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STORM WATER PREDICTIONS BY KINEMATIC WAVE DIMENSIONLESS UNIT HYDROGRAPH

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Abstract This paper presents a theoretical derivation of a dimensionless unit hydrograph using the kinematic wave approach. The dimensionless kinematic wave unit hydrograph (KWUH) is normalized by the parameters associated with the equilibrium condition and can be generated according to the selected rainfall duration. For comparison, the KWUH whose rainfall duration is equal to the time of equilibrium produces good agreement with the Soil Conservation Service dimensionless unit graph (SCSUH). Both the KWUH and SCSUH are kinematic in nature and have a 37.5% runoff volume that occurred before the time to peak. To verify the applicability of the KWUH derived in this study, a set of hypothetical rectangular watersheds ranging from 20 to 200 acres was investigated. The predicted 100-year peak flow rates from these five hypothetical rectangular watersheds are compared with the SCSUH method and the Colorado Unit Hydrograph Procedures (CUHP). In general, the KWUH produces good agreements with the CUHP when the catchment has an area between 60 and 120 acres. Based on the study of hypothetical rectangular watersheds, it is recommended that the KWUH be applicable to urban catchments with a drainage area of up to 120 acres.

Key words: dimensionless, non-dimension, unit graph, hydrograph, runoff, storm water, rainfall, kinematic wave,

INTRODUCTION

Rainfall and runoff processes can be studied by a physical watershed model in the laboratory or a numerical model using computers. Applying hydrodynamic laws to the flow network, temporal and spatial rainfall distributions are converted to runoff hydrographs by various linear or nonlinear numerical schemes. Although the rainfall/runoff process is highly nonlinear, several linear methods such as the unit hydrograph and the rational method are still recommended. The major assumption of unit hydrograph approach is the linear relationship between runoff rates and rainfall excess amount. Strictly speaking, such a linear relationship is only applicable to a linear reservoir system where storage volume is linearly related to outflow. However, in the absence of knowledge of the watershed, the assumption of linearity is acceptable.

The concept of a unit hydrograph was originally introduced for the analysis of observed rainfall and runoff events (Sherman 1932). Such a linear approach was later expanded to the synthetic unit hydrograph method for flood predictions (Snyder 1955). Coefficients used in Snyder's formulas are correlated to watershed topographic parameters including drainage area, length of waterway, length to the centroid of watershed, and slope of watershed. A synthetic unit hydrograph is often constructed using peaking parameters. For instance, the Soil Conservation Service unit hydrograph (SCSUH) utilizes time to peak and peak discharge as the parameters to predict storm hydrographs. For convenience, the SCS unit graphs were further normalized to simplify the convolution process when generating storm hydrographs (SCS 1957).

The SCSUH was derived from a large number of agricultural unit hydrographs observed from the Appalachian Mountain region (McCuen 1982). Although the application of the SCS unit graph has been extended to many metropolitan areas, acceptance of the SCSUH is often justified by the consistence of the method. In the State of Colorado, the Colorado Unit Hydrograph Procedure (CUHP) was developed for metropolitan areas. The major parameters used in the CUHP were calibrated by area imperviousness ratio using urbanized watershed data. The CUHP has been widely accepted by metropolitan areas in the front range of the Rocky Mountains for predicting flood hydrographs (Urbonas 1979). Both the SCSUH and CUHP are similar to Snyder's approach. Time to peak and peak discharge per square mile are used as the major parameters in the determination of storm hydrographs from a large watershed. Both the CUHP and SCSUH have been revised for small, urbanized catchments. For instance, TR 55 was published to introduce the peaking factors as an adjustment to SCSUH when applying to urban areas (SCS 1986). And the time to peak used in the CUHP was also replaced with the time of concentration when applying the CUHP to urban catchments less than 150 acres (Guo and Urbonas in 1988). With the modifications, both methods are still sensitive to the size or imperviousness ratio of the catchment. For instance, the hydrologic losses using the SCS curve number are exponentially sensitive to a high impervious area ratio. And the CUHP may run into numerical instability when the catchment is smaller than three acres. As a result, it is imperative that new unit hydrograph methods be developed for small urban catchments.

This paper presents an attempt to derive the dimensionless unit hydrograph method using the kinematic wave approach. The kinematic wave theory for overland flow is a simplified approach of dynamic waves described by Saint Venant equations (Chow 1962). Solutions of kinematic waves have been developed for various flow conditions (Chow and Maidment 1976). In this study, the solution for overland flow on a sloping plane was achieved by the characteristic method. The normalized kinematic wave unit hydrograph, KWUH, was derived and then compared with the SCSUH and CUHP. The applicability of KWUH was studied by a set of hypothetical watersheds ranging from 20 to 200 acres and comparisons with the CUHP indicate that the KWUH is applicable to urban catchments up to 150 acres.

DERIVATION OF UNITGRAPH

Overland flow has been studied by Lighthill and Whitham (1955), Wooding (1965), Yen and Chow (1974), Morgali (1970) and many others. The solution of overland flow on an impervious surface has been translated into the dimensionless rising hydrograph without a recession limb (Woolhiser and Liggett in 1967). As a result, its application becomes limited. The dimensional solution of an overland flow on a pervious surface was also achieved for the cases with various ratios of infiltration rate to rainfall intensity (Guo 1998). The latest developments in the overland flow modeling techniques include neural network technology (Guo 2000A). But, so far, none has provided a complete normalized KWUH.

As recommended (EPA 1983), an irregular urban catchment can be converted to its equivalent rectangular shape. As illustrated in Figure 1, the uniform rainfall distribution is applied to the rectangular watershed that has a central channel collecting the overland flows from both sloping planes.

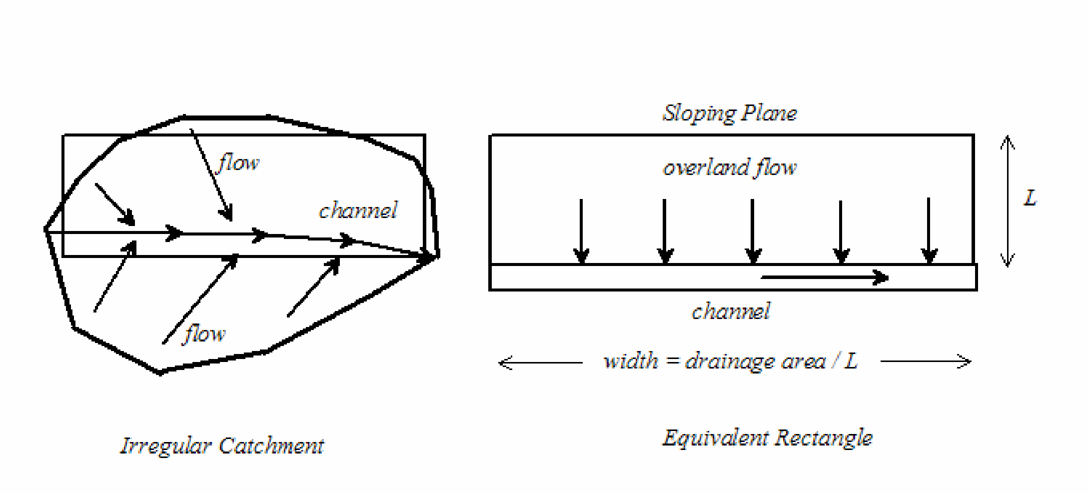


Figure 1 Conversion of Irregular Catchment into Equivalent Rectangular Shape

During the shape conversion, the drainage area and its transverse slope toward the collection channel must be preserved. An overland flow is often modeled by the distributed flow routing method using the Saint Venant equations. The distributed flow routing model essentially transforms the rainfall distribution into the runoff distribution over a unit-width strip on the sloping plane. The unit-width overland flow hydrograph can then be treated as the lateral inflow to the collection channel for channel storage and routing simulations. If the channel storage is negligible, the catchment hydrograph is approximated by the unit-width hydrograph multiplied by the width of the catchment.

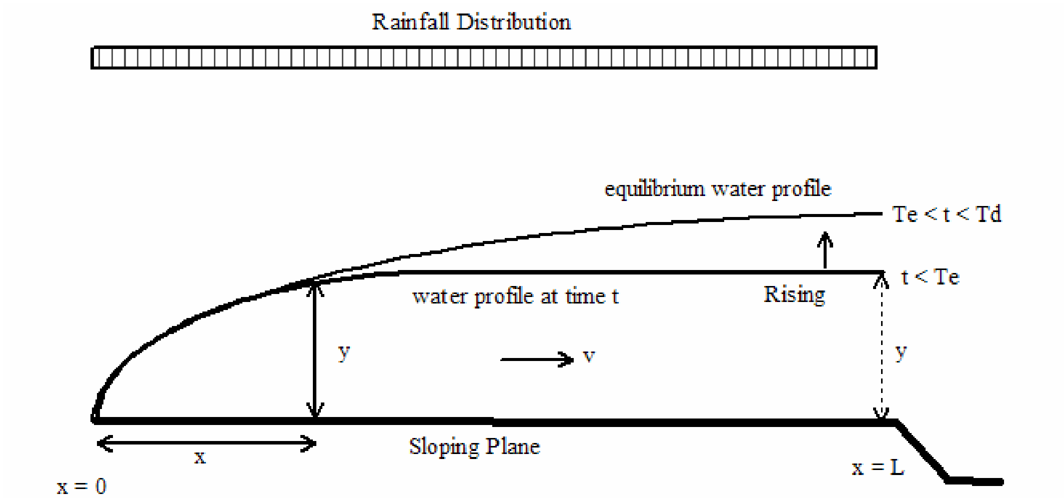
Under the assumption that the flow friction is balanced by the gravitational effect, the governing equations for unit-width overland flow are

$$\frac{\partial y}{\partial t} + \frac{\partial q}{\partial x} = i_e \qquad (1)$$

$$q = \alpha y^{\beta} \qquad (2)$$

in which y = flow depth, q = flow rate per unit width, i_e= excess rainfall intensity, α = empirical factor, depending on plane slope and roughness according to Manning's equation, β = empirical factor between 3/2 and 5/3 for practical purpose, t= time, and x = distance varied from $x=0$ to $x=L$ in which L = length of the overland flow. Derivation of kinematic wave governing equations can be readily found elsewhere (Bedient and Huber in 1992).

The governing equations of kinematic wave flow can be analytically solved by the characteristic method or numerically solved by the implicit or explicit method. As illustrated in Figure 2, the characteristic solutions for kinematic wave flows are formed by flow travel distance (x), and travel time (t).



Figue 2 Rising Water Surface Profile Before Equilibrium

This study applies the characteristic method to analytically solve the overland flow, and the normalized solutions were further converted into a dimensionless unit hydrograph. Taking the first partial derivative of Eq. 2 with respect to x yields

$$\frac{\partial q}{\partial x} = \alpha\beta\, y^{\beta-1}\frac{\partial y}{\partial x} \quad (3)$$

Substituting Eq 3 into Eq 1 yields the total derivative for flow depth as:

$$\frac{dy}{dt} = i_e \quad (4)$$

According to Eq. 3, the character wave speed is defined as

$$v = \frac{dx}{dt} = \alpha\beta\, y^{\beta-1} \quad (5)$$

in which v = kinematic wave speed. Eq's 1 through 5 will be used to predict the runoff generated by the uniform rainfall excess on a unit width catchment. By definition, a unit hydrograph must have a unit volume for the specified rainfall duration. And the rainfall duration has been selected, the intensity of rainfall excess for the unit hydrograph can be determined as :

$$i_e = \frac{D}{T_d} \quad (6)$$

in which T_d = design rainfall duration, and D = unit rainfall depth. The quantity of unit rainfall depth in Eq 6 may be one inch or one mm. The initial condition for the overland flow is: y(t) = 0 everywhere, and upstream boundary condition is: y(t) = 0 at x = 0. Under these conditions, Eq 4 is integrated as:

$$y = i_e \ t \tag{7}$$

Aided by Eq 7, Eq 5 is integrated as:

$$x = \alpha \, i_e^{\beta-1} t^{\beta} = \frac{\alpha \, y^{\beta}}{i_e} \tag{8}$$

in which *x* = location for flow depth, y, in a rising water surface profile as shown in Figure 2. The characteristic solutions are formed with flow length, time, and depth. For a specified flow depth, Eq 8 defines the required upstream tributary area in terms of the flow length, x, and Eq 7 defines the travel time in terms of the elapsed time, t, for water to flow through the upstream tributary area. The time of equilibrium is defined as the travel time for the kinematic wave flow from x = 0 to reach the outfall point at x = L. Aided by Eq. 8, the time of equilibrium for the sloping plane is derived as

$$T_e = \left(\frac{L}{\alpha \, i_e^{\beta-1}} \right)^{\frac{1}{\beta}} \tag{9}$$

in which T_e = time of equilibrium. Substituting Eq 6 into Eq 9 yield:

$$T_e = \frac{L}{\alpha \, D^{\beta-1}} \tag{10}$$

Eq 10 indicates that the time of equilibrium under a unit rainfall depth linearly varies with respect to the overland flow length. The denominator in Eq 10 is essentially the same as Manning's formula with a flow depth equal to the unit rainfall depth. Under the equilibrium condition, the flow depth is determined by Eq 7 as:

$$y_e = i_e \ T_e \tag{11}$$

in which y_e = equilibrium flow depth at x = L or the outfall point. Aided by Eq 2, the equilibrium flow rate is

$$q_e = \alpha \ y_e^{\beta} \tag{12}$$

in which q_e = unit-width equilibrium flow rate. Aided by Eq 6, the equilibrium depth can be calculated by Eq's 11 and 12.

When modeling storm water, it is essential that the design rainfall duration be longer than the equilibrium time of the catchment so that the entire catchment contributes to the peak flow. When a long rainfall is applied to a catchment, the overland flow can reach its equilibrium condition in which the discharge from the tributary area is equal to the rate of rainfall amount fallen on the tributary area. After the rain ceases, the runoff begins to taper off accordingly. As illustrated in Figure 3, the hydrograph can be divided into three segments (Guo 2000B). In this study, the normalized unit hydrograph is separately derived for these three segments as follows:

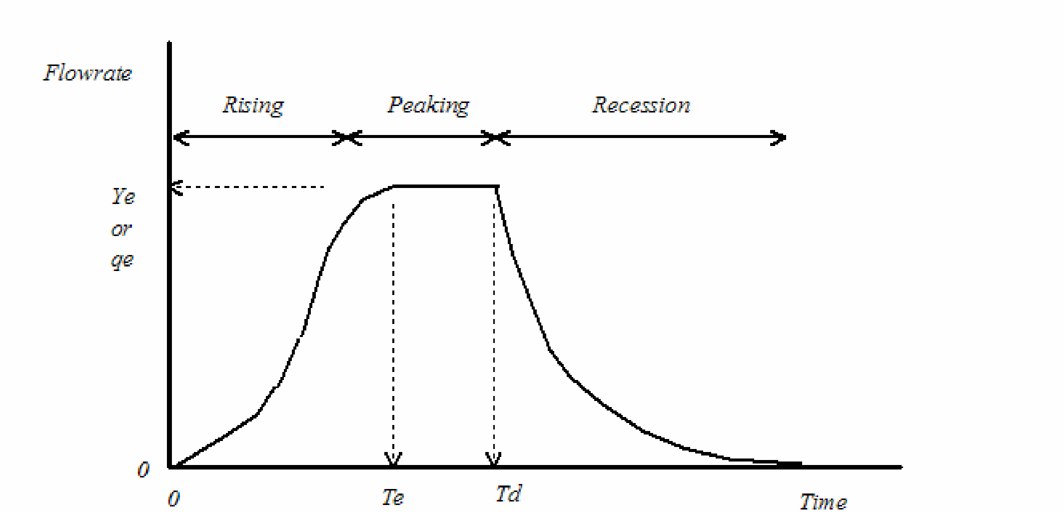


Figure 3 Hydrograph under a Long Rainfall

(1) Rising Limb ($0 \le t \le T_e \le T_d$)
At an elapsed time t, the flow depth, y(t), and its location, x, can be determined by solving Eq's 7 and 8 simultaneously. The elapsed time, t, on the rising hydrograph at x = L is normalized as:

$$T^* = \frac{t}{T_e} \qquad (13)$$

in which T^* = dimensionless elapsed time. Aided by Eq's 7 and 13, the flow depth on the rising hydrograph at x = L is derived as:

$$y^*(T^*) = \frac{y(t)}{y_e} = T^* \qquad (14)$$

in which y^* = dimensionless flow depth. Aided by Eq's 2, 12, and 13, the normalized flow rate per unit width, q^*, can be calculated as:

$$q^*(T^*) = \frac{q(t)}{q_e} = T^{*\beta} \qquad (15)$$

Eq's 14 and 15 agree with previous studies (Eagleson 1970 and Wooding 1965). They represent the rising hydrograph for flow depth as the 45-degree line between zero and one.

(2) Peaking Portion ($T_e \le t \le T_d$)

During the peaking portion, the inflow volume is equal to the outflow volume. Therefore, the normalized peak flow depth and rate are defined as:

$$y^*(T^*) = 1.0 \qquad (16)$$

$$q^*(T^*) = 1.0 \qquad (17)$$

The elapsed time used in Eq's 16 and 17 is also defined by Eq 13. Eq's 16 and 17 agree with the previous studies (Guo 2001) and represent the linear line of unity.

(3) Recession Portion ($t \geq T_d$, after rain ceases)

After the time of equilibrium, the catchment has developed the equilibrium water profile defined by the pairs of (x, y) using Eq's 7 and 8. As soon as the rain ceases, the flow depth under the equilibrium water surface profile in Figure 4 begins to propagate toward the outfall point.

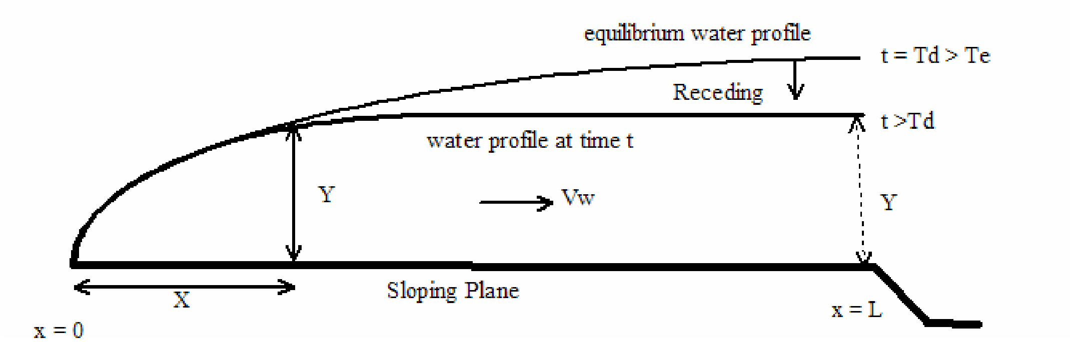


Figure 4 Receding Water Surface Profile After Rainfall Ceases

For instance, the kinematic wave under a flow depth of Y will travel through the distance from x =X to x = L during the period of time from T_d to t. Mathematically, the kinematic wave movement must satisfy:

$$L = X + V_w(t - T_d) \qquad \text{for } t>T_d>T_e \tag{18}$$

According to Eq. 8, both variables, X and L, in Eq. 18 satisfy:

$$X = \frac{\alpha Y^{\beta}}{i_e} \tag{19}$$

$$L = \frac{\alpha Y_e^{\beta}}{i_e} \tag{20}$$

$$V_w = \alpha\beta Y^{\beta-1} \tag{21}$$

Substituting Eq's 19 through 21 into Eq 18 and then normalizing the resulting equation by T_e and y_e yield:

$$T^* = \frac{1}{\beta Y^{*\beta-1}} - \frac{1}{\beta}Y^* + \frac{T_d}{T_e} \tag{22}$$

$$Y^* = \frac{Y}{y_e} \tag{23}$$

In practice, Manning's formula indicates that β=5/3. As a result, Eq 22 becomes

$$T^* = \frac{1}{1.67\, Y^{*\,0.667}} - \frac{3}{5}Y^* + \frac{T_d}{T_e} \tag{24}$$

To use Chezy's formula, β=1.5. As a result, Eq 22 becomes

$$T^* = \frac{1}{1.5\, Y^{*\,0.5}} - \frac{2}{3}Y^* + \frac{T_d}{T_e} \tag{25}$$

For a given elapsed time on the recession hydrograph at x = L, the flow depth can be calculated by solving Eq 24 or 25. The corresponding flow depth and rate on the recession limb are

$$y^*(T^*) = Y^* \tag{26}$$

$$q^*(T^*) = Y^{*\beta} \tag{27}$$

The recession hydrograph ends when the elapsed time, T^*, becomes so long that the flow depth, Y^*, vanishes.

COMPARISON WITH SCS UNITGRAPH

The SCSUH summarized in Table 1 represents the average values derived from a large number of agricultural watersheds. The time to peak and peak flow rate per sq mile are used to normalize the SCSUH. As shown in Figure 5, the base time of the SCSUH was approximately 5 times the time-to-peak. The inflection point on the recession limb is located at approximately 1.7 times the time-to-peak. As recommended, the curvilinear SCS UH can be approximated by a triangular SCSUH which has a base time as short as 8/3 time to peak.

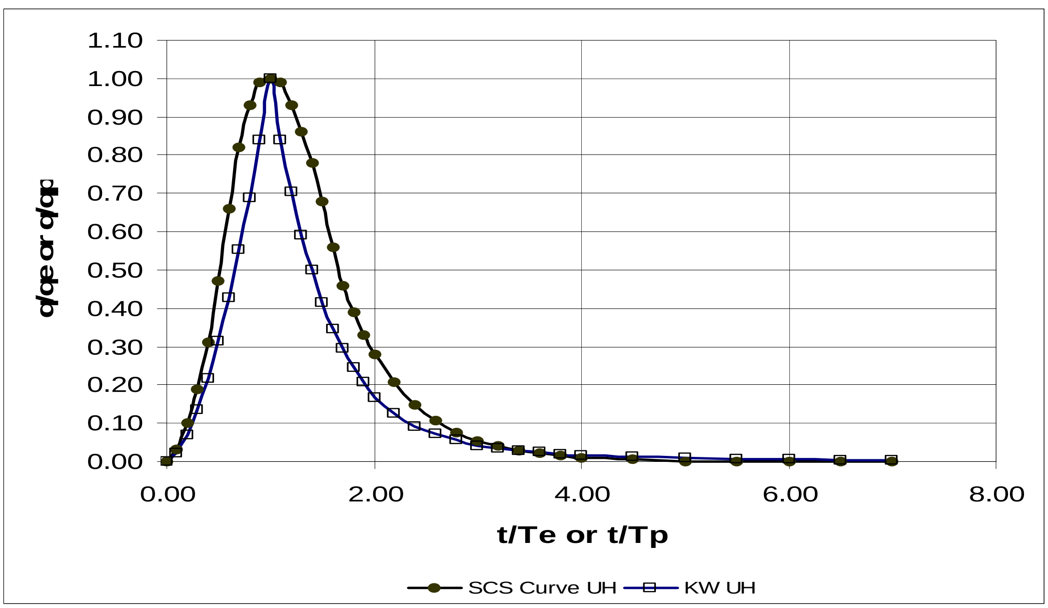


Figure 5 Comparison between SCS Curve and KW Unit Hydrographs

t/Tp or t/Te	Ordinates on Unit Graph			Ordinates on Mass Curve		
	SCS Triangle q(t)/qp	SCS Curve q(t)/qp	KW q(t)/qe	SCS Triangle	SCS Curve	KW
0.00	0.000	0.000	0.000	0.000	0.000	0.000
0.10	0.100	0.030	0.022	0.001	0.001	0.001
0.20	0.200	0.100	0.068	0.006	0.006	0.006
0.30	0.300	0.190	0.134	0.012	0.012	0.016
0.40	0.400	0.310	0.217	0.035	0.035	0.033
0.50	0.500	0.470	0.315	0.065	0.065	0.060
0.60	0.600	0.660	0.427	0.107	0.107	0.097
0.70	0.700	0.820	0.552	0.163	0.163	0.146
0.80	0.800	0.930	0.689	0.228	0.228	0.208
0.90	0.900	0.990	0.839	0.300	0.300	0.284
1.00	1.000	1.000	1.000	0.375	0.375	0.376
1.10	0.941	0.990	0.839	0.450	0.450	0.468
1.20	0.882	0.930	0.704	0.522	0.522	0.545
1.30	0.824	0.860	0.592	0.589	0.589	0.610
1.40	0.765	0.780	0.500	0.650	0.650	0.665
1.50	0.706	0.680	0.415	0.700	0.700	0.711
1.60	0.647	0.560	0.347	0.751	0.751	0.749
1.70	0.588	0.460	0.294	0.790	0.790	0.781
1.80	0.529	0.390	0.245	0.830	0.822	0.808
1.90	0.471	0.330	0.208	0.860	0.849	0.830
2.00	0.412	0.280	0.166	0.890	0.871	0.849
2.20	0.294	0.207	0.127	0.920	0.908	0.878
2.40	0.176	0.147	0.093	0.950	0.934	0.900
2.60	0.059	0.107	0.071	0.980	0.953	0.917
2.80	0.000	0.077	0.055	1.000	0.967	0.929
3.00	0.000	0.055	0.042	1.000	0.977	0.939
3.20	0.000	0.040	0.036	1.000	0.984	0.947
3.40	0.000	0.029	0.029	1.000	0.989	0.953
3.60	0.000	0.021	0.024	1.000	0.993	0.959
3.80	0.000	0.015	0.020	1.000	0.995	0.963
4.00	0.000	0.011	0.016	1.000	0.997	0.967
4.50	0.000	0.005	0.012	1.000	1.000	0.974
5.00	0.000	0.000	0.008	1.000	1.000	0.979
5.50	0.000	0.000	0.006	1.000	1.000	0.982
6.00	0.000	0.000	0.005	1.000	1.000	0.985
6.50	0.000	0.000	0.004	1.000	1.000	0.987
7.00	0.000	0.000	0.003	1.000	1.000	0.990

Table 1 Comparison between SCS and KW Unit Graphs

For comparison, a KWUH was produced using $T_d = T_e$. As a result, $T_p = T_e$, and $q_p = q_e$ for the case study. The KWUH has a base time approximately 7 times that of equilibrium. The inflection point on the recession limb of the KWUH occurs at approximately 2.28 times the time of equilibrium. Although the KWUH has a longer base time than the SCSUH, the major portions of the two recession limbs are almost parallel.

The principle of mass conservation requires that the total runoff volume be equal to the rainfall excess volume. Both volumes are calculated as:

$$V_r(t) = \int q(t)dt \approx \sum_{t=0}^{t=t} q(t)\Delta t \qquad (28)$$

$$V_i = i_e T_d L \qquad (29)$$

in which V_r (t) = cumulative runoff volume at time t, V_i = total rainfall volume, and Δt = time increment. Aided by Eq's 28 and 29, the dimensionless mass curve is derived as:

$$V^*(T^*) = \frac{V_r(t)}{V_i} = \frac{T_e}{T_d} \sum_{T^*=0}^{T^*=T^*} q^*(T^*)\Delta T^* \qquad (30)$$

in which V(T^*)= cumulative dimensionless runoff volume at t=T^*, ΔT^*= incremental dimensionless time interval used in the dimensionless unit hydrograph. For a unit hydrograph, Eq 30 must exhibit a unit volume, i.e. when $T^*= \infty$, V(∞)=1.0. As tabulated in Table 1, the KWUH mass curve has T_e/T_d =1.0 and Eq 30 reaches 0.99 at T^*=7.0. The asymptotic nature of the recess limb will reach unity when the elapsed time is long enough. To apply Eq 28 to the SCSUH, the time of equilibrium is replaced with the time to peak. In this study, it was found that for Eq 30 to reach unity on the SCSUH mass curve, the ratio of T_e/T_d must be 1.35 or greater, depending on the length of the recession limb. This may be one of the reasons why the ratio, T_p/T_d = 1.67, has been recommended when using the SCSUH. As shown in Table 1 and Figure 6, the recommended SCSUH mass curve has been adjusted to provide a volume of unity (SCS 1972). Figure 6 indicates that the runoff volume under the KWUH mass curve is distributed with 37.6% of the total runoff volume occurred before the time of equilibrium. This agrees with the SCSUH mass curve in which 37.5% of total runoff volume occurred before time of peak. However, this is a surprise conclusion because the SCSUH was supposed to reflect the nature of agricultural surface depression and detention. Comparison of two mass curves reveals that the SCSUH, in fact, inherits a nature similar to kinematic wave. This may explain why the SCSUH has been found to produce acceptable flood predictions for urban areas.

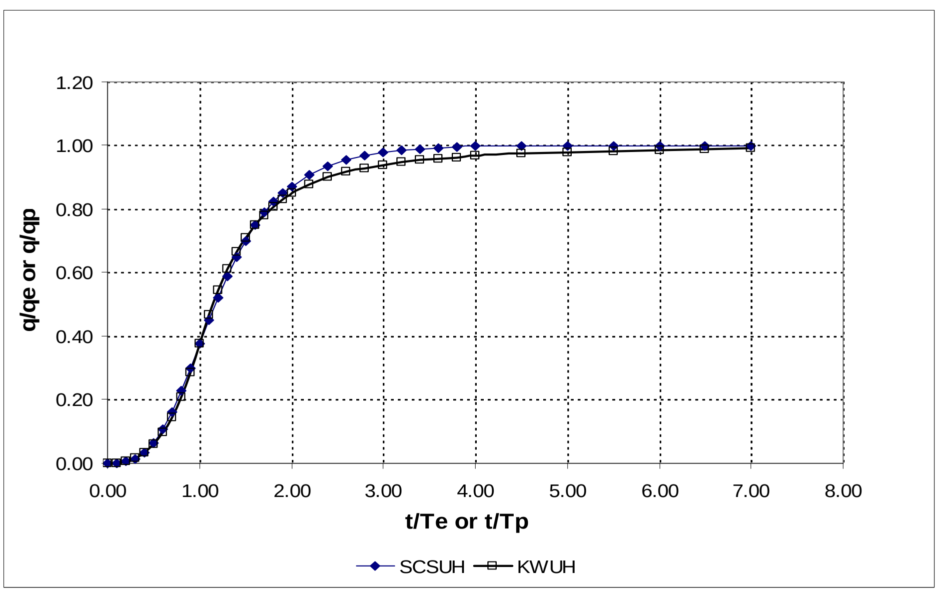


Figure 6 Comparison between SCS and KW Mass Curves of Unit Graphs

APPLICATION LIMIT OF KWUH

The Colorado Unit Hydrograph Procedure (CUHP) was developed for the Denver metropolitan area and calibrated by the field data collected from several urbanized watersheds ranging from one to five square miles (Guo and Urbonas, 1988). The empirical formulas used in the CUHP were similar to the Snyder's approach. In general, the synthetic unit hydrograph approach tends to overestimate flood flows from a small urban watershed, while the kinematic wave approach tends to underestimate the flood flows from a large watershed.

To define the applicability limit for the KWUH, a test was conducted in this study using 5 hypothetical square watersheds with areas of 20, 50, 100, 150, and 200 acres. The surface imperviousness for these five hypothetical watersheds is set to be 80%. Manning's roughness is set at 0.035 on a plane with a slope of 0.02. For this example, the values of α and β are determined by Manning's formula as:

$$\alpha = \frac{1.49}{n}\sqrt{S} = 6.02 \quad (31)$$

$$\beta=5/3 \quad (32)$$

Considering that the unit rainfall depth is one inch, Eq 10 is converted to

$$T_e = 0.0146L \quad (33)$$

The value of 0.0146 was derived for the example using minutes and feet. In this study, the 2-hr 100-year storm event developed for the State of Colorado was investigated using the computer model of CUHP-2000 (USWDCM 2001). The 2-hr Colorado rainfall distribution is similar to the most intense 2-hr period on the SCS 24-hr Type II rainfall distribution. The soil loss was described by Horton's formula for Type B soil. CUHP-2000 calculates the hydrologic abstractions and reports the distribution of rainfall excess using a five-minute time interval. For convenience, the distribution of rainfall excess is further divided into several periods of time of equilibrium. The total rainfall amount in each period is then applied to the storm hydrograph convolution process using the dimensionless KWUH in Figure 5.

For comparison, the study applies the computer models, CUHP-2000 and HEC-1 using the SCSUH (HEC-1 1985) for the hypothetic catchments. Figure 7 is the distribution of peak runoff rates versus drainage areas for three methods. As expected, the predictions by the SCSUH are consistently higher than the KWUH for the range of urban catchments. The predictions by the CUHP overlay the predictions by the KWUH for catchments between 60 and 120 acres. But, the CUHP generates higher peak flows for catchments less than 60 acres as well as for catchments greater than 120 acres. Since the CUHP has been calibrated by the field data obtained from one to five square-mile urban watersheds monitored in Denver, Colorado, this comparison suggests that the KWUH be applicable for catchments smaller than 120 acres. The applicability limit is attributed to the significant natural depression and non-uniformity of rainfall distribution when the watershed becomes larger.

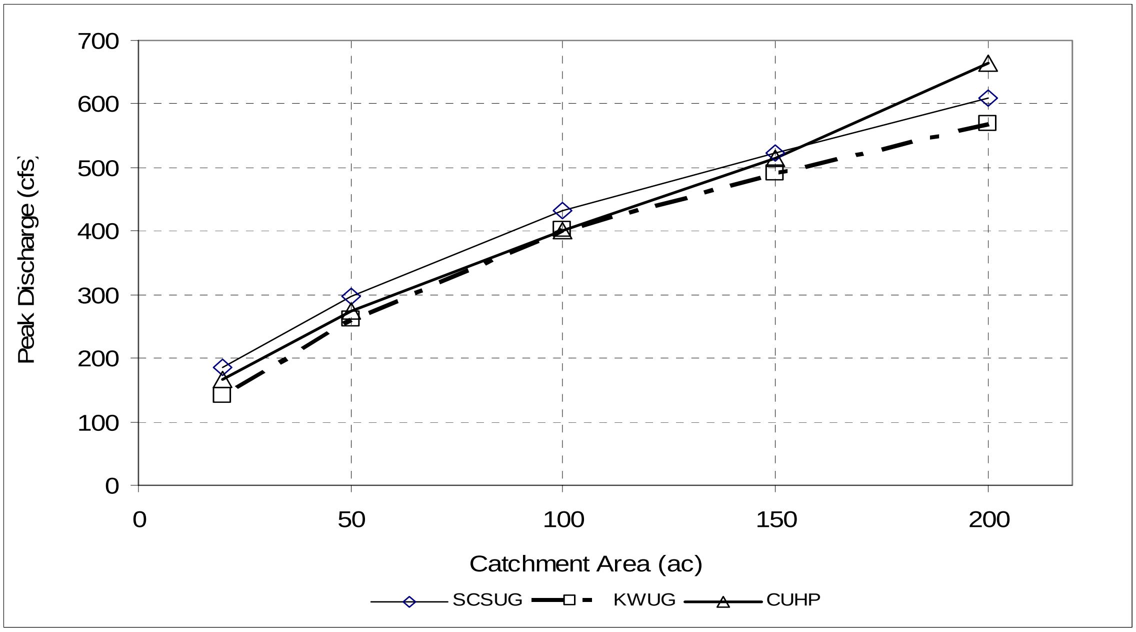


Figure 7 Peak Flow Rates Predicted by SCS, CUHP, and KW Unit Graphs

CONCLUSION

(1) This paper presents the normalized unit hydrograph derived by the kinematic wave approach. With dimensionless ordinates, the KWUH and its mass curve present a handy solution. A series of the KWUH can be derived, depending on the rainfall duration. But, comparison with the SCSUH suggests that the time of equilibrium for the rainfall duration gives good agreement with the SCS method. Eq 10 was derived to calculate the time of equilibrium under a unit rainfall depth.

(2) The KWUH was derived for urban catchments. The comparison study suggests that the applicable limit for the KWUH be up to 120 acres. The comparison with the SCSUH indicates that the SCSUH is in fact kinematic in nature.

(3) The kinematic wave approach is successful for regular or ideal watershed geometry. In practice, the question of how to preserve the drainage characteristics of an irregular watershed by its equivalent rectangular shape has not yet been sufficiently addressed yet. It is imperative to understand further how to cope with a catchment asymmetric to its collection channel, and how to relate the transverse slope on the sloping plane to the longitudinal slope along the collection channel.

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APPENDIX II. NOTATIONS

D = unit rainfall depth
L = length of the overland flow
i_e= excess rainfall intensity
q = flow rate per unit width
q_e = unit-width equilibrium flow rate
q^*= the normalized flow rate per unit width
V_r (t) = cumulative runoff volume at time t
V_i = rainfall volume
$V^*(T^*)$ = cumulative dimensionless runoff volume at $t=T^*$,
t = time variable
Δt = time increment on the hydrograph
T^* = dimensionless elapsed time
ΔT^* = dimensionless time increment
T_d = design rainfall duration
T_e = time of equilibrium
x = distance or location varied from $x=0$ to $x=L$
X = location under equilibrium profile
Y = flow depth under equilibrium profile.
y_e = equilibrium flow depth at x =L
y^* = dimensionless flow depth
y = flow depth,
α = empirical factor according to Manning's formula
β = empirical factor between 3/2 and 5/3 for practical purpose,