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FLOOD PREDICTION USING RAINFALL – RUNOFF SPATIAL VARIATION: AN OVERVIEW OF FLOOD PREDICTION MODELS

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Abstract
High intensity rainfall and associated floods have become frequent in most cities and urban areas in recent years, within the lower reaches of the Niger Delta. The magnitude and time variation of rainfall and associated runoff has proved more difficult to predict. This is mainly as a result of the inherent stochastic nature of such events. The need for a systematic approach to flood forecasting based on rainfall is of the essence. This work presents rainfall data obtained for an urban city located within the Niger Delta proximal location to Port Harcourt, Rivers State, with a corresponding time series analysis of the data as the basic input information. The associated runoff data will be simulated to investigate runoff related events and the spatial variation of flow directions within the catchment under study. The review of selected appropriate mathematical models and graphical comparisons between quantities of rainfalls observed form the theoretical frame for an effective flood prediction procedure. The considered time-series analysis techniques, and especially those based on the use of Artificial Neural Network (ANN), provide a significant improvement in the flood forecasting accuracy in comparison to the use of simple prediction approaches, which are often applied in hydrological practice.

Key Words: *Rainfall, Flood, Runoff data, Time series, Spatial*

Introduction

Hydrologist are often faced with the challenge of predicting the peak discharge, time to peak and magnitude of rainfall generated runoffs for watersheds for the purposes of appropriate design and management of flood control and ecological issues. The deterministic characteristics of the dynamics of the hydrological cycle are fairly well understood. The magnitude and time variation of rainfall has proved more difficult to predict mainly as a result of the inherent stochastic nature of such events. Extreme rainfall events are difficult to predict. Associated runoffs from these events are thus expected to be as difficult to predict. Similarly, the prediction of floods is not an easy exercise as factors that cause floods can be due to natural or man-made activities. In the design and application of rainfall-runoff phenomena, model inputs should accurately represent environmental properties. Most inputs for environmental models involve spatial variations, and particularly hydrological models are sensitive to the spatial variation of rainfall events (Lopes, 1996; Chaplot, 2005).

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Records of rainfall and associated runoff data over long periods of time (e.g. ten years, fifty years, one hundred years etc.) are also difficult to obtain in the lower reaches of the Niger-Delta basin where methodology for obtaining these records have only been recently put in place by agencies such as the Nigeria Meteorological Agency, Federal Inland Water Ways Authority, and similar governmental bodies.

Though these information are required for a wide variety of application, many streams and non-tidal waters in the Niger Delta are ungauged and do not have flow records. One way of solving the problem of non-availability of flow records is by resorting to the use of rainfall-runoff models.

Study Area

The urban city of Port Harcourt is the capital of Rivers State located within the lower reaches of the Niger Delta basin. The city is situated about 66km from the Atlantic Ocean on the Bonny River and approximately located on Longitude 07° 10′4″E and latitude 04° 47′0″N. The study area (catchment area) is situated in the transitional belt with generally flat topography and strong tidal influence that affects the drainage efficiency of the area. The various geomorphological zones for the Port Harcourt and proximal areas are shown on Table 1 and figure 1 (Bell-Gam, 2002)

Table1: Geomorphological Zones of Rivers State

S/N	Geomorphological /Geotechnical Zones	Spatial Extent in Terms of Local Government Area of Rivers State
1	Saltwater (Marine) Coastal Zone	Akuku-Toru, Andoni, Asari-Toru, Bonny, Degema, Okirika, Ogu-Bolo and Opobo-Nkoro
2	Saltwater/Freshwater transitional Zone	Abua-Odual, Obio-Akpor, Port Harcourt, Oyigbo, Tai, Eleme, Emohua, Khana, Gokana
3	Freshwater Upland Zone	Ahoada-East, Ahoada-West, Etche, Gokana, Ikwerre, Ogba/Egbema/Ndoni, Omuma, Oyigbo

Figure 1: Map of the Study Area

Methodology

We shall in this work present rainfall data obtained for a proximal location to Port Harcourt. We shall also attempt to carry out a time series analysis of this data. Associated runoff from these events will be simulated to investigate runoff related events for the catchment under study.

The maximum monthly rainfall data is presented in table 2. The data covers the period from 1995 to 2009. Figure 3.1 shows a graphical plot of this data that covers the mean monthly rainfall from 1995 to 2009. Figures 3.2, 3.3 and 3.4 show the monthly plots of the rainfall data series (the mean monthly data set and variations from the mean) (Oyegun and Ologunorisa, 2002)

Table 2: Mean Monthly Maximum Rainfall (mm) in the Study Area

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1995	0	80.9	105	108	432	257	360	310	139	172	111	20
1996	61.1	0	178	89.9	179	170	389	271	260	272	87.9	14
1997	0	43	158	248	139	327	439	426	369	267	147	21
1998	37.4	66.8	80.9	129	373	256	462	314	342	219	93.9	0
1999	79.6	15.6	118	120	363	247	394	334	320	413	72.9	53
2000	0	130	114	321	374	161	241	229	478	273	22.8	5.9
2001	23.3	18.6	96.4	175	380	353	360	306	207	133	248	30
2002	22.6	36.9	87.6	188	279	415	370	247	489	265	137	32
2003	40.9	52.1	107	186	292	233	294	257	454	511	73.5	0
2004	11.6	7.2	59.2	190	202	182	420	245	455	153	51.6	17
2005	31.3	2.4	156	118	315	245	337	310	365	137	108	28
2006	0	79	75.8	103	118	324	285	557	265	284	67	28
2007	17.3	86.2	93	169	174	255	481	206	535	240	98.6	4.1
2008	1.8	46.5	50.7	121	133	244	400	210	352	218	95.7	5.4
2009	17.1	85.9	171	121	247	383	254	229	284	195	28.7	39
Mean	22.93	50.1	110	159.1	266.5	270	365.7	296.8	354.3	250	96.21	19.8

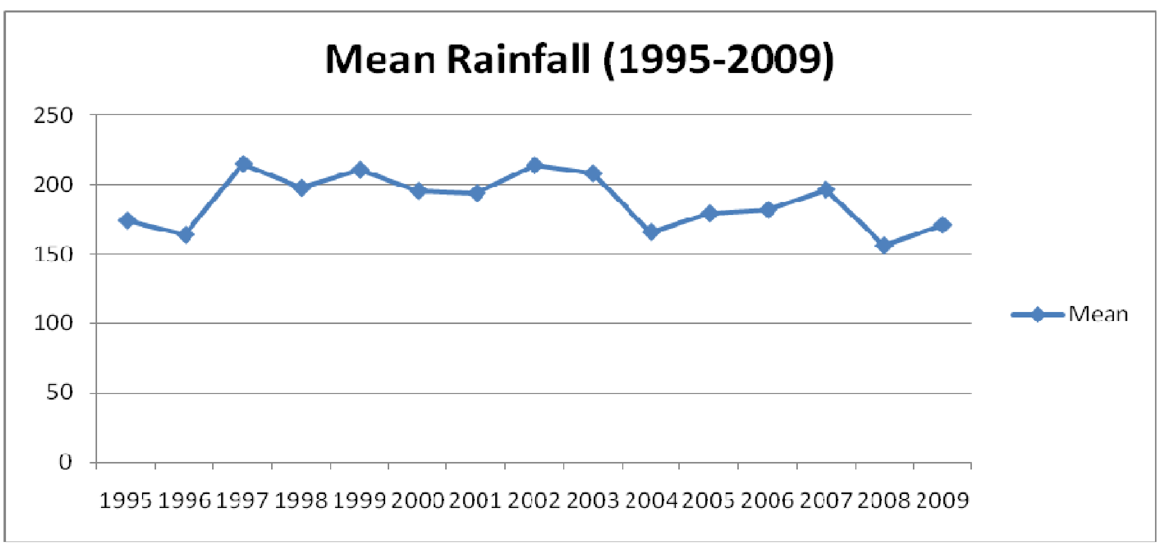

Figure 3.1: Plot of Mean Monthly Rainfall (1995-2009)

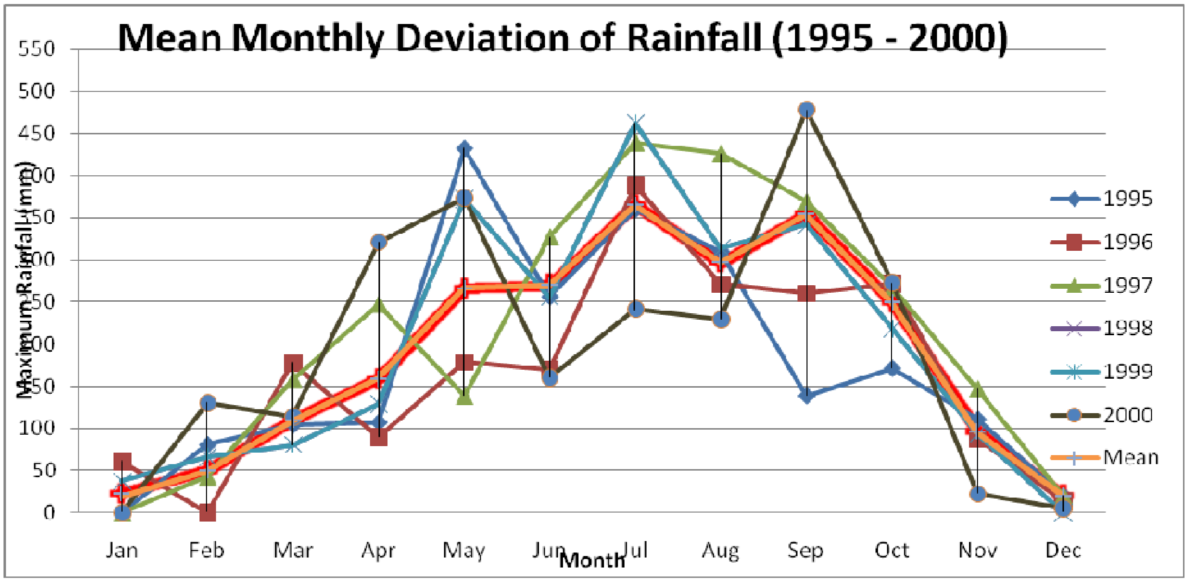

Figure 3.2: Mean Monthly Rainfall Data and Variation from Mean Value (1995-2000)

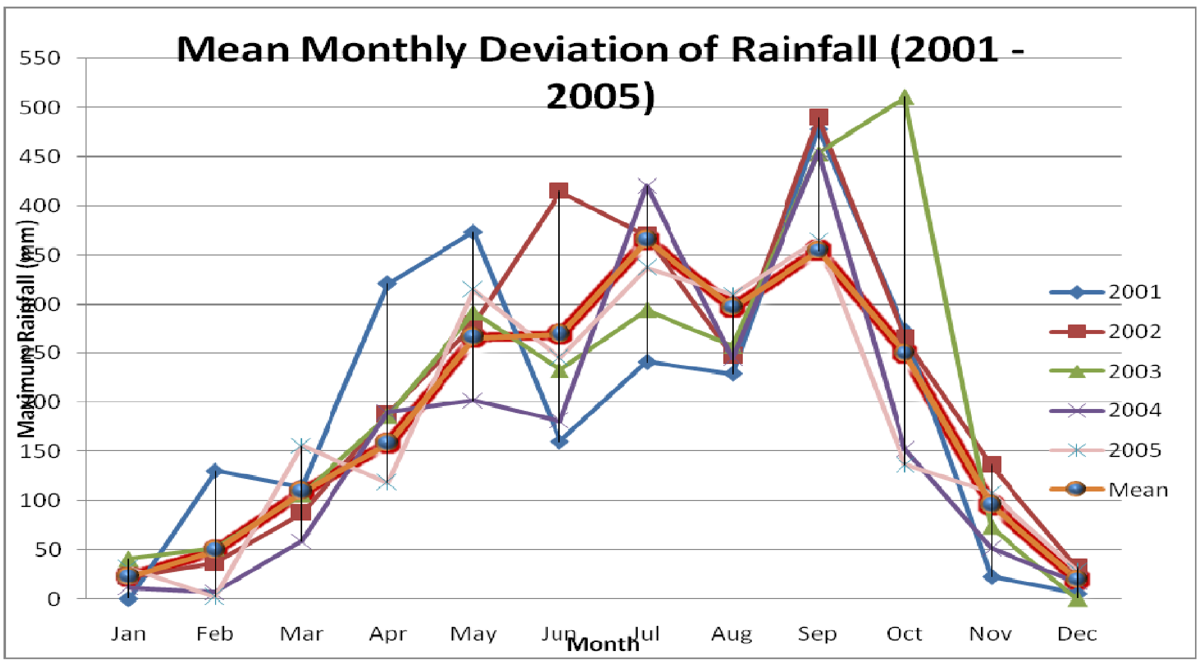

Figure 3.3: Mean Monthly Rainfall Data and Variation from Mean Value (2001-2005)

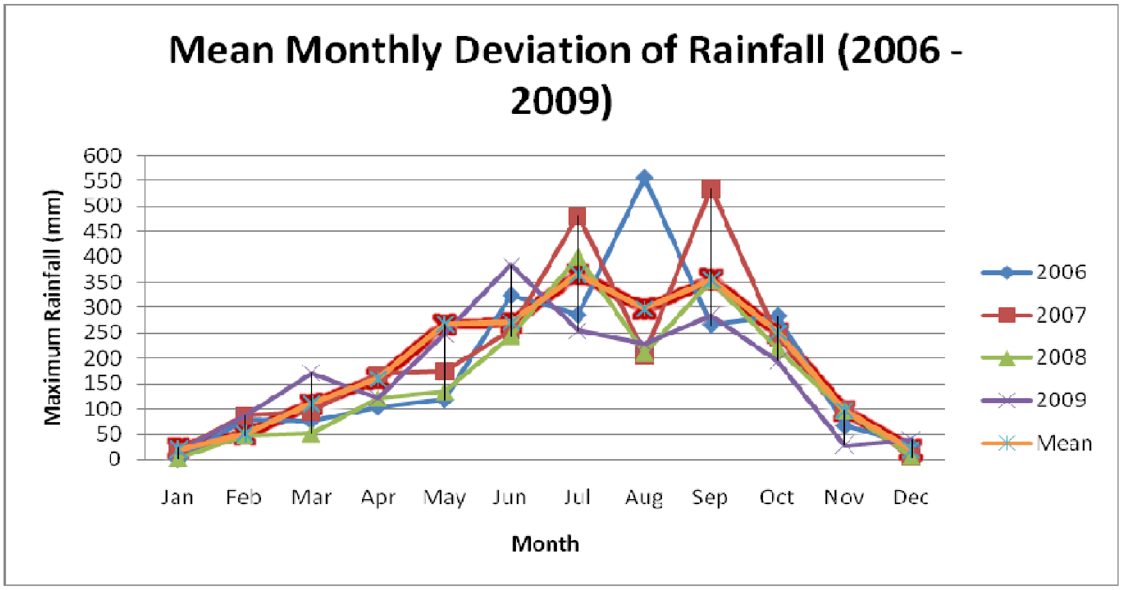

Figure 3.4: Mean Monthly Rainfall Data and Variation from Mean Value (2006-2009)

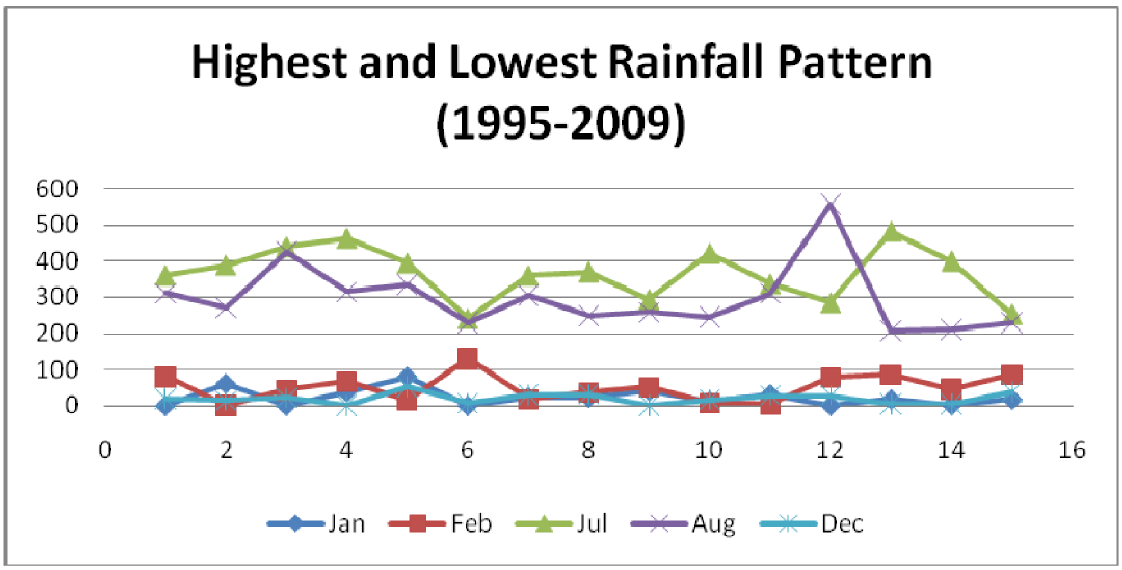

Figure 3.5: Highest and Lowest Rainfall Pattern (1995-2009)

3.1 Observations on Data Set

It can be deduced from table 2.1 and figure 3.2 that the quantity of rainfall for 1997 was the highest (478mm) in the month of September and the minimum rainfall was in January, February and December of the period under review. The mean highest rainfall for the period is (350mm) and this was experienced in the month of July. From figure 3.3 the quantity of rainfall for 2003 was the highest (525mm) in the month of October and the minimum rainfall was in January, February and December of the period under review. The mean highest rainfall for the period is (360mm) and this was experienced in the month of July.

Also, from figure 3.4 the quantity of rainfall for 2006 was the highest (560mm) in the month of August and the minimum rainfall was in January, and December of the period under review. The mean highest rainfall for the period is (360mm) and this was experienced in the month of July.

Generally, from figure3.5, the quantity of rainfall in the month of July was the highest throughout the period under review. The lowest rainfall period remains in January and December.

3.2 Hydrologic Time Series Data

Hydrological data observed over discrete or continuous time intervals may be analysed using the methodologies applicable to time series data. Section 3.1 and associated plots of section 3.0 has presented the discrete time series data in graphical format. From such presentation, it may be feasible to discern pictorially the discrete data set as a continuous series of hydrological data observation. Thus, issues such as maximum rainfall months, yearly rainfall data and associated mean monthly data are easily discernible. A more rigorous treatment of the data set obtained will require a modelling technique for the hydrological time series obtained. Components of a given time series Y(t) can be resolved as follows:

$$Y(t) = y_1(t) + y_2(t) + y_3(t) + y_4(t) \quad .... (1.0)$$

Where,

Y_1 (t) represents the trend in the given data
Y_2 (t) represents the periodic components
Y_3 (t) represents extreme value components
Y_4 (t) represents the stochastic component

While Y_1 (t), Y_2(t) and Y_4(t) can be predicted and modelled with relative ease, Y_3 (t) the extreme value component of the time series, represents catastrophic events which presents difficulties with modelling techniques as they do not adhere to any recognisable pattern. Best attempts at modelling such events will require the careful observation and modelling of time series data for long periods of time. Such data sets, (e.g. for rainfall data for over 30 – 100 years) are not readily available in the developing parts of the world. The occurrence of unpredicted flood events are largely the outcome of such extreme rainfall events.

However, conversion models are available for translating the rainfall events to the runoff event in hydrological modelling. In other words, given some rainfall data series Y(t) for a given catchment with defined physical hydrological characteristics e.g. catchment boundaries, catchment area, evapo-transpiration characteristics, infiltration characteristics etc., it is possible to model the runoff (Q(t)).

Historical hydrological variables e.g. rainfall-runoff characteristics over extended periods of time can provide an insight into the pattern of occurrence of extreme events. The long sequence of data with similar hydrological characteristics (e.g. flood events) can also provide the frequency or return period for the infrequent notable events. Historic time series data simply provide a number of such events in the longer combined data.

This technique has been applied around the Lake Chad Basin (Maiduguri Catchment) where rainfall data was assembled over an extended period of over 60 years (Shaw, 1989). A return period of 30 years was discovered for the time series data.

However, it has been difficult to obtain similar data for river flow (run-off). When such data become available, stochastic models such as the AUTO-Regressive Moving Average (ARMA) (Chatfield, 1980) can be applied to gain an insight into river modelling with a view to ascertaining return periods for flood events.

3.3 Estimation of Runoff

One method for the physical estimation of runoff values consists of the production of rating curves for runoff derived from water level observations at some stable section along the drainage channel.

Figure 1 below shows a cross section of such channel.

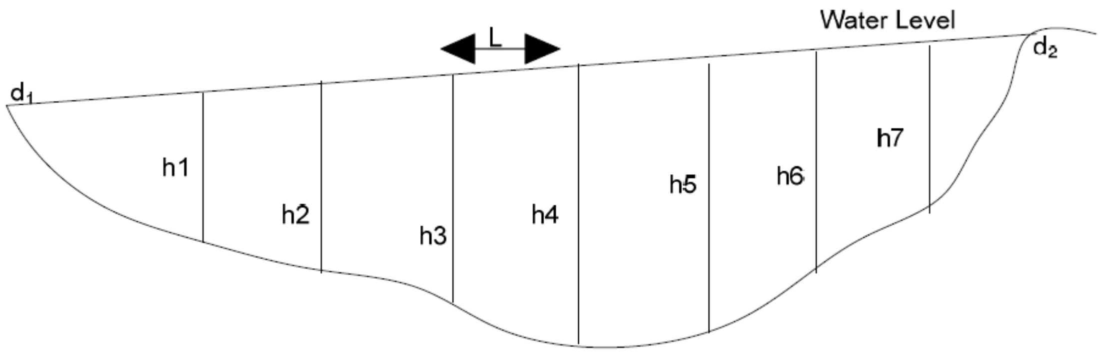

Figure 3.6: Cross Section of a Drainage Channel

Let Q be the flow through this stable section, A the area of the cross and the velocity of fluid flow through this section be V. (Ojinaka, 2007).

Then

$Q = AV$ ….. (2.0)

Physical methods of obtaining the value of A may be by the use of the trapezoidal formula for obtaining areas of cross-sections. The values of the bottom profile may be obtained by the use of echo-soundings techniques carried out for the x – section. The area of the cross section can be deduced from the h_i values as (Uren and Price, 2006).

$$A=\frac{L}{2}(h_1+h_n)+2(h_2+h_3+------h_{n-1}) \quad \text{…. (3.0)}$$

Where L is the interval between soundings (assumed to be equally spaced).

3.4 Determination of Velocities

The observation of water velocity values could be obtained directly by the use of current meters in open channels. There are various designs of this equipment that operate basically by a rotating cup or propeller mechanism emplaced normal to the direction of flow. Read-out mechanisms on these equipment provide directly velocity values along the channel in the direction of flow. (Chadwick *et al*., 2004). With the values of A and V determined as in equations 2 and 3, the discharge through the channel is computed.

3.5 Stage–Discharge Relationship

For practical determination of Q values, it is more convenient to establish graduated staff (more or less a levelling staff) placed vertically at stable sections of a channel where A and V values have been predetermined and then relate the readings h of the graduated staff to the Q values.

Thus, various Q values are related with the readings h of the graduated staff. The most commonly employed technique for this methodology is to design a rating-curve. The rating curve is a graph of Q values plotted against h. Since the h values are more convenient to measure, Q values are determined directly from the relationships of the rating curve.

Hydraulic Techniques for Determination of Discharge

The hydraulic technique makes use of the Saint Venant equations for fluid flow in open channels. Where no "back-water" effects exist, the equation can be simplified to approximate kinematic waves where (see also equations 13.0 and 14.0 of section 3.6b).

$$S_f \approx S_0 \qquad \text{.... (4.0)}$$

Where S_f, $S_o$are referred to as friction slope and bed slope respectively. The discharge Q under such conditions can then be computed using the chezy equation (equation 5) or the manning's formula (equation 6):

$$Q = AV = AC\sqrt{RS_0} \qquad \text{.... (5.0)}$$

$$Q = AV = \frac{AR^{\frac{2}{3}} S_0^{\frac{1}{2}}}{n} \qquad \text{.... (6.0)}$$

C is the chazy's constant of equation 5 while n represents the Manning's formula. R is the hydraulic radius obtained through

$$R = \frac{A}{P} \qquad \text{.... (7.0)}$$

P is the wetted perimeter which can be determined by measuring the distance d_1 to d_2 along the river cross-sectional reach of the river bed, equations such as equation 5.0 and 6.0 can then be applied. It is also possible to model the Q (t) at given sections through the hydrological catchment up to the point of eventual outflow.

Thus, with the bed slope $S_o$determined along consecutive stable cross sections of the flow regime, the velocity and hence the discharge Q can be determined directly by assigning appropriate values to C in the Chezy's equation or n for the Manning's formula.

3.6 Modelling Q (t)

The process of modelling a flood event (usually downstream) from one point to another comes under the general principles of flood routing. It is convenient to visualize three (3) main routes of travel for rainfall from the time it reaches the ground until it enters a stream channel. The possible routes are overland flow, interflow and groundwater flow. The overland is the main component of runoff cycle and it is a dominant mechanism for flood prediction and soil erosion (Zhang, 1990).

Consider the curves of figures 3.7 and 3.8 where A represent the hydrograph observed or derived for section A of figure 3.7and O represent the hydrograph characteristics for a river downstream at section B.

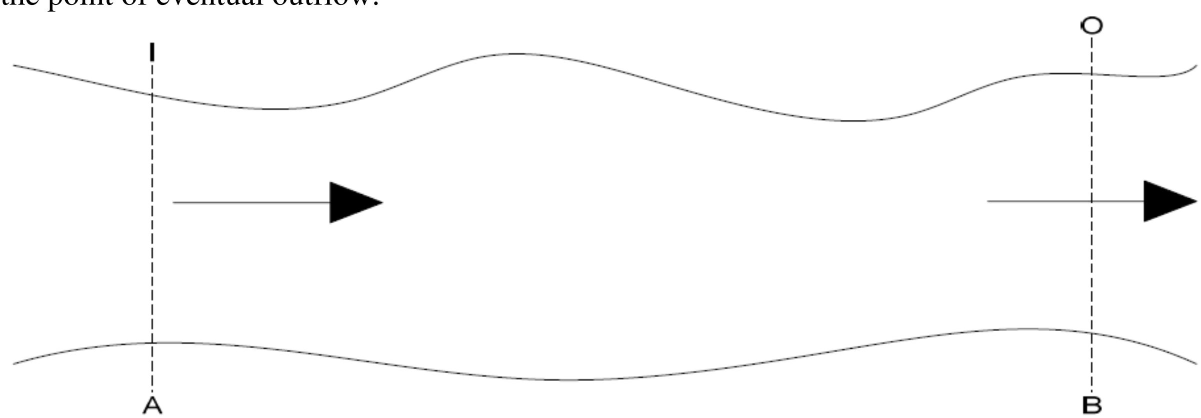

Figure 3.7: A Section of Hydrograph

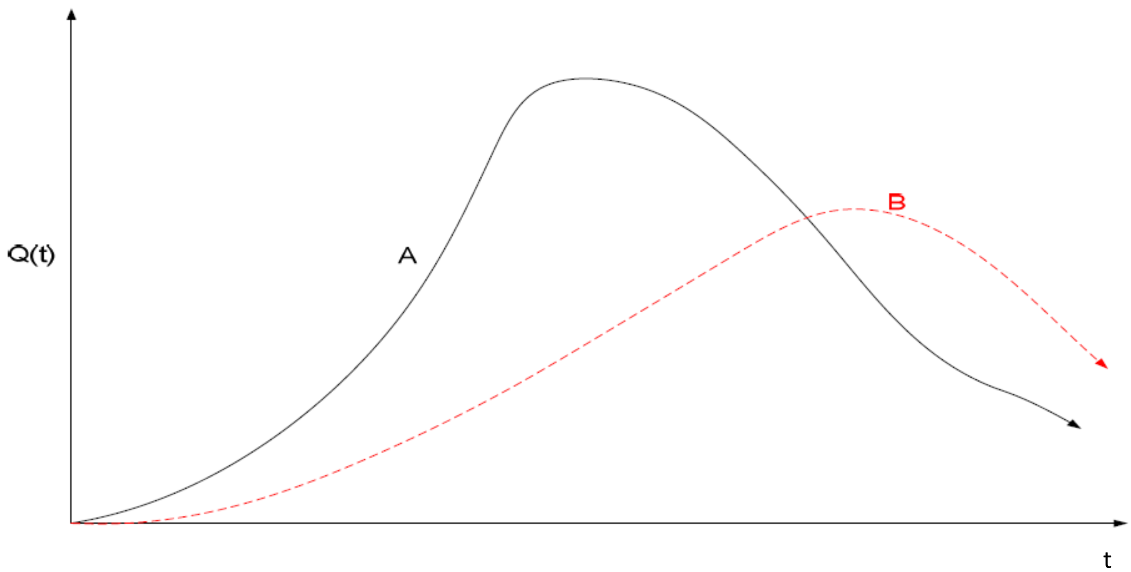

Figure 3.8: Hydrograph Characteristics

The basic principle of the continuity equation in hydrodynamics can be stated as shown in figures 3.7 and 3.8.

$$I - O = \frac{ds}{dt} \qquad \text{.... (8.0)}$$

In other words the hydrograph characteristics in terms of volume Q (t) values will not be the same along the sections 1 and 2 of figure 3.7. The storage S within the reach over the time interval dt affects the outflow at section B. The characteristics of the storage element (S) and as it relates to near-shore topographical characteristics may in some instances result to flooding catchment.

There are two commonly applied models for routing the Q (t) along the river reach.

a. The Muskingum Model

The governing equations for this model can be expressed as:

$$I - O = \frac{ds}{dt} \qquad \text{.... (9.0)}$$

and

$$S = f_1(O) + f_2(I - O) \qquad \text{.... (10.0)}$$

The model after simplification can be expressed as (Shaw, 1989)

$$O_2 = C_1 I_1 + C_2 I_2 - - - C_3 O_1 \qquad \text{.... (11.0)}$$

Where I_1 and I_2 denotes Q(t) values at times t_1 and t_2 at some upstream section and O_1 and O_2 represent corresponding outflow values at some outflow section of the river reach. C_1, C_2 and C_3 are constants whose values are given over a time interval ΔT.

$$C_1 = \frac{\Delta T + 2Kx}{\Delta T + 2k - 2kx} \qquad \text{.... (12a)}$$

$$C_2 = \frac{\Delta T - 2kx}{\Delta T + 2k - 2kx} \qquad \text{.... (12b)}$$

$$C_3 = \frac{\Delta T + 2k - 2kx}{\Delta T + 2k - 2kx} \qquad \text{.... (12c)}$$

K and x are also constants that apply to the Muskingum routing techniques (Sharma and Sharme 1999; Reddy, 2006).

b. The Hydraulic Routing

Again, the continuity equation applies here to the hydraulics of open channel flow. The equation of dynamics uses the momentum or energy flow equation for a relatively short length of channel. The resulting St Venant equation is:

$$S_f = S_0 - \frac{dy}{dx} - \frac{v}{g}\frac{dv}{dx} - \frac{1}{g}\frac{dv}{dt} \qquad \text{... (13.0)}$$

Given that there are no "back water" effects, equation 10 becomes (Townson, 1991)

$S_f \approx S_0$ …. (14.0)

Equations 5.0 and 6.0 of section 3.5 are derived from this principle.

Conclusion

A general mathematical modelling system for real-time flood forecasting and flood control planning is described. The system comprises a lumped conceptual rainfall-runoff model, a hydrodynamic model for river routing, and Muskingum method are reviewed as veritable models for efficient flood forecasting.

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