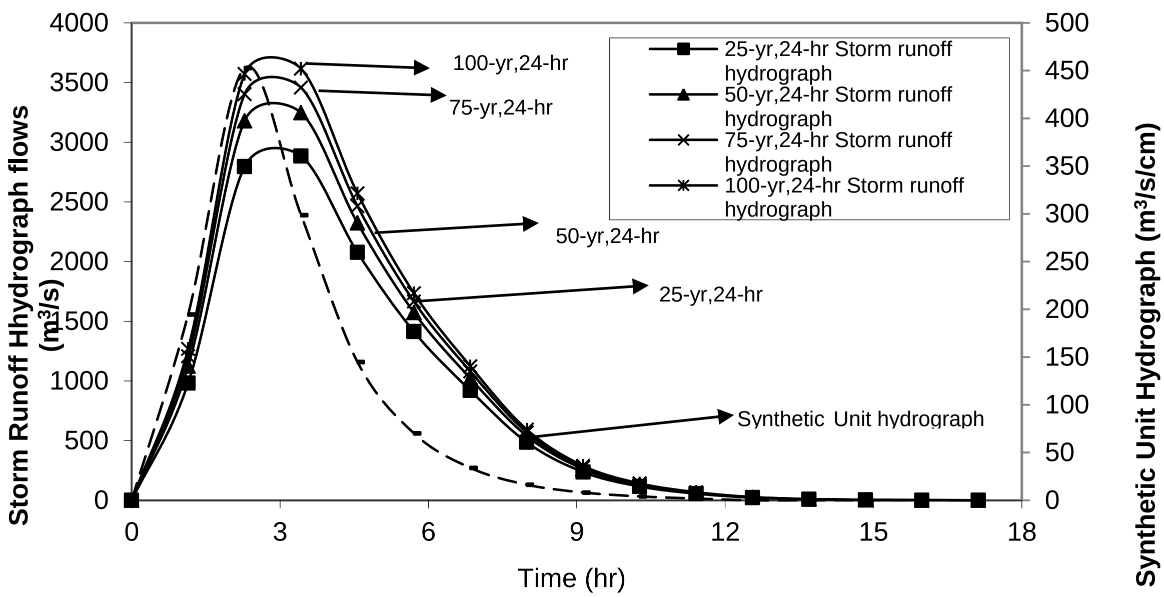


Figure 3.7 Synthetic Unit hydrograph and storm hydrographs of different return periods

## 4.0 Results and Discussion

### 4.1 Peak runoff

The summary for the parameters for generating unit hydrograph was presented in Table 3.3. while the unit hydrograph ordinates for the catchment was presented in Table 3.4. The unit hydrograph for the hydrological area is depicted in figure 3.2. The rainfall excess for four different return periods were presented in Tables 3.6 -3.9. The peak runoff hydrograph for the catchment for 25yr, 50yr, 75yr, and 100yr recurrence interval were presented in Tables 3.10-3.14.

**a) Storm Runoff Hydrographs**

The time versus synthetic unit hydrograph ordinates for the return periods of 25yr, 50yr, 75yr, and 100yr were plotted to determine their peak runoff. The graphs are presented in figures 3.3-3.6 accordingly.

**b) Result of Comparison of the peak runoff of 25yr, 50yr, 75yr, and 100yr**

The synthetic Unit hydrograph and storm hydrograph of different return periods was determined and presented in figure 3.7.

## 4.3 Flood Design discharge

The design of any hydraulic structure is based on hydrologic events which are random in nature due to the uncertainties in their occurrences. The obtained watershed attributes was used with the synthetic unit hydrograph adopted to determine the peak runoff for the various return periods of 25yr, 50yr, 75yr, and 100yr for the planning or designing hydraulic structures and forecast of future events with some degree of accuracy. The peak runoff obtained varied from 2883.89$m^3/s$ to 3616.19$m^3/s$ for the catchment as presented in Table 3.10-3.14. The results are reliable because the method adopted used morphometric parameters such as the catchment area, length, slope, excess rainfall, and curve number for the determination of the peak runoff.

## 5.0 Conclusion

The analyzed watershed attribute parameters were used in forecasting the peak runoff for return periods of 25yr, 50yr, 75yr, and 100yr using the SCS methods. And from the determined hydrograph the return periods of 25yr has the lowest peak runoff with 2883.89$m^3/s$ while the 100yr return period has the highest peak runoff of 3616.19$m^3/s$ respectively. The quantitative analysis of morphometric parameters of the watershed were also found to be of immense utility in the development of the management scenario for the basin.

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