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KINEMATIC FLOOD ROUTING OF ASA RIVER, ILORIN, NIGERIA

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***Abstract*— Floods result from excessive runoff quantities and rates, which usually cause costly damage to properties and sometimes loss of lives. Flood routing is the spatio-temporal monitoring of flood wave as it traverses an area. Flood routing techniques are generally classified as hydrologic and hydraulic. Flood routing of Asa River was conducted using kinematic wave method, a hydraulic technique. The Asa River is an important river in Ilorin, the capital of Kwara State, Nigeria. The United States Soil Conservation Service (SCS) method was used in developing the synthetic unit hydrograph using the computed peak discharge of 2.44 m^3/s and time to peak of 0.213 hr. Kirpich's equation was used to compute the time of concentration of the watershed as 0.32 hr.**

***Index Terms*— Flood routing, Kinematic wave, Hydrograph**

I. INTRODUCTION

Flooding may result from the volume of the water within a body of water, such as a river or lake, exceeding the total capacity of its bounds, with the result that some of the water flows outside the catchment. It can also occur in rivers, when the strength of the river is so high it flows out of the river channel, particularly at bends or meanders (Khatibi, 2012).

The sudden change in the inflow due to occurrence of heavy rain in the upstream usually leads to the unsteadiness in a river as well as breaches in the embankment system. This may results to the collapse of a river valley structure (Sudarshan and Arup, 2013)

Framji and Garg (1976) defined flood as a relatively high flow in a river above normal, also the inundation of low land which may result from the high flow. Fread (1981) and Linsley *et al.*, (1989) defined flood routing as a mathematical method for predicting the changing magnitude and celerity of a flood wave as it propagates through rivers or reservoirs. Generally, flood routing methods are categorized into two broad, but somewhat related applications, namely reservoir routing and open channel routing (Lawler, 1964). These methods are frequently used to estimate inflow or outflow hydrographs and peak flow rates in reservoirs, river reaches, farm ponds, tanks, swamps and lakes (SCS, 1972; Viessman *et al.*, 1989; Smithers and Caldecott, 1995).

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Flood routing is an important technique used in the design of flood protection measures in order to estimate how the proposed measures will affect the behaviour of flood waves in rivers so that adequate protection and economic solutions can be found (Wilson, 1990). In practical applications, two steps are involved in the prediction and assessment of flood level inundation. To estimate the flow-gauging station to a downstream location, a flood routing model is employed. Then the flood hydrograph is input to a hydraulic model in order to estimate the flood levels at the downstream site (Blackburn and Hicks, 2001). Routing is used to predict the temporal and spatial variations of a flood wave as it traverses a river reach or reservoir, or it can be employed to predict the outflow hydrograph from a watershed subjected to a known amount of precipitation (Viessman *et al.*, 1989).

Flood-routing procedures may be classified as either hydrological or hydraulic (Choudhury *et al.*, 2002). Hydrological methods use the principle of continuity and a relationship between discharge and the temporary storage of excess volumes of water during the flood period (Shaw, 1994) while hydraulic methods of routing on the other hand use both the equation of continuity and equation of motion, customarily the momentum equation (Michael and Ojha, 2006). This particular form utilizes the partial differential equations for unsteady flow in open channels. It adequately describes the dynamics of flow more than does the hydrologic routing techniques.

Hjelmfelt (1984) stated that kinematic wave equation can be expressed in the form of a convolution integral. The kernel of this convolution integral can then be compared to a unit hydrograph. Kinematic wave equations are non-linear and hyperbolic; their exact solution is found using the method of characteristics (Wang and Tartakovsky, 2012).

Field (1982) proposed using the method of characteristics to solve the kinematic wave equation to produce the discharge hydrograph. In his work, he demonstrated that the model is a generalization of several other popular models and typical values of some catchment parameters were suggested for their application to design.

Numerical technique for solving the kinematic wave equation requires the initial values for the entire domain of the distance travelled by the flood as well as the upstream boundary conditions for the time of travelled. At each of the grid point, finite difference method can be adopted to obtain the solutions of the kinematic wave equation (Hossain *et al.*, 2013). The development of the kinematic wave flow routing equation and certain of the fundamental conditions necessary for its successful application are discussed by Smith (1980).

The objectives of this research work were to route an inflow hydrograph through a given river reach and obtain its outflow hydrograph and also to determine the amount of attenuation in hydrographs between the inflow and outflow at the end of a routing reach using kinematic wave method.

II. METHODOLOGY

A. *The Study Area*

Asa River has its source from Oyo State, Nigeria and it flows through Ilorin, Kwara State, Nigeria in a South-North direction forming a dividing boundary between Eastern and Western Ilorin. It is about 56 km long, with a maximum width of about 100 m (at the dam site). Asa River has its estuary at River Awon, which is one of the tributaries of River Niger, at 12,200 m North of Ilorin. It is joined by River Oyun to the East and to the West by River Imoru. Afidikodi, Ekoro, Obe are among the earliest tributaries of Asa River while its tributaries in Ilorin include River Agba, Aluko, Atikeke, Mitile, Odota, Okun, and Osere. (Ojo, 1998; Ibrahim *et al.*, 2013).

The Asa catchment in Ilorin, North Central Nigeria is located between latitudes $8^0$36′N and $8^0$24′N and longitudes $4^0$36′E and $4^0$10′E. Its total catchment is 1037 km^2 at the confluence and lies within Kwara State and Oyo State, in Nigeria with about one third the basin area in Oyo State (Okekunle, 2000). Asa River is a very significant source of water in terms of economic, agricultural and environmental purposes in the city as it is used in homes and industries (Ahaneku and Animashaun, 2013). There are farmlands, residential and industrial buildings along the bank of the river downstream. The catchment is formed by a ridge of hills rising to almost 580 m above sea level and in most places the catchment is a gently undulating plane. The soils in the catchment area are as a result of weathering of parent rocks, which support vegetation along the source of the river (Salawu, 1987).

The Asa Dam is located at a point about 5 km south of Ilorin across Asa River. It is a composite dam of earth embankment at its extreme ends, with a central spillway followed to the right by a mass concrete non-overflow gravity section. The dam is 597 m long and 27 m high at its deepest section and a crest width of 6 m. A stilling basin of the entire width of the spillway dissipates the energy of the spill flow thereby preventing erosion of the stream bed (Kwara State Water Corporation, 1985).

B. *Development of Synthetic Unit Hydrograph*

The SCS, Soil Conservation Service – Natural Resources Conservation Service, NRCS method was used for computing unit hydrograph needed to develop the storm hydrograph. The time to peak and the peak discharge were estimated using the area of the watershed, slope and length of travel.

Watershed area, A = 2.5 km^2
Length of travel, L = 0.289 km
Watershed slope, S = 0.0014 (Olaniran, 1983)

The time of concentration, t_c, was computed using Kirpich's equation

$$t_c = 0.066\left[\frac{L}{\sqrt{S}}\right]^{0.77} \quad (1)$$

where:

L = length of travel (km)
S = watershed slope m/m
t_c = 0.32 hr

Duration of excess rainfall, D was calculated using equation (2)

$$D = 0.133\, t_c \quad (2)$$

D = 0.042 hr

Time to peak, t_p was then calculated using equation (3)

$$t_p = \frac{(t_c + D)}{1.7} \quad (3)$$

t_p = 0.213 hr

Peak discharge was calculated from equation (4)

$$q_p = \frac{(0.208AQ)}{t_p} \quad (4)$$

q_p = 2.44 m^3/s

The calculated values for peak discharge, q_p, and time to peak, t_p, were used to obtain points for unit hydrograph flow rate and its corresponding time.

Table 1: The sectional geometry and other hydraulic properties of Asa River

Distance from Initial Point (m)	Width, B (m)	Hydraulic Depth, Y (m)	Area, A (m^2)	Wetted Perimeter (m)	Hydraulic Radius, R (m)	Discharge (m^3/s)
1.5	1.75	0.73	1.28	3.21	0.52	0.73
3.5	2.25	0.75	1.69	3.75	0.56	0.63
6.0	2.00	0.76	1.71	3.52	0.62	0.51
8.0	2.00	1.28	2.56	4.56	0.78	0.95
10.0	2.00	1.55	3.10	5.10	0.87	1.57
12.0	2.00	2.50	5.00	7.00	1.11	3.08
14.0	2.00	2.56	5.12	7.12	1.12	3.97
18.0	2.00	2.13	4.26	6.26	1.03	3.32
20.0	2.00	2.80	5.60	7.60	1.17	3.11
22.0	2.00	2.80	5.60	7.60	1.17	3.14
24.0	2.00	2.26	4.52	6.52	1.06	2.65
26.0	2.00	1.49	2.98	4.98	0.85	1.92
28.0	2.00	1.34	2.68	4.68	0.80	1.93
30.0	2.00	1.55	3.10	5.10	0.87	2.16
32.0	2.00	1.34	2.68	4.68	0.80	1.74
34.0	2.00	1.43	2.86	4.86	0.83	2.00
36.0	1.9	1.22	2.32	4.34	0.74	1.54

Source: Kwara State Water Corporation, Ilorin, Kwara State.

C. *Development of Storm Hydrograph*

Storm hydrographs of 10-yr, 24-hr; 25-yr, 24-hr; 50-yr, 24-hr; and 100-yr, 24-hr were developed from the synthetic unit hydrograph using convolution procedures (Wanielista, 1990), with rainfall data from Ogunlela (2001). The cumulative runoff and the watershed potential storage were estimated from:

$$Q = {(P - 0.2S)^2}/{(P + 0.8S)} \quad \text{for } P > 0.2S \qquad (5)$$

$(Q = 0 \text{ for } P \leq 0.2S)$

$$S = \left[\frac{(25400)}{CN}\right] - 254 \qquad (6)$$

where:

Q = cumulative runoff (mm)
S = potential storage (mm)
P = cumulative precipitation (mm)
CN = curve number

A curve number of 71 was obtained for the Asa river watershed from its hydrologic, soil and land – use characteristics.

Holding and Stephenson (1995) stated that for numerical stability, the grid ratio $\frac{\Delta x}{\Delta t}$ must be greater than or equal to the wave celerity, c, where Δx = distance interval and Δt = time step.

Kinematic wave velocity, U was estimated using (Hussain and Ferdous, 2013):

$$U = V + \sqrt{gy} \qquad (7)$$

where:

V = average velocity of flow; 0.588 m/s (from Table 1)

g = acceleration due to gravity; 9.81 m/s^2

y = average depth of flow; 1.701 m (from Table 1)

U = 4.67 m/s

Time required for a small kinematic wave to traverse a distance interval Δx, K, was calculated using;

$$K = \frac{\Delta x}{U} \qquad (8)$$

where:

Δx = change in length of the river (at any interval). (100 m was used)

U = kinematic velocity; 4.67 m/s as previously calculated.

K = 21.4 secs ≈ 21 secs

The area at any given point interval Δx was calculated using;

$$A_{2,2} = \frac{\Delta t}{K + \Delta t} A_{2,2} + \frac{K}{K + \Delta t} A_{1,2} + \frac{I}{U\,(K + \Delta t)} \qquad (9)$$

where:

$A_{1,2}$ is the original area at point where Δx = 0; $A_{1,2}$ = 61.314 m^2

$A_{2,1}$ is the assumed area between $A_{1,2}$ and $A_{2,2}$; $A_{2,1} = A_{2,2} - 0.02$

$A_{2,2}$ is the new area at every length interval, Δx

Δt is the time step which equals to (K – t) and Δt ≤ Δx/U (Courant condition).

And t = 2 secs. Therefore, Δt = (K – 2) in secs.

The outflow was computed using (Barati *et al.*, 2012):

$$Q_{2,2} = \frac{-\Delta x}{\theta \Delta t} A_{2,2} + \left[\frac{1-\theta}{\theta}(Q_{1,1} - Q_{2,1}) + Q_{1,2}\right] + \frac{\Delta x}{\theta \Delta t} A_{2,1} \qquad (10)$$

where:

$Q_{1,1}$ = the first inflow
$Q_{1,2}$ = second inflow
$Q_{2,1}$ = first outflow
$Q_{2,2}$ = second outflow
θ = weighting factor (θ = 0.75)

Since $A_{2,1}$(assumed area) is 0.02 greater than $A_{1,2}$ (area at point where Δx = 0), therefore $A_{2,2}$ (new area at Δx interval) is 0.02 greater than $A_{2,1}$(assumed area).

Before routing, it was assumed that the initial inflow equaled to the initial outflow. After the outflow $Q_{2,2}$ was calculated, it was used as I in equation (9) to get new $A_{2,2}$. The computed $A_{2,2}$ was applied in equation (10) and subsequently used as $A_{1,2}$ in equation (9) for the next iteration to get new $A_{2,2}$ which was used in equation (10) to get new $Q_{2,2}$ and so on.

III. RESULTS AND DISCUSSION

A. *Results*

The results obtained from the whole procedures and all the parameters involved in routing the gauged Asa River are summarized in Table 2. The synthetic unit hydrograph peak discharges ranged from 0.01m^3/s to 2.44m^3/s while the time to peak ranged from 0.02 hr to 1.07 hrs.

The 10-yr, 24-hr, 25-yr, 24-hr, 50-yr, 24-hr and 100-yr, 24-hr storm hydrographs developed gave peak flows of 73.47 m^3/s, 97.01 m^3/s, 113.41 m^3/s and 134.61 m^3/s, respectively with time to peak of 0.213 hr.

The Inflow and Outflow Hydrographs; Synthetic Unit Hydrograph; 10 yr, 24 hr Design Storm Hydrograph; 25 yr, 24 hr Design Storm Hydrograph; 50 yr, 24 hr Design Storm Hydrograph and 100 yr, 24 hr Design Storm Hydrograph for Asa River are shown in figures 1,2,3,4,5 and 6 respectively.

B. *Discussion*

The time of concentration obtained from the estimated watershed parameters of the catchment 0.32 hr as shown in Table 2, was used to compute the time peak and peak discharge requires developing a synthetic unit hydrograph using the dimensionless hydrograph ratio developed by U.S. Department of Agriculture Soil Conservation Service (SCS) (1972), now the Natural Resources Conservation Services (NRSC). The corresponding values of 2.44 m^3/s and 0.213 hr were obtained for peak discharge and time to peak respectively. The developed SCS unit hydrograph was used to construct storm hydrographs of 10 yr, 24 hr; 25 yr, 24 hr; 50 yr, 24 hr; 100 yr, 24 hr.

The peak discharge before routing was 3.97 m^3/s at 1.8 hr and the peak discharge after routing was 3.87 m^3/s at 1.8 hr indicating 0.1 m^3/s attenuation.
Also, the results show that there was some amount of water stored within the reach as well as a delay in the flow of water within the reach which indicates the occurrence of attenuation.
The effect of the discharge will be felt at the upstream of the catchment towards the downstream. There were losses due to seepage, evaporation, inflow into minor tributaries and underground reservoirs. These losses affected the attenuation and storage within the reach.

IV. CONCLUSIONS

The routing of Asa River was carried out using Kinematic wave method. This method exhibited some level of accuracy as shown from the results in Table 2 when compared to some other methods of routing. Kinematic wave is one of the hydraulic methods of routing that takes into consideration watershed characterics and other hydraulic properties of the channel. Moreover, this method of flood routing is tedious and time consuming especially when all the required data are not available.

V. RECOMMENDATIONS

For proper and effective control of flood in the Ilorin metropolis, there should be proper maintenance of all hydraulic structures along the river, and due to the rapid development in the town there should be no erection of any structures along the flood plains. There should be proper channelization and trenching of the river from adequate monitoring of the river to forecast flood. Refuse disposal into the river should be prohibited and be backed up with legislation.

Table 2: Summary of Results

Parameters	Results
Watershed (Estimated) Area, A	2.5 km^2
Length of travel, L	0.289 km
Watershed slope, S	0.0014
Curve Number	71
<u>Inflow Synthetic unit hydrograph</u>	
Time of concentration, T_c	0.32 hr
Duration of Excess rainfall, D	0.042 hr
Time to peak, T_p	0.213 hr
Peak discharge, q_p	2.44 m^3/s
<u>Kinematic Parameters</u>	
Kinematic wave velocity, U	4.67 m/s
Time step, Δt	19 sec
Time required to traverse, K	21 sec
Weighting factor, θ	0.75
Change in length, Δx	100 m

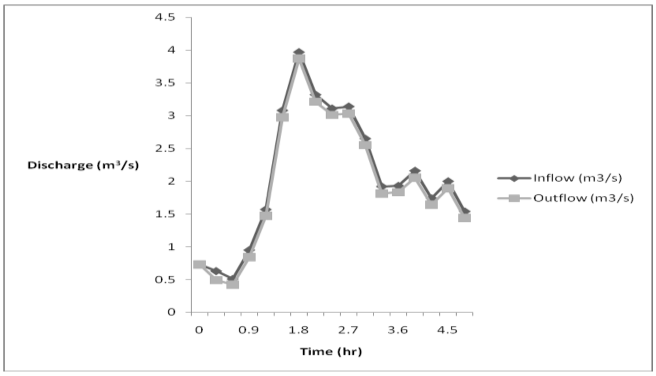

Figure 1: Inflow and Outflow Hydrographs for Asa River

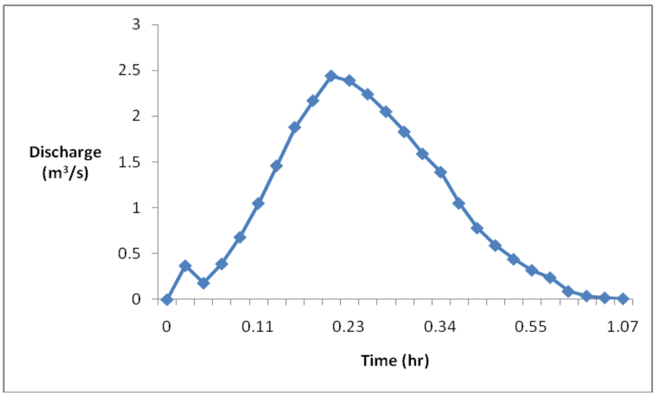

Figure 2: Synthetic Unit Hydrograph for Asa River

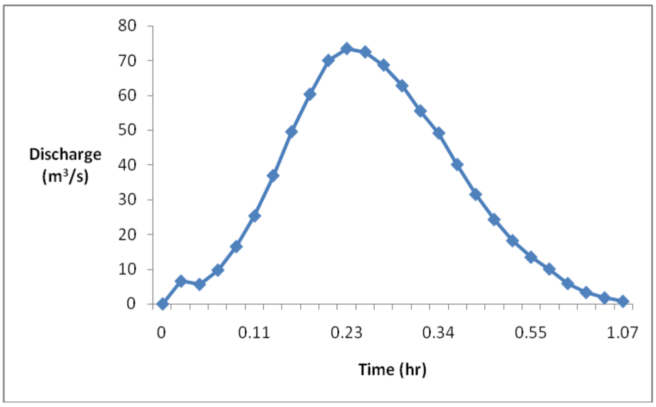

Figure 3: 10 yr, 24 hr Design Storm Hydrograph

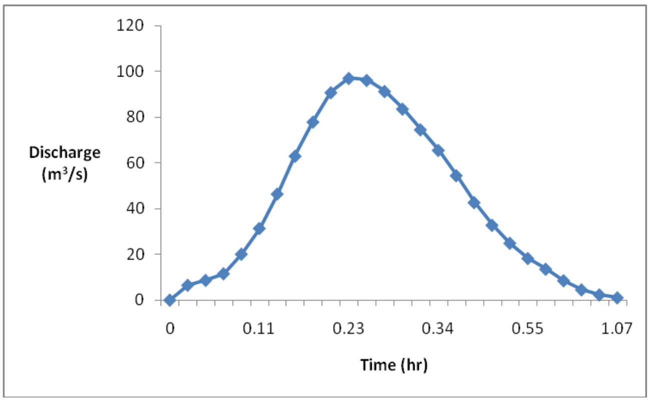

Figure 4: 25 yr, 24 hr Design Storm Hydrograph

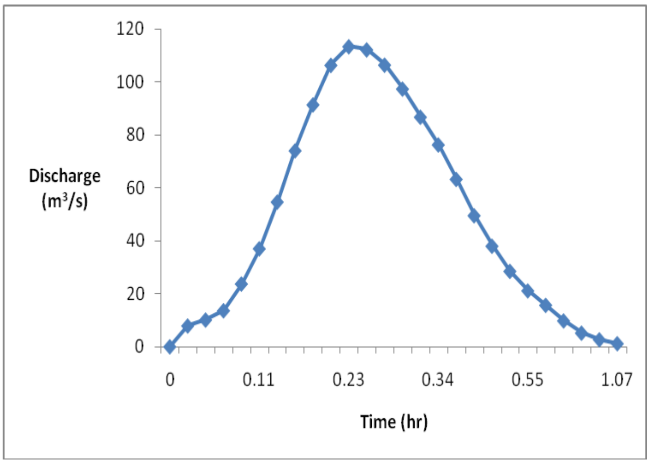

Figure 5: 50 yr, 24 hr Design Storm Hydrograph

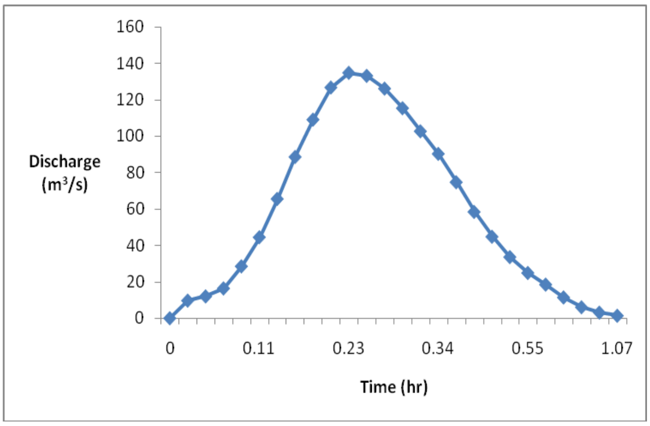

Figure 6: 100 yr, 24 hr Design Storm Hydrograph

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