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STORM HYDROGRAPHS FROM SMALL URBAN CATCHMENTS

by
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Abstract: The Rational method was reviewed, and found to be a special case of the kinematic wave solution under a uniform rainfall excess. In this study, a moving average procedure is introduced to the kinematic wave model. The kinematic wave solution is modified from an instantaneous rainfall to an average rainfall over a period of time of concentration. The revised kinematic wave solution provides the complete storm hydrograph with a peak flow consistent with the Rational method. This new method has been tested using 25 observed rainfall-runoff events from four urban catchments, and yielded a new formula for estimating the time of concentration. The moving average procedure was also examined by continuous non-uniform rainfall events. In general, the kinematic wave solution with the moving average procedure provides close agreements with more sophisticated watershed models for drainage areas up to 150 acres.

Key Words: Kinematic Waves, Rational, Rainfall, Runoff, Time of Concentration

INTRODUCTION

In an urban area, a watershed is divided into small catchments by streets, highways, and channels. Most urban drainage designs are associated with small catchments ranging from 100 to 200 acres. Since traffic safety is the primary concern in an urban area, street drainage has been designed to quickly remove storm water from streets (Guo, 2000). Storm sewers, street gutters, and roadside ditches are major facilities to deliver storm water to nearby water bodies. Since these storm water conveyance systems were designed to pass peak runoff rates, the Rational method has been the most widely used approach because of its simplicity. In the last decade, urban storm water management has changed quite rapidly from a conventional focus on quantity to a focus on both quantity and quality. There are two basic issues that have been exerting considerable influence for this change (Guo and Urbonas, 1996).

(1) There is a fundamental heightening of environmental awareness and concern by the public. Urban storm water along with non-point sources transport pollutants to receiving waters. Efforts to address water quality problems have developed support and momentum.

(2) Water Quality Act (WQA) of 1987, which amended the Federal Water Pollution Control Act, is a reflection of the public's support for pollution control, and such legislation gives focus and direction to general issues. Local governments and industries throughout the United States have a mandate from Congress to minimize the discharge of pollutants to receiving waters.

Since 1987, renaissance of urban drainage systems is to add storm water storage facilities to conveyance systems. Pollutants in storm water shall undergo a settlement process in a detention or retention basin before draining into downstream water bodies. Detention and retention systems are sized by the water volumes associated with a storm hydrograph. Although the Federal Aviation Administration has recommended a volumetric approach based on the Rational method, the difficulty in estimating the average outflow from a detention basin leads to inconsistent predictions from the hydrograph routing method (Guo, 1999). Therefore, under the mandate of the 1987 WQA, the Rational method falls short for urban drainage designs.

The Rational method is a simplified procedure to predict peak runoff rates only. The design rainfall distribution used in the Rational method is converted into an intensity-duration relationship. To predict the peak runoff, the critical rainfall period is set to be the time of concentration of the catchment (Clark County Hydrologic Criteria and Drainage Design Manual, 1999). When introducing the Rational method to drainage designs, Kuichling (1889) stated that the peak rate of runoff at a design point is a direct function of the tributary area, and the tributary rainfall amount over the time of concentration of the catchment. Chow (1964) applied the concept of hydrologic system to a catchment, and suggested that the output from a small catchment cannot precede its corresponding inputs over a period of time in the past. Such a time period is termed the system memory over which the historical input affects the present system behavior. Singh (1982) further suggested that the system memory is, in fact, the time of concentration of the catchment. In practice, the time of concentration is used as a moving window to select the most intense period on the rainfall distribution. The average rainfall intensity over such an intense period is considered as the contributing rainfall to the peak runoff (Guo, 1998).

Storm hydrograph prediction methods were developed with an emphasis on the rainfall-runoff relationships for large watersheds (Snyder 1938; Sherman 1932). Empirical formulas for estimating unitgraph parameters and hydrologic losses do not conform to the Rational method. As a result, engineers have experienced the inconsistency between the predicted runoff peak rates and water volumes due to different methodologies. For instance, in order to achieve the consistency between the Rational method and the SCS unitgraph method, Clark County in Las Vegas, Nevada recommends a modified Rational method which applies an adjustment factor to the conventional Rational method (*Clark County Hydrologic Criteria and Drainage Design Manual, 1999*). Denver City and County has also revised the local empirical formula for estimating the time to peak on the unitgraph for watersheds smaller than 90 acres (*Urban Storm Water Design Criteria Manual, 1989).* All of these methods represent efforts to overcome the insufficiency of the Rational method in estimating storm hydrographs under a non-uniform rainfall distribution.

This study presents an investigation on the kinematic wave method, and concludes that the Rational method is a special case of the kinematic wave solution. The kinematic wave solution was then expanded from an instantaneous rainfall to an average rainfall over the time of concentration. These modifications lead to a moving average procedure to expand the kinematic wave solution from a peak runoff prediction method under a uniform rainfall distribution to a hydrograph prediction method under a non-uniform rainfall distribution. The moving average procedure can generate a complete hydrograph with a peak flow consistent with the Rational method. With this modification, the conventional Rational method is employed as a peak flow prediction method for storm water conveyance designs, and can be expanded into a hydrograph prediction method wherever a storm water storage facility needs to be sized in the drainage system.

MODIFICATION TO KINEMATIC WAVE MODEL

As suggested by the Urban Highway Storm Drainage Model using the EPA SWMM procedure (FHWA 1983), an urban catchment can be converted into an equivalent rectangular watershed. The continuity equation of kinematic waves applied to a rectangular watershed is (Yen and Chow, 1974):

$$\frac{\partial y}{\partial t} + \frac{\partial q}{\partial x} = i_e \tag{1}$$

in which t = time variable, x = distance from the upper boundary, q = flow rate per unit width, y = runoff depth, and i_e = instantaneous excess rainfall intensity. A kinematic wave solution takes a form similar to a rating curve (Wooding 1965; Morgali 1970) as:

$$q = \alpha y^m \tag{2}$$

in which α = conveyance factor and m = empirical parameter varying between 1.5 to 3.0, depending on the regime of flow. Taking the first partial derivative of Eq 2 with respect to time and re-arranging the variables yields:

$$\frac{\partial y}{\partial t} = \frac{1}{\alpha m y^{m-1}} \frac{\partial q}{\partial t} \tag{3}$$

Substituting Eq 3 into Eq 1 yields

$$\frac{\partial q}{\partial t} + \alpha m y^{m-1} \frac{\partial q}{\partial x} = \alpha m y^{m-1} \quad i_e \tag{4}$$

Eq 4 is the total derivative of runoff rate, q, namely:

$$\frac{dq}{dt} = V_w i_e \tag{5}$$

Eq 5 indicates that the variation of flow rate is directly related to the wave speed and instantaneous rainfall intensity. The speed of kinematic waves in Eq 5 is

$$\frac{dx}{dt} = V_w = \alpha m y^{m-1} \tag{6}$$

The initial condition for Eq's 5 and 6 can be a dry bed condition as:

$$q = 0 \quad for \quad 0 \le x \le L \quad at \;\; t = 0 \tag{7}$$

in which L = waterway length. The condition at the upper boundary of the catchment is described as:

$$q = 0 \;\; at \;\; x = 0 \tag{8}$$

Consider the concept of system memory as illustrated in Figure 1. The runoff rate, q(T), at time T after the rain begins, is attributed to the rainfall amount from t = (T-T_c) to t = T. Aided by Eq 2, Eq's 5 and 6 are integrated over the time of concentration as (Singh and Cruise 1992, Guo 1998):

$$L = \int_{t=T-T_c}^{t=T} \alpha m y^{m-1} dt \tag{9}$$

$$q(T) = \int_{t=T-T_c}^{t=T} V_w \, i_e \, dt = I_e(T) \int_{t=T-T_c}^{t=T} \alpha m y^{m-1} dt = L \, I_e(T) \tag{10}$$

in which I_e(T) = average excess rainfall intensity over the period from (T-T_c) to T.

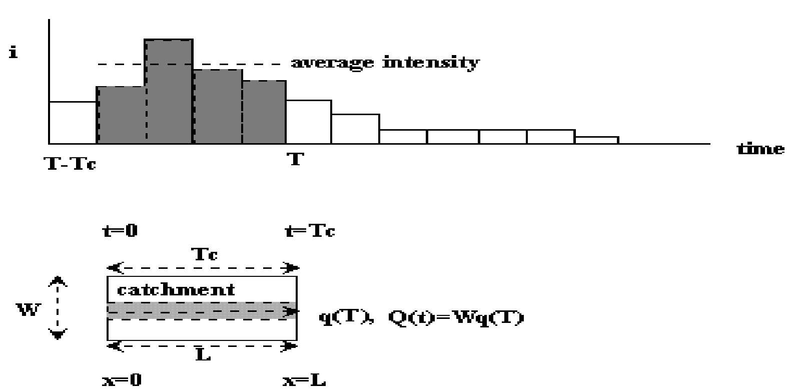

Figure 1 Integration Domains for Moving Average Procedure

Eq 10 is achieved with the assumption that the instantaneous excess rainfall intensity can be replaced by the average excess rainfall intensity over the time of concentration. Such an assumption is acceptable for a small urban catchment when the time of concentration is less than 30 minutes. Under a non-uniform rainfall distribution, the average rainfall intensity over a period of T_c is defined as:

$$I(T) = \frac{1}{T_c} \sum_{t=T-T_c}^{t=T} \Delta p(t) \tag{11}$$

in which $I(T)$ = average rainfall intensity at time T, and $\Delta p(t) =$ incremental rainfall depth. By the definition of runoff coefficient, the average excess rainfall intensity is equal to

$$I_e(T) = C\, I(T) \tag{12}$$

in which C = runoff coefficient. To apply the unit-width runoff in Eq 10 to the entire catchment yields

$$Q(T) = Wq(T) = WLI_e(T) = AI_e(T) \tag{13}$$

in which W= catchment width, A = catchment area, and Q(T) = runoff from the catchment at time T. Substituting Eq's 11 and 12 into Eq 13 yields

$$Q(T) = \frac{CA}{T_c} \sum_{t=(T-T_c)}^{t=T} \Delta p(t) = C\, I(T)\, A \qquad for \quad T_c \leq T \leq T_d \tag{14}$$

in which T_d = duration of rainfall distribution. Eq 14 implies that the entire watershed has become the tributary area to runoff. Therefore, it is only applicable to the peaking portion on a hydrograph. Before the time of concentration, the contributing area to the runoff rate at time, T, is proportional to the ratio of T to T_c. Unless a time-area relationship is specified for the catchment, such a contributing area in a linear model may otherwise be approximated as:

$$A_e = A\frac{T}{T_c} \tag{15}$$

in which A_e = effective tributary area. As a result, the runoff rate before the time of concentration is estimated as:

$$Q(T) = C\, A_e\, [\frac{1}{T} \sum_{t=0}^{t=T} \Delta p(t)] = CI(T)\, A\frac{T}{T_c} \qquad for \quad 0 \leq T \leq T_c \tag{16}$$

The recession of a hydrograph begins when the rain ceases. In general, kinematic waves on the recession limb of a hydrograph are slower than on the rising limb. The recession is often described by a decay function. In this study the following linear approximation is adopted for a small catchment as:

$$Q(T) = Q(T_d)[1 - \frac{T-T_d}{T_c}] \qquad for \quad T_d \leq T \leq (T_d + T_c \tag{17}$$

in which $Q(T_d)$ = runoff rate after the rain ceases.

IDENTIFICATION OF CATCHMENT PARAMETERS

As indicated in Eq's 9 and 10, the time of concentration varies with respect to the amount of rainfall. This conclusion concurs with the experience of McCuen et al. (1984) concerning the reliability and measurability of the time of concentration. In this study, the best value of the time of concentration is identified by the least-squares method applied to the entire hydrograph as:

$$Min\ E = Min \sum_{T=0}^{T=T_d} (Q_o(T) - Q(T))^2 \tag{18}$$

in which $Q_o(T)$ = observed runoff at time T, $Q(T)$ = predicted runoff by Eq's 14 and 16, and E = cumulative squared error. In this study, USGS Open File 82-873 (1983) of 5-minute rainfall and runoff records is employed to establish a data base. A total of 44 storm events were analyzed for four small urban watersheds in the metropolitan areas of Denver, Colorado. The hydrologic parameters of these four watersheds are summarized in Table 1.

USGS Gage Number	Location of Catchment	Drainage Area acres	Waterway Length ft	Watershed Slope percent	Land Use	Predicted Time of Concentration		
						Kirpitch Formula min	SCS Upland min	Eq 20 min
6728200	Kennedy Drive Near Northglenn	83.0	2500.0	0.0231	Commercial	13.78	18.28	16.22
6714310	Sand Creek Near Denver	186.0	6600.0	0.0187	Residential	31.51	40.22	33.00
6711600	Sanderson Gultch near Denver	243.0	5000.0	0.0130	Residential	29.27	36.54	30.98
6728400	Boulder Creek near Boulder	128.0	3500.0	0.0485	Residential	13.40	17.66	15.85

Table 1 Hydrologic Parameters for Sample Watersheds used in the study

For each event, Eq 18 was tested for all possible combinations of pairs (T_c , C) using a matrix approach. The pair that produced the smallest squared error was selected as the event-averaged values. Figure 2 presents the study of the event observed on August 18, 1979 at Kennedy Drive Watershed. The cumulative squared errors are plotted for runoff coefficients of 0.40, 0.52, and 0.60 with times of concentration of 10, 20, 30, 40, and 50 minutes. It can be seen that the minimum deviation is achieved by the pair of 20 minutes for T_c and 0.52 for C.

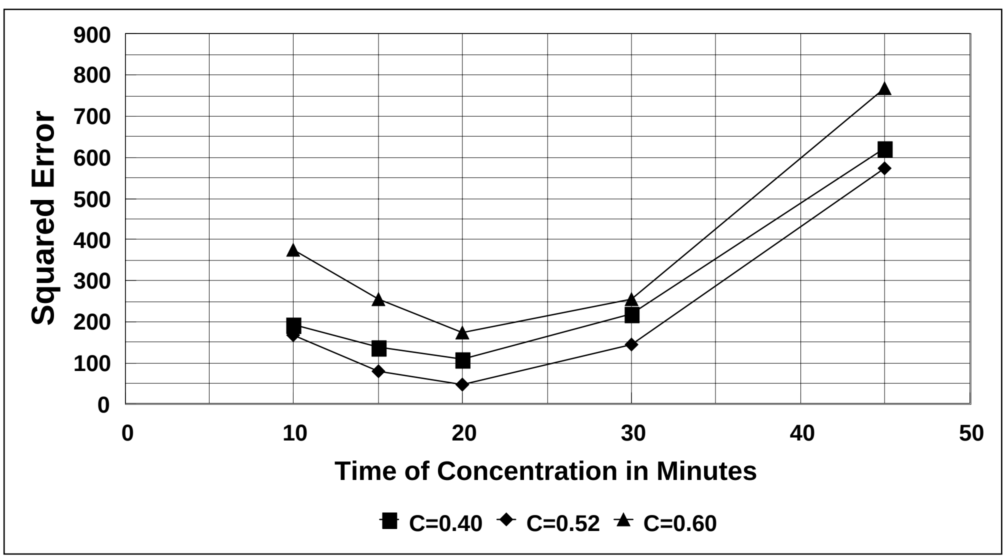

Figure 2 Distribution of Squared Error for Various Pairs Tc and C

Existing empirical equations for estimating the time of concentration are classified into velocity-based methods and time lag methods (Guo 1998). These empirical formulas were developed for different hydrologic conditions. In this study, the times of concentration were not measured by lag times, nor calculated by waterway characteristics, but derived by Eq 18 as an event-averaged value. Therefore, it is necessary to detect any difference between the derived and measured times of concentration. In this study, a regression analysis was conducted on 25 rainfall/runoff events from four catchments as shown in Table 2.

Date of Rainfall Event	Rainfall Depth inch	Rainfall Duration minutes	Derived Time of Concentration minutes	Derived Runoff Coefficient	Observed Peak Runoff Rate cfs	Predicted Peak Runoff Rate cfs	Sum of Runoff Prediction Error for Hydrograph cfs	Predicted Time of Concentration Kirpitch Formula min	SCS Upland min	Eq 20 min
04/01/78	0.15	85.00	20.00	0.45	13.00	11.50	8.60	13.78	18.32	15.18
05/28/78	0.10	20.00	15.00	0.50	19.00	17.50	6.30	13.78	18.32	15.18
05/30/78	0.15	20.00	10.00	0.40	31.00	29.50	14.00	13.78	18.32	15.18
03/06/78	0.35	65.00	20.00	0.65	30.00	28.30	15.80	13.78	18.32	15.18
06/07/78	0.24	15.00	15.00	0.70	62.00	55.70	16.00	13.78	18.32	15.18
07/10/78	0.12	15.00	10.00	0.60	33.00	32.37	11.90	13.78	18.32	15.18
05/20/79	0.32	120.00	20.00	0.45	8.80	8.20	5.80	13.78	18.32	15.18
05/30/79	0.11	50.00	15.00	0.85	11.00	10.60	6.90	13.78	18.32	15.18
08/09/79	0.39	70.00	20.00	0.50	27.00	25.30	9.90	13.78	18.32	15.18
08/10/79	0.66	50.00	10.00	0.60	150.00	125.80	62.40	13.78	18.32	15.18
04/29/78	0.28	65.00	35.00	0.22	10.50	10.10	10.00	31.51	53.63	30.84
07/21/78	0.09	45.00	25.00	0.28	5.50	5.50	6.30	31.51	53.63	30.84
07/09/80	0.41	30.00	25.00	0.13	20.00	18.80	12.90	31.51	53.63	30.84
07/16/80	0.24	45.00	35.00	0.31	25.00	16.50	23.00	31.51	53.63	30.84
07/21/80	0.22	50.00	35.00	0.32	24.50	22.00	18.30	31.51	53.63	30.84
06/29/78	0.22	75.00	40.00	0.20	12.00	9.80	7.50	29.27	48.73	28.95
07/17/78	0.16	70.00	30.00	0.25	11.00	8.90	9.40	29.27	48.73	28.95
04/10/79	0.34	100.00	40.00	0.27	17.50	17.30	10.10	29.27	48.73	28.95
05/01/79	0.38	100.00	35.00	0.27	18.50	18.20	9.20	29.27	48.73	28.95
05/20/79	0.50	115.00	30.00	0.46	54.00	42.50	46.30	29.27	48.73	28.95
06/17/79	0.07	30.00	30.00	0.42	14.20	14.29	6.70	29.27	48.73	28.95
06/23/79	0.16	35.00	35.00	0.33	28.00	22.00	11.70	29.27	48.73	28.95
07/04/79	1.11	50.00	30.00	0.19	118.00	112.00	47.30	29.27	48.73	28.95
07/09/78	0.20	40.00	25.00	0.21	8.50	7.40	12.19	13.40	17.66	14.82
08/25/80	0.36	105.00	20.00	0.26	7.10	7.05	13.90	13.40	17.66	14.82

Table 2 Rainfall and Runoff Events Used in the Study.

An equation was produced as:

$$T_c = m\left(\frac{L}{\sqrt{S}}\right)^{0.66} \qquad (r^2 = 0.71) \tag{19}$$

in which, T_c = time of concentration in minutes, L = catchment waterway length in meters or feet, S = catchment basin slope in ft/ft, r = correlation coefficient, m = constant equal to 0.065 for metric units or 0.029 for English units. In this study, the data base is inadequate to further investigate the value of m for various land uses and vegetal roughness in waterways. Among the 11 empirical formulas for estimating time of concentration (McCuen et al., 1984), the exponent for the watershed length varies from 0.13 to 0.80. Eq 19 suggests a value of 0.66 for the watershed length, which is compatible with the value of 0.60 derived by the kinematic wave equation under uniform rainfall (Wooding 1965) or the value of 0.77 recommended by Kirpitch formula (Kirpitch 1941). Aided by Eq 19, the waterway-averaged flow velocity is derived as:

$$V = \frac{L}{60T_c} = \frac{1}{N}(LS)^{0.34} = \frac{1}{N}\Delta H^{0.34} \tag{20}$$

in which V = waterway-averaged flow velocity in mps or fps, ΔH = vertical drop in meters or feet, and N= coefficient equal to 3.54 for metric units or 1.50 for English units. Eq 20 represents the average velocity over the waterway length under runoff cumulative effects. Such an average value represents a spatial and temporal unsteady flow process, and is smaller than the cross sectional average value based on the peak flow at the watershed outlet. For instance, the waterway-averaged flow velocity in a 3000-ft waterway on a slope of 5.0% is 3.25 fps by Eq 20, and 7.0 to 9.0 fps by Manning's formula with a roughness coefficient of 0.030. To apply the SCS upland method to the above example, the flow velocity is predicted to be 3.35 fps in a grass waterway. Therefore, this study confirms that the SCS upland method in fact was derived to provide a waterway-averaged velocity, not a cross sectional average velocity.

COMPARISON AND APPLICABILITY

The storm event recorded on August 18, 1979, at Kennedy Drive Watershed near Denver, Colorado, is used as an example to illustrate application of the moving average procedure. The USGS records indicate that the total precipitation for this event was 0.55 inch with a duration of approximately two hours and a peak runoff rate of 23.0 cfs. As shown in Table 1, the time of concentration of this catchment was estimated to be between 13 to 18 minutes. The runoff coefficient is identified to be 0.52 in Figure 2. For this case, T_c=15 minutes was tested by the moving average procedure. The EPA's Storm Water Management Model (SWMM) using the differential kinematic wave equations, and the Colorado Urban Hydrograph Procedures (CUHP) using the unit hydrograph method were also tested for this case. Figure 3 presents the observed and predicted hydrographs by various methods. The moving average method with T_c=15 minutes produces the smallest deviation by Eq 18. The predicted hydrograph by the moving average procedure reflects the temporal variations on the hyetograph.

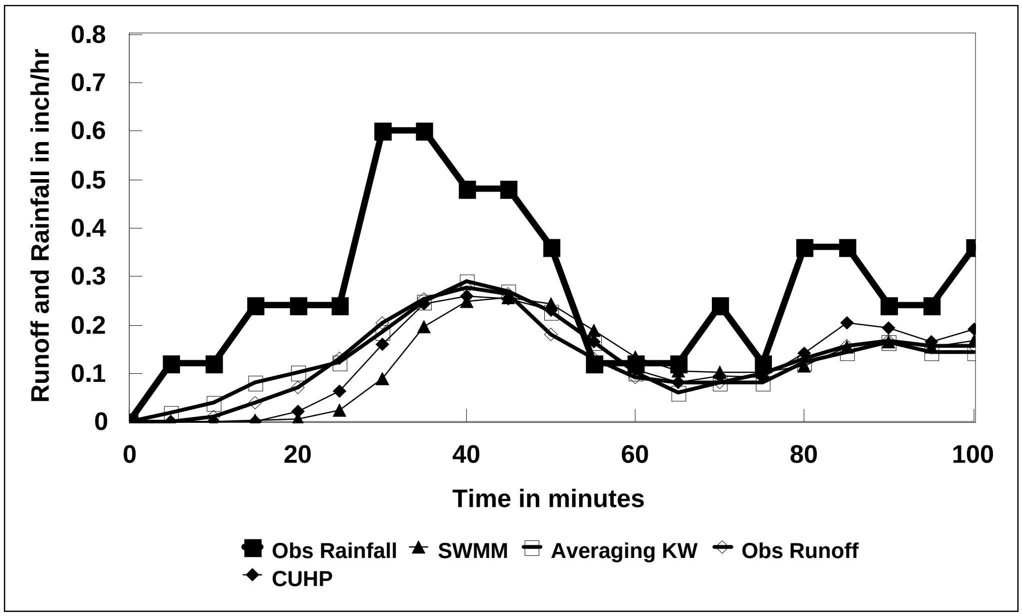

Figure 3 Comparisons Between Observed and Predicted Hydrographs by Various Models

The applicable limit of the moving average procedure was further investigated by a series of hypothetical square watersheds ranging from 0.01 to 1.0 square mile. With a rainfall depth of 2.77 inch distributed on the SCS 6-hour curve, a runoff coefficient of 0.75, and an equivalent SCS Curve Number of 85, determined by the SCS loss function (McCuen 1982), runoff hydrographs were predicted by the HEC-1 model (HEC-1 1985) using the SCS unit hydrograph method. Figure 4 presents the comparisons of the predicted peak flow rates between HEC-1 and the moving average procedure. It can be seen that the predicted peak runoff rates by both methods are very comparable. The difference is less than 6.0% for these hypothetical square watersheds up to 450 acres. However, considering the range of field data used in the study, and the variations of storage capacities and depression losses in natural watersheds, it is suggested that the moving average procedure for kinematic wave solutions is applicable up to 150 acres.

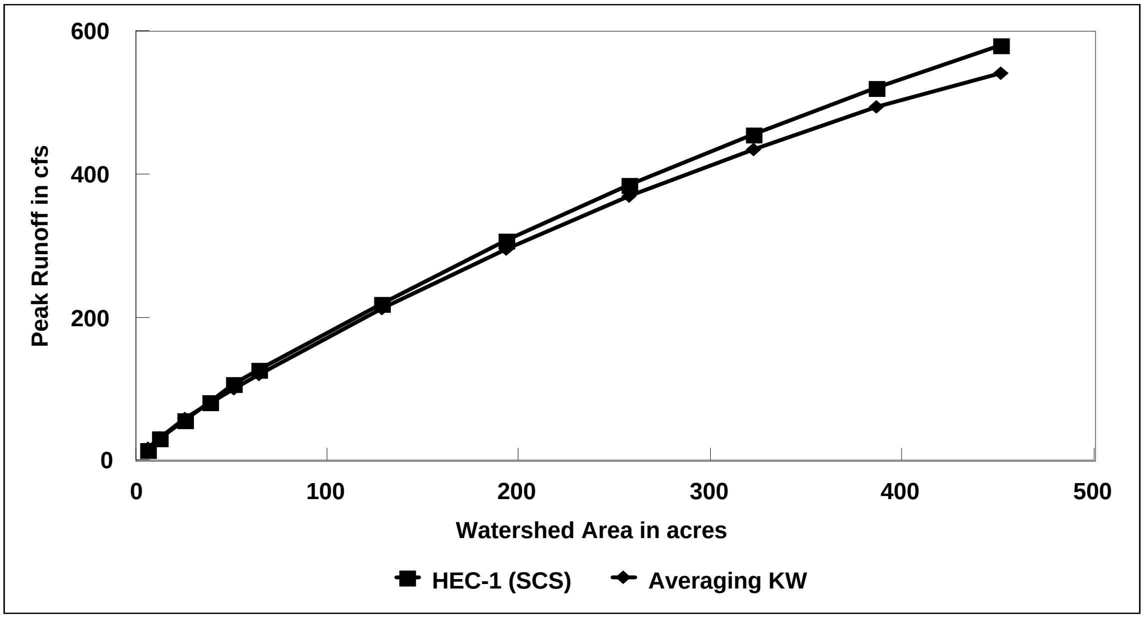

Figure 4 Comparison Between HEC-1 and the Moving Average Procedure used in Kinematic Wave Method.

CONCLUSION

In this study, a moving average procedure is introduced to the kinematic wave approach. The kinematic wave solution is modified from an instantaneous rainfall to an average rainfall over the time of concentration. This modification expands the Rational method from a peak flow prediction method under a uniform rainfall distribution to a hydrograph prediction method under a non-uniform rainfall distribution. When the rainfall distribution is uniform, the moving average procedure is reduced to the conventional Rational method which produces triangular or trapezoidal hydrographs. The least-square scheme developed in this study provides an algorithm to derive the event-averaged time of concentration, and runoff coefficient from a runoff hydrograph. In this study, a new equation for estimating the time of concentration was generated from 25 rainfall-runoff events observed in four urban catchments. In comparison, the new equation is comparable with the velocity-based empirical formulas. The flow velocity predicted by the new equation is a waterway-averaged value.

The moving average procedure was also tested for the 6-hour SCS design rainfall distribution and compared with SWMM, HEC-1, and Colorado Urban Hydrograph Procedure (CUHP) models. The moving average procedure is consistent with the Rational method, and provides close agreements to those sophisticated models when the watershed area is smaller than 150 acres.

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Appendix II - Notation

C = runoff coefficient
E = summation of squared error
ΔH = vertical drop'
$i_e(T)$ = average rainfall intensity for the period from $(T-T_c)$ to T
I(T) = average rainfall intensity for the period from $(T-T_c)$ to T
$I_e(T)$ = average excess rainfall intensity for the period from $(T-T_c)$ to T

K = system constant associated with the system memory
L = length of waterway
m = exponent constant
N = roughness coefficient of waterway
M = regression constant
P = event precipitation
$\Delta p(t)$ = incremental rain depth
q(T) = runoff rate per unit width
Q(T)= predicted runoff rate per watershed
$Q(T_d)$ = runoff rate after the rain ceases.
$Q_o(T)$ = observed runoff rate per watershed
S = slope of the watershed.
t = time variable between 0 and T_c
T = time on hydrograph.
T_c = time of concentration
T_d = rainfall duration
x = distance along waterway between 0 and L.
u = flow velocity
V = waterway-averaged velocity
V_w = celerity of kinematic wave
α = conveyance factor

Dr. James C.Y. Guo's Bio

Dr. James C.Y. Guo graduated from the University of Illinois at Champaign-Urban with a Ph. D. in Water Resources in 1982. Since then he has been associated with the University of Colorado at Denver as a Professor in the Department of Civil Engineering. Dr. Guo has published more than 50 technical papers, three books, and ten computer models in the area of storm water design and management. Dr. Guo has been recognized with the teaching, research, and service awards from the University of Colorado at Denver, and the 1996 Honor Award by the Colorado Engineering Council