CHAPTER

HYDROLOGY 2

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2.1 Hydrologic Methods

Many hydrologic methods are available. The following methods are recommended and the circumstances for their use are listed in Table 2-1 below. If other methods are used they must first be calibrated to local conditions and tested for accuracy and reliability. In addition, complete source documentation must be submitted to the DeKalb County Public Works Department for approval.

The following methods have been selected for use in DeKalb County based on several considerations, including the following.

- Verification of their accuracy for local hydrologic estimates of a range of design storms.
- Availability of equations, nomographs, and computer programs.
- Use and familiarity with the methods by DeKalb County and local consulting engineers.
- Demonstrated reliability for hydrologic analysis in estimating peak flows and hydrographs.

Table 2-1 Recommended Hydrologic Methods

Method	Size Limitations[1]	Comments
Rational	0 - 25 Acres	Method can be used for estimating peak flows and the design of small sub-division type storm sewer systems. Method shall not be used for storage design and hydrograph calculations.
DeKalb	0 – 10 Acres	Method can be used for estimating peak flows and hydrographs. Method can be used for the design of all drainage structures including storage facilities.
SCS	All Sites	Method can be used for estimating peak flows and hydrographs. Method can be used for the design of all drainage structures including storage facilities. The method may be used with results calculated by hand or using documented computer programs using the TR-55 method. A Type II SCS Rainfall Distribution and Average antecedent soil moisture conditions should be used.
USGS	25 Acres to 25 Sq .Miles	Method can be used for estimating peak flows for all design applications.
USGS	128 Acres to 25 Sq.Miles	Method can be used for estimating hydrographs for all design applications.

NOTE: There are many readily available programs (such as HEC-1) that utilize these methodologies.

[1]Size limitation refers to the drainage basin for the stormwater management facility (i.e., culvert, inlet).

2.2 Symbols And Definitions

To provide consistency within this chapter as well as throughout this manual the following symbols will be used. These symbols were selected because of their wide use in hydrologic publications. In some cases the same symbol is used in existing publications for more than one definition. Where this occurs in this chapter, the symbol will be defined where it occurs in the text or equations.

Table 2-2 Symbols And Definitions

Symbol	Definition	Units
A or a	Drainage area	acres
A_f	Channel flow area	ft^2
B	Channel bottom width	ft
C	Runoff coefficient	-
C_f	Frequency factor	-
CN	SCS-runoff curve number	-
D	Depth of flow	ft
d	Time interval	hours
F	Pond and swamp adjustment factor	-
I	Runoff intensity	in./hr
I	Percent of impervious cover	%
I_a	Initial abstraction from total rainfall	in
L	Flow length	ft
n	Manning roughness coefficient	-
P	Accumulated rainfall	in
P_w	Wetted perimeter	ft
Q	Rate of runoff	cfs
q	Storm runoff during a time interval	in
q_u	Unit peak discharge	cfs
q_p	Peak rate of discharge	cfs
R or r	Hydraulic radius	ft
S or Y	Ground slope	ft/ft or %
S	Potential maximum retention	in
S or s	Slope of hydraulic grade line	ft/ft
SCS	Soil Conservation Service	-
T	Channel top width	ft
T_L or T	Lag time	hours
T_t	Travel time	hours
t_c	Time of concentration	min
V	Velocity	ft/s

2.3 Design Frequency And Rainfall

Following are the design frequencies to be used for the design of different stormwater management facilities.

Stormwater Management Facility	Design Frequency
Culverts, Open Channels and Conveyance Systems	100-year
Storage Facilities	All intensities up to and including the 100-year using reservoir routing techniques
Inlets	10-year
Erosion & Sediment Control	25-year
Water Quality	1.2 inches of rainfall

Note: All drainage system design shall be checked using the 100-year design rainfall frequency to be sure structures are not flooded or increased damage does not occur to the highway or adjacent property.

The following rainfall intensities (Table 2-3) shall be used for all hydrologic analysis.

Table 2-3 Rainfall Intensity: DeKalb County, Georgia

Storm Duration		Rainfall Intensity (in.hr.)					
hours	minutes	2-year	5-year	10-year	25-year	50-year	100-year
0	5	5.89	6.74	7.38	8.39	9.16	9.95
0	10	4.72	5.51	6.10	6.99	7.68	8.38
0	15	3.96	4.69	5.24	6.03	6.67	7.28
0	30	2.72	3.32	3.75	4.38	4.87	5.36
1	0	1.72	2.17	2.49	2.95	3.30	3.65
6	0	0.48	0.60	0.69	0.80	0.90	0.97
12	0	0.28	0.36	0.41	0.47	0.53	0.58
24	0	0.17	0.20	0.23	0.27	0.30	0.33
24-hour Volumes (inches)		4.1	4.8	5.5	6.5	7.2	7.9

2.4 Rational Method

2.4.1 Introduction

When using the rational method some precautions should be considered.

- In determining the C value (land use) for the drainage area, hydrologic analysis should take into account any changes in land use.
- Since the rational method uses a composite C value for the entire drainage area, if the distribution of land uses within the drainage basin will affect the results of hydrologic analysis, then the basin should be divided into sub-drainage basins for analysis.
- The graphs, and tables included in this section are given to assist the engineer in applying the rational method. The engineer should use good engineering judgment in applying these design aids and should make appropriate adjustments when specific site characteristics dictate that these adjustments are appropriate.

2.4.2 Equation

The rational formula estimates the peak rate of runoff at any location in a watershed as a function of the drainage area, runoff coefficient, and mean rainfall intensity for a duration equal to the time of concentration (the time required for water to flow from the most remote point of the basin to the location being analyzed). The rational formula is expressed as follows:

$$\mathbf{Q = CIA} \qquad \mathbf{(2.1)}$$

Where: Q = maximum rate of runoff (cfs)
C = runoff coefficient representing a ratio of runoff to rainfall
I = average rainfall intensity for a duration equal to the t_C (in./hr)
A = drainage area contributing to the design location (acres)

2.4.3 Infrequent Storms

The coefficients given in Table 2-5 are applicable for storms of 2-yr to 10-yr frequencies. Less frequent, higher intensity storms will require modification of the coefficient because infiltration and other losses have a proportionally smaller effect on runoff (Wright-McLaughlin 1969). The adjustment of the rational method for use with major storms can be made by multiplying the right side of the rational formula by a frequency factor C_f. The rational formula now becomes:

$$\mathbf{Q = CC_fIA} \qquad \mathbf{(2.2)}$$

The C_f values that can be used are listed below in Table 2-4. The product of C_f times C shall not exceed 1.0.

Table 2-4 Frequency Factors For Rational Formula

Recurrence Interval (years)	C_f
25	1.1
50	1.2
100	1.25

2.4.4 Time Of Concentration

Use of the rational formula requires the time of concentration (t_c) for each design point within the drainage basin. The duration of rainfall is then set equal to the time of concentration and is used to estimate the design average rainfall intensity (I) from Table 2-3. The time of concentration consists of an overland flow time to the point where the runoff enters a defined drainage feature (i.e., open channel) plus the time of flow in a closed conduit or open channel to the design point.

Figure 2-1 can be used to estimate overland flow time based on the kinematic wave formulation. For each drainage area, the distance is determined from the inlet to the most remote point in the tributary area. From a topographic map, the average slope is determined for the same distance. The runoff coefficient © is determined by the procedure described in a subsequent section of this chapter Manning's equation can also be used to estimate the velocity of runoff from the slope and hydraulic characteristics of the waterway which can then be used to estimate overland flow time. A nomograph and procedure for determining time of concentration for overland flow based on the kinematic wave theory is available in the State of Georgia Drainage Manual for Highways. Other methods and figures may be used to calculate overland flow time if approved by the DeKalb County Public Works Department.

To obtain the total time of concentration, the pipe or open channel flow time must be calculated and added to the inlet time. After first determining the average flow velocity in the pipe or channel, the travel time is obtained by dividing velocity into the pipe or channel length. Velocity can be estimated by using the nomograph shown on Figure 2-2. Note: time of concentration cannot be less than 5 minutes.

Two common errors should be avoided when calculating time of concentration - t_c. First, in some cases runoff from a portion of the drainage area which is highly impervious may result in a greater peak discharge than would occur if the entire area were considered. In these cases, adjustments can be made to the drainage area by disregarding those areas where flow time is too slow to add to the peak discharge. Second, when designing a drainage system, the overland flow path is not necessarily the same before and after development and grading operations have been completed. Selecting overland flow paths in excess of 100 feet should be done only after careful consideration.

2.4.5 Rainfall Intensity

The rainfall intensity (I) is the average rainfall rate in in./hr for a duration equal to the time of concentration for a selected return period. Once a particular return period has been selected for design and a time of concentration calculated for the drainage area, the rainfall intensity can be determined from Rainfall-Intensity-Duration data. Table 2-3 gives the data for DeKalb County. Straight-line interpolation can be used to obtain rainfall intensity values for storm durations between the values given in Table 2-3.

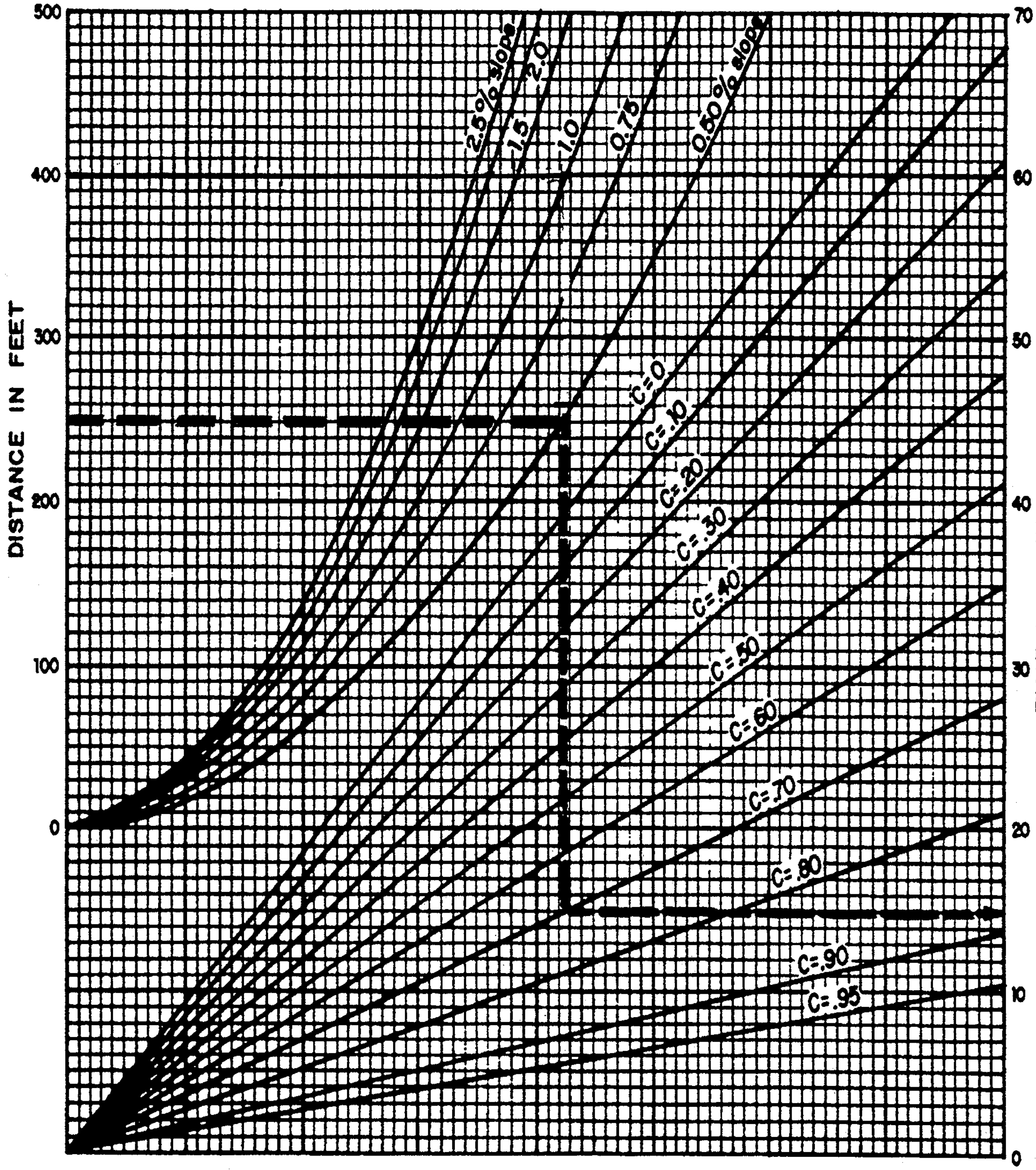
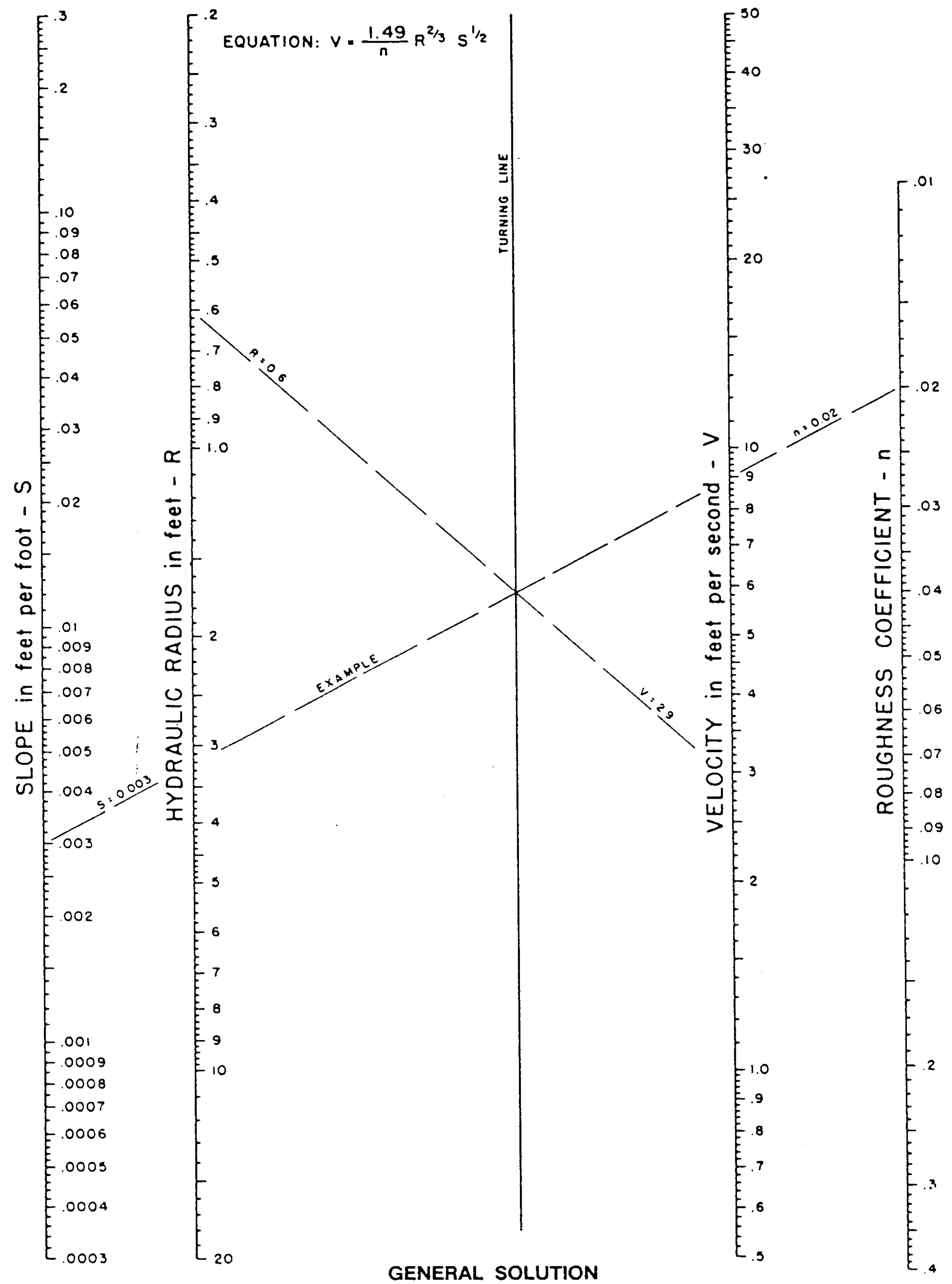


Figure 2-1
Rational Formula – Overland Time of Flow Nomograph



Reference: USDOT, FHWA, HDS-3 (1961).

Figure 2-2
Manning's Equation Nomograph

2.4.6 Runoff Coefficient

The runoff coefficient (C) is the variable of the rational method least susceptible to precise determination and requires judgment and understanding on the part of the design engineer. While engineering judgment will always be required in the selection of runoff coefficients, typical coefficients represent the integrated effects of many drainage basin parameters. Table 2-5 gives the recommended runoff coefficients for the Rational Method.

2.4.7 Composite Coefficients

It is often desirable to develop a composite runoff coefficient based on the percentage of different types of surfaces in the drainage areas. Composites can be made with the values from Table 2-5 by using percentages of different land uses. In addition, more detailed composites can be made with coefficients for different surface types such as roofs, asphalt, and concrete streets, drives and walks. The composite procedure can be applied to an entire drainage area or to typical "sample" blocks, as a guide to the selection of reasonable values of the coefficient for an entire area. It should be remembered that the rational method assumes that all land uses within a drainage area are uniformly distributed throughout the area. If it is important to locate a specific land use within the drainage area then another hydrologic method should be used where hydrographs can be generated and routed through the drainage system.

2.5 Example Problem - Rational Method

2.5.1 Introduction

Following is an example problem which illustrates the application of the Rational Method to estimate peak discharges.

2.5.2 Problem

Estimates of the maximum rate of runoff are needed at the inlet to a proposed culvert for a 100-year return period.

<u>Site Data</u>

From a topographic map and field survey, the area of the drainage basin upstream from the point in question is found to be 4.3 acres. In addition the following data were measured:

Average overland slope = 2.0%
Length of overland flow = 50 ft
Length of main basin channel = 1020 ft
Slope of channel = 1.5%
Roughness coefficient (n) of channel was estimated to be 0.08
From existing land use maps, land use for the drainage basin was estimated to be:
Business (Downtown) - 80% Graded - clay soil, 1% slope - 20%
From existing land use maps, the land use for the overland flow area at the head of the basin was estimated to be: Lawn - clay soil, 2% slope

Table 2-5 Recommended Runoff Coefficient Values

Description of Area	Runoff Coefficients (C)
Lawns:	
Sandy soil, flat, 2%	0.10
Sandy soil, average, 2 - 7%	0.15
Sandy soil, steep, > 7%	0.20
Clay soil, flat, 2%	0.17
Clay soil, average, 2 - 7%	0.22
Clay soil, steep, > 7%	0.35
Business:	
Downtown areas	0.95
Neighborhood areas	0.70
Residential:	
Single-family areas	0.50
Multi-units, detached	0.60
Multi-units, attached	0.70
Suburban	0.40
Apartment dwelling areas	0.70
Industrial:	
Light areas	0.70
Heavy areas	0.80
Parks, cemeteries	0.25
Playgrounds	0.35
Railroad yard areas	0.40
Unimproved areas (forest)	0.20
Streets:	
Asphaltic and Concrete	0.95
Brick	0.95
Drives, walks, and roofs	0.95
Gravel areas – loose compaction	0.50
Graded or no plant cover	
Sandy soil, flat, 0 - 5%	0.30
Sandy soil, flat, 5 - 10%	0.40
Clayey soil, flat, 0 - 5%	0.50
Clayey soil, average, 5 - 10%	0.60

Note: Engineering judgment should be used in adjusting runoff coefficient values for particular land uses.

Overland Flow

A runoff coefficient (C) for the overland flow area is determined from Table 2-5 to be 0.17.

Time of Concentration

From Figure 2-1 with an overland flow length of 40 ft, slope of 2.0 percent and a C of 0.10, the overland flow time is 9 min. Channel flow velocity is determined from Figure 2-2 to be 3.1 ft/s (n = 0.08, R = 1.6 and S = .015). Therefore,

$$\text{Flow Time} = \frac{1020 \text{ feet}}{(3.1 \text{ ft/s})/(60 \text{ s/min})} = 5.5 \text{ minutes}$$

and t_c = 9 + 5.5 = 14.5 min - say 15 min

Rainfall Intensity

From Table 2-3 with duration equal to 15 min,

I_{100} (100-yr return period) = 7.28 in./hr

Runoff Coefficient

A weighted runoff coefficient (C) for the total drainage area is determined in the following table by utilizing the values from Table 2-5.

Land Use	(1) Percent of Total Land Area	(2) Runoff Coefficient	(3) Weighted Runoff Coefficient*
Business (business neighborhood)	.80	.70	.56
Graded area	.20	.50	.10
Total Weighted Runoff Coefficient =			.66

*Column 3 equals column 1 multiplied by column 2.

Peak Runoff

From the rational method equation:

$$Q_{100} = C_f CIA = 1.25 \times .66 \times 7.28 \times 4.3 = 25.8 \text{ cfs}$$

Note: Be sure that C_f times C does not exceed 1.0.

This is the estimate of peak runoff for a 100-yr design storm for the given basin.

2.6 SCS Unit Hydrograph

2.6.1 Introduction

The Soil Conservation Service (NRCS) hydrologic method requires basic data similar to the Rational Method: drainage area, a runoff factor, time of concentration, and rainfall. The SCS approach, however, is more sophisticated in that it also considers the time distribution of the rainfall, the initial rainfall losses to interception and depression storage, and an infiltration rate that decreases during the course of a storm. Details of the methodology can be found in the SCS National Engineering Handbook, Section 4.

The SCS method includes the following basic steps:

1. Determination of curve numbers which represent different land uses within the drainage area.
2. Calculation of time of concentration to the study point.
3. Using the Type II rainfall distribution, total and excess rainfall amounts can be determined.
4. Using the unit hydrograph approach, triangular and composite hydrographs are developed for the drainage area.

2.6.2 Equations And Concepts

The following discussion outlines the equations and basic concepts used.

Drainage Area - The drainage area of a watershed is determined from topographic maps and field surveys. For large drainage areas it might be necessary to divide the area into sub-drainage areas to account for major land use changes, obtain analysis results at different points within the drainage area, and route flows to points of interest.

Rainfall - The SCS method applicable to DeKalb County is based on a storm event which has a Type II time distribution. To use this distribution it is necessary for the user to obtain the 24-hour rainfall volume (24 hour rainfall volumes for DeKalb County are given in Table 2-3).

Rainfall-Runoff Equation - A relationship between accumulated rainfall and accumulated runoff was derived by SCS from experimental plots for numerous soils and vegetative cover conditions. The following SCS runoff equation is used to estimate direct runoff from 24-hour or 1-day storm rainfall. The equation is:

$$\mathbf{Q = (P - 0.2S)^2 / (P + 0.8S)} \qquad \mathbf{(2.3)}$$

Where: Q = accumulated direct runoff (in.)
P = accumulated rainfall (potential maximum runoff) (in.)
S = potential maximum soil retention (in.)
S = (1000/CN) - 10 and CN = SCS curve number

Figure 2-3 shows a graphical solution of this equation. For example, 4.1 inches of direct runoff would result if 5.8 inches of rainfall occurs on a watershed with a curve number of 85.

2.6.3 Runoff Factor

The principal physical watershed characteristics affecting the relationship between rainfall and runoff are land use, land treatment, soil types, and land slope. The SCS method uses a combination of soil conditions and land-uses (ground cover) to assign a runoff factor to an area. These runoff factors, called runoff curve numbers (CN), indicate the runoff potential of an area. The higher the CN, the higher is the runoff potential. Soil properties influence the relationship between runoff and rainfall since soils have differing rates of infiltration. Based on infiltration rates, the Soil Conservation Service (SCS) has divided soils into four hydrologic soil groups.

Group A — Soils having a low runoff potential due to high infiltration rates. These soils consist primarily of deep, well-drained sands and gravels.

Group B — Soils having a moderately low runoff potential due to moderate infiltration rates. These soils consist primarily of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures.

Group C — Soils having a moderately high runoff potential due to slow infiltration rates. These soils consist primarily of soils in which a layer exists near the surface that impedes the downward movement of water or soils with moderately fine to fine texture.

Group D — Soils having a high runoff potential due to very slow infiltration rates. These soils consist primarily of clays with high swelling potential, soils with permanently high water tables, soils with a claypan or clay layer at or near the surface, and shallow soils over nearly impervious parent material.

A list of soils for DeKalb County and their hydrologic classification is presented in Table 2-6. Soil Survey maps can be obtained from local SCS (NRCS) office.

Table 2-6 Hydrologic Soils Groups For DeKalb County

Series Name	Hydrologic Group	Series Name	Hydrologic Group
Altavista	C	Louisburg	B
Appling	B	Madison	B
Cecil	B	Mecklenburg	C
Chewacla	C	Musella	B
Congree	B	Pacolet	B
Davidson	B	Red Bay	B
Gwinnett	B	Wedowee	B
Iredell	D	Wehadkee	D
Linker	B	Wickham	B
Louisa	B	Wilkes	C

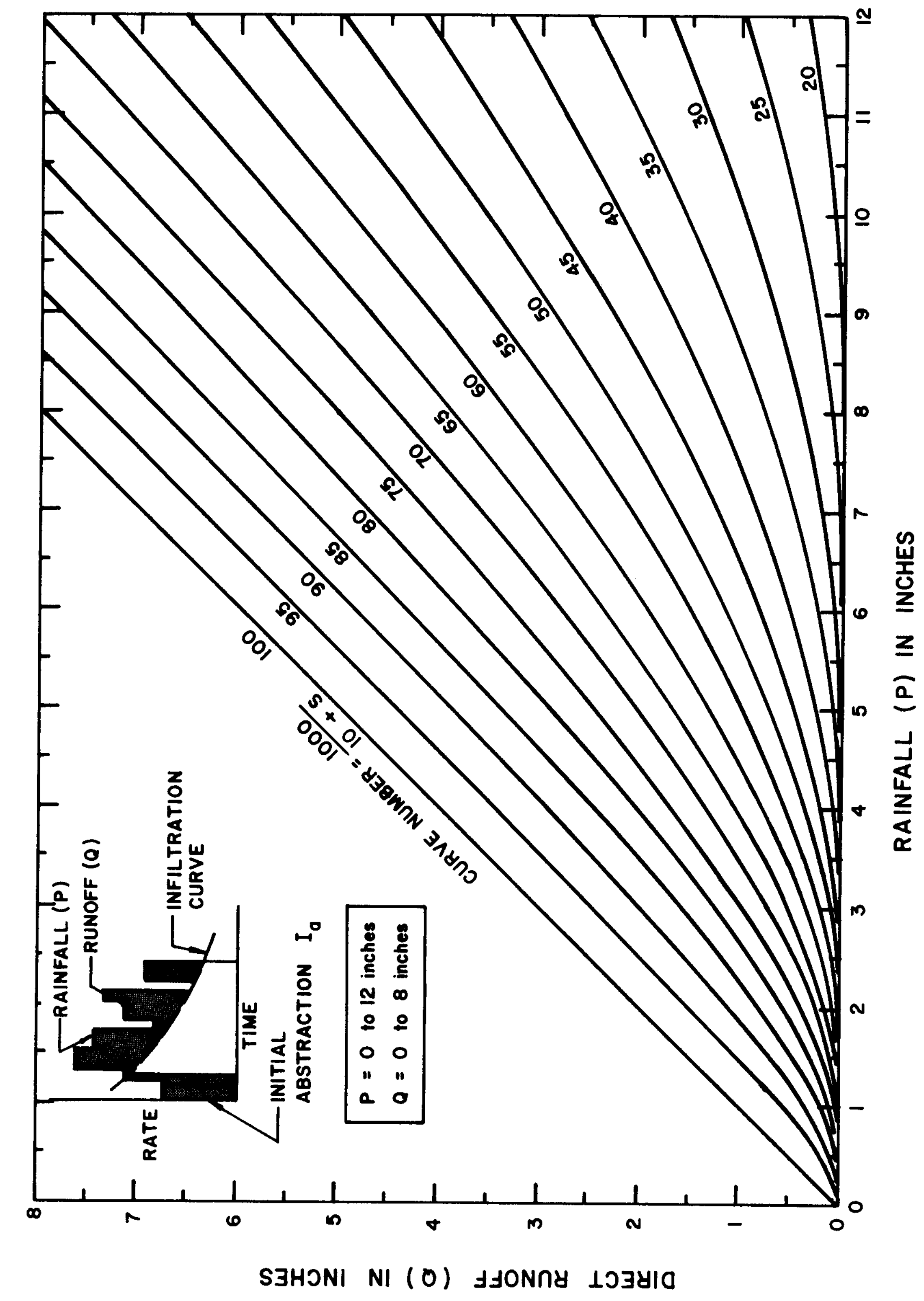


Figure 2-3
SCS Solution Of The Runoff Equation

Consideration should be given to the effects of urbanization on the natural hydrologic soil group.

If heavy equipment can be expected to compact the soil during construction or if grading will mix the surface and subsurface soils, appropriate changes should be made in the soil group selected. Also runoff curve numbers vary with the antecedent soil moisture conditions. Average antecedent soil moisture conditions (AMC II) are recommended for all hydrologic analysis, except in the design of state regulated Category I dams where AMC III may be required.

Table 2-7 gives recommended curve number values for a range of different land uses.

2.6.4 Urban Modifications

Several factors, such as the percentage of impervious area and the means of conveying runoff from impervious areas to the drainage system, should be considered in computing CN for urban areas. For example, do the impervious areas connect directly to the drainage system, or do they outlet onto lawns or other pervious areas where infiltration can occur?

The curve number values given in Table 2-7 are based on directly connected impervious area. An impervious area is considered directly connected if runoff from it flows directly into the drainage system. It is also considered directly connected if runoff from it occurs as concentrated shallow flow that runs over pervious areas and then into a drainage system. It is possible that curve number values from urban areas could be reduced by not directly connecting impervious surfaces to the drainage system. The following discussion will give some guidance for adjusting curve numbers for different types of impervious areas.

Connected Impervious Areas

Urban CN's given in Table 2-7 were developed for typical land use relationships based on specific assumed percentages of impervious area. These CN values were developed on the assumptions that:

(a) pervious urban areas are equivalent to pasture in good hydrologic condition, and
(b) impervious areas have a CN of 98 and are directly connected to the drainage system.

Some assumed percentages of impervious area are shown in Table 2-7.

If all of the impervious area is directly connected to the drainage system, but the impervious area percentages or the pervious land use assumptions in Table 2-7 are not applicable, use Figure 2-4 to compute a composite CN. For example, Table 2-7 gives a CN of 70 for a 1/2-acre lot in hydrologic soil group B, with an assumed impervious area of 25 percent. However, if the lot has 20 percent impervious area and a pervious area CN of 61, the composite CN obtained from Figure 2-4 is 68. The CN difference between 70 and 68 reflects the difference in percent impervious area.

Unconnected Impervious Areas

Runoff from these areas is spread over a pervious area as sheet flow. To determine CN when all or part of the impervious area is not directly connected to the drainage system, (1) use Figure 2-5 if total impervious area is less then 30 percent or (2) use Figure 2-4 if the total

Table 2-7 Runoff Curve Numbers[1]

Cover description	Curve numbers for hydrologic soil groups

Cover type and hydrologic condition		Average percent impervious area[2]	A	B	C	D
Cultivated land:	without conservation treatment		72	81	88	91
	with conservation treatment		62	71	78	81
Pasture or range land:	poor condition		68	79	86	89
	good condition		39	61	74	80
Meadow: good condition			30	58	71	78
Wood or forest land:	thin stand, poor cover		45	66	77	83
	good cover		25	55	70	77
Open space (lawns, parks, golf courses, cemeteries, etc.)[3]						
Poor condition (grass cover <50%)			68	79	86	89
Fair condition (grass cover 50% to 75%)			49	69	79	84
Good condition (grass cover > 75%)			39	61	74	80
Impervious areas:						
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)			98	98	98	98
Streets and roads:						
Paved; curbs and storm drains (excluding right-of-way)			98	98	98	98
Paved; open ditches (including right-of-way)			83	89	92	93
Gravel (including right-of-way)			76	85	89	91
Dirt (including right-of-way)			72	82	87	89
Urban districts:						
Commercial and business		85%	89	92	94	95
Industrial		72%	81	88	91	93
Residential districts by average lot size:						
1/8 acre or less (town houses)		65%	77	85	90	92
1/4 acre		38%	61	75	83	87
1/3 acre		30%	57	72	81	86
1/2 acre		25%	54	70	80	85
1 acre		20%	51	68	79	84
2 acres		12%	46	65	77	82
Developing urban areas and Newly graded areas (pervious areas only, no vegetation)			77	86	91	94

[1] Average runoff condition, and $I_a = 0.2S$

[2] The average percent impervious area shown was used to develop the composite CNs. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. If the impervious area is not connected, the SCS method has an adjustment to reduce the effect.

[3] CNs shown are equivalent to those of pasture. Composite CNs may be computed for other combinations of open space cover type.

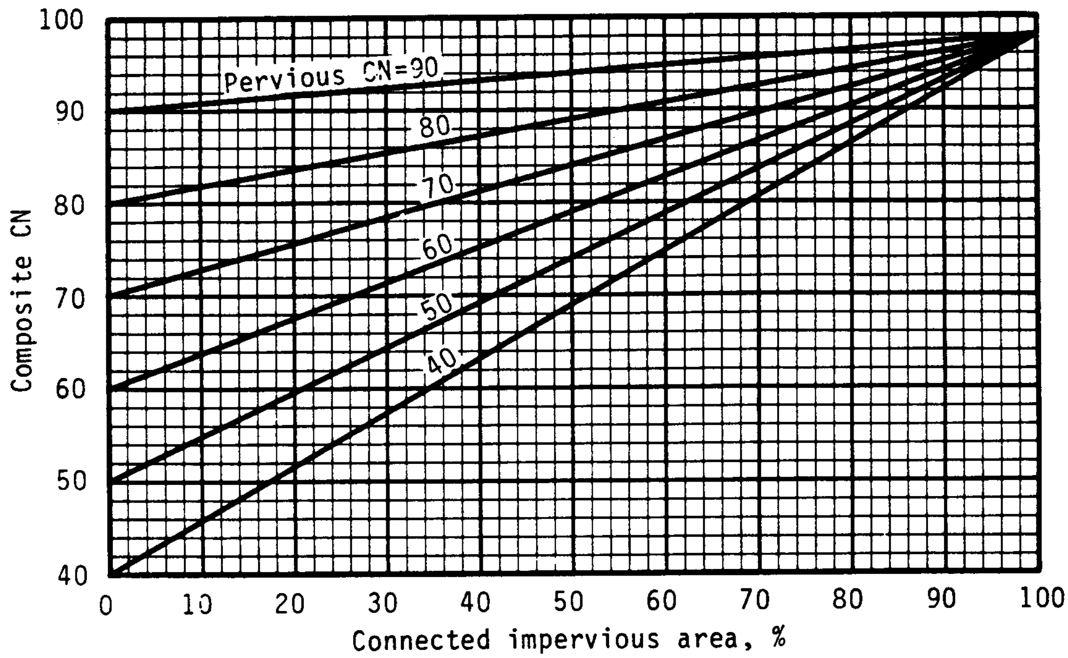


Figure 2-4 Composite CN With Connected Impervious Areas

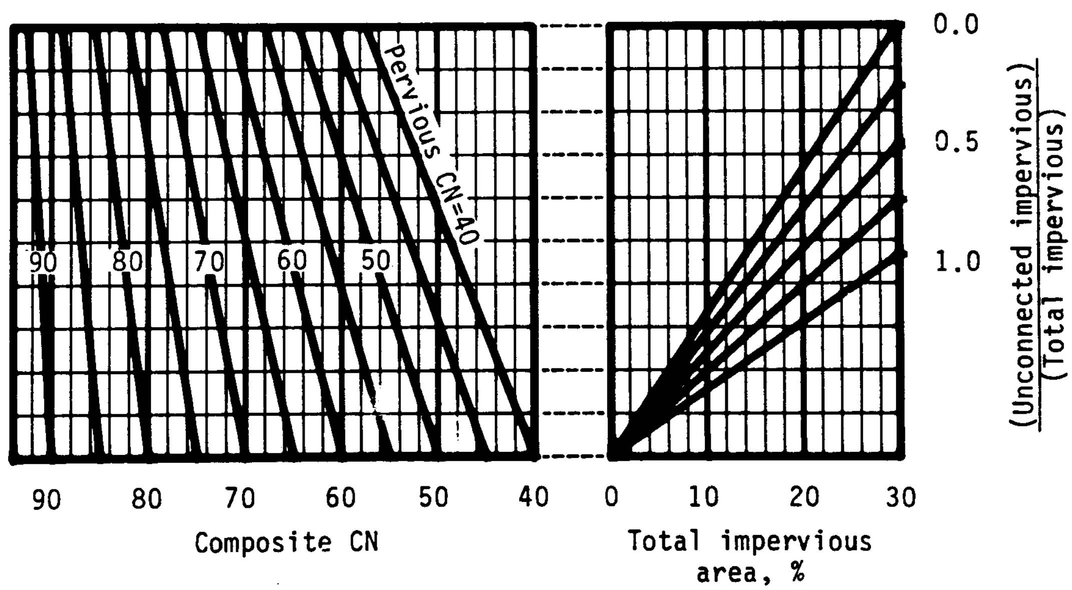


Figure 2-5 Composite CN With Unconnected Impervious Areas (Total Impervious Area Less Than 30%)

impervious area is equal to or greater than 30 percent, because the absorptive capacity of the remaining pervious areas will not significantly affect runoff.

When impervious area is less than 30 percent, obtain the composite CN by entering the right half of Figure 2-5 with the percentage of total impervious area and the ratio of total unconnected impervious area to total impervious area. Then move left to the appropriate pervious CN and read down to find the composite CN. For example, for a 1-acre lot in hydrologic soil group B with 20 percent total impervious area (75 percent of which is unconnected) and pervious CN of 61, the composite CN from Figure 2-5 is 66. If all of the impervious area is connected, the resulting CN (from Figure 2-4) would be 68.

2.6.5 Travel Time Estimation

Travel time (T_t) is the time it takes water to travel from one location to another within a watershed, through the various components of the drainage system. Time of concentration (t_c) is computed by summing all the travel times for consecutive components of the drainage conveyance system from the hydraulically most distant point of the watershed to the point of interest within the watershed. Following is a discussion of related procedures and equations (TR-55, 1986).

2.6.5.1 Travel Time

Water moves through a watershed as sheet flow, shallow concentrated flow, open channel flow, or some combination of these. The type that occurs is a function of the conveyance system and is best determined by field inspection.

Travel time is the ratio of flow length to flow velocity:

$$\mathbf{T_t = L/(3600V)} \qquad \mathbf{(2.4)}$$

Where: T_t = travel time (hr)
L = flow length (ft)
V= average velocity (ft/s)
3600 = conversion factor from seconds to hours

2.6.5.2 Sheet Flow

Sheet flow can be calculated using the following formulas:

$$\mathbf{T_t = [0.42\ (nL)^{0.8} / (P_2)^{0.5}(S)^{0.4}]} \qquad \mathbf{(2.5)}$$

Where: T_t = travel time (min),
n = Manning roughness coefficient (see Table 2-8)
L = flow length (ft),
P_2 = 2-year, 24-hour rainfall = 4.1 in., and
S = slope of hydraulic grade line (land slope ft/ft).

Table 2-8 Roughness Coefficients (Manning's n) For Sheet Flow

Surface Description	n[1]
Smooth surfaces (concrete, asphalt, gravel, or bare soil)	0.011
Fallow (no residue)	0.05
Cultivated soils:	
Residue cover 20%	0.06
Residue cover > 20%	0.17
Grass:	
Short grass prairie	0.15
Dense grasses[2]	0.24
Bermuda grass	0.41
Range (natural)	0.13
Woods[3]	
Light underbrush	0.40
Dense underbrush	0.80

[1]The n values are a composite of information by Engman (1986).

[2]Includes species such as weeping lovegrass, bluegrass, buffalo grass, blue grama grass, and native grass mixtures.

[3]When selecting n, consider cover to a height of about 0.1 ft. This is the only part of the plant cover that will obstruct sheet flow.

Source: SCS, TR-55, Second Edition, June 1986.

Substituting the constant rainfall amount the travel time equation becomes:

$$\mathbf{T_t = [0.207\ (nL)^{0.8}] / (S)^{0.4}} \qquad \textbf{(2.6)}$$

2.6.5.3 Shallow Concentrated Flow

After a maximum of 100 feet sheet flow usually becomes shallow concentrated flow. The average velocity for this flow can be determined from Figure 2-6 on the next page, in which average velocity is a function of watercourse slope and type of channel.

Average velocities for estimating travel time for shallow concentrated flow can be computed from using Figure 2-6, or the following equations. These equations can also be used for slopes less then 0.005 ft/ft.

Unpaved $\mathbf{V = 16.1345(S)^{0.5}}$ **(2.7)**

Paved $V = \mathbf{20.3282(S)^{0.5}}$ **(2.8)**

Where: V = average velocity (ft/s)
S = slope of hydraulic grade line (watercourse slope, ft/ft)

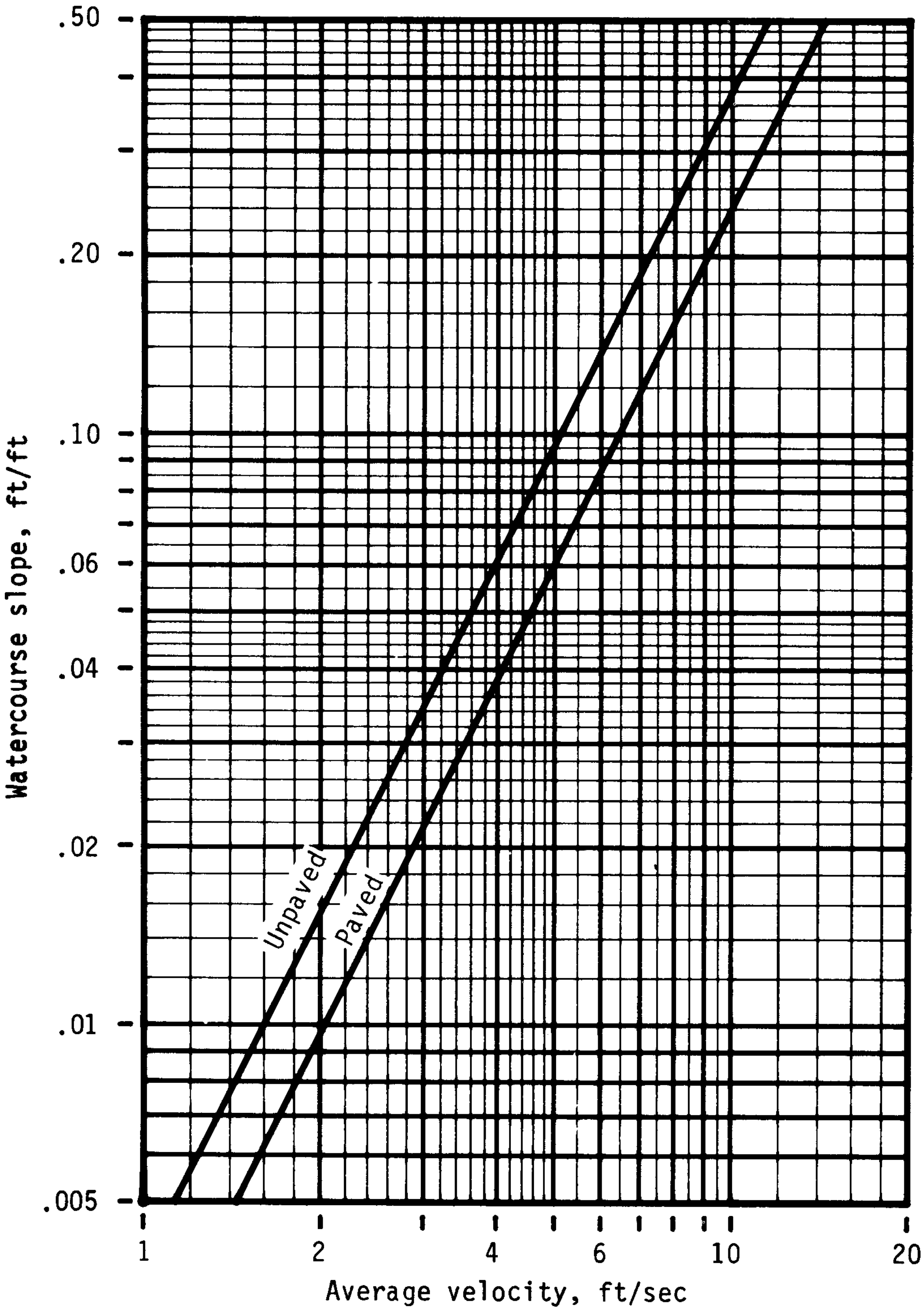


Figure 2-6
Average Velocities - Shallow Concentrated Flow

These two equations are based on the solution of Manning's equation with different assumptions for n (Manning's roughness coefficient) and r (hydraulic radius, ft) for unpaved areas, n is 0.05 and r is 0.4; for paved areas, n in 0.025 and r is 0.2.

After determining average velocity using Figure 2-6 or equations 2.7 or 2.8, travel time for the shallow concentrated flow segment can be estimated by dividing the flow length by the velocity.

2.6.5.4 Open Channels

Open channels are assumed to begin where surveyed cross section information has been obtained, where channels are visible on aerial photographs, or where blue lines (indicating streams) appear on United States Geological Survey (USGS) quadrangle sheets. Manning's equation or water surface profile information can be used to estimate average flow velocity. Average flow velocity is usually determined for bank-full elevation.

Manning's equation is $\mathbf{V = [1.49\ (r)^{2/3}\ (s)^{1/2}] / n}$ **(2.9)**

Where: V= average velocity (ft/s)
r = hydraulic radius (ft) and is equal to a/p_w
a = cross sectional flow area (ft^2)
p_w = wetted perimeter (ft)
s = slope of the hydraulic grade line (ft/ft)
n = Manning's roughness coefficient for open channel flow

After average velocity is computed using equation 2.9, T_t for the channel segment can be estimated by dividing the flow length by the velocity.

Velocity in channels should be calculated from the Manning equation. Cross sections from all channels that have been field checked should be used in the calculations. This is particularly true of areas below dams or other flow control structures.

2.6.5.5 Limitations

- Equations in this section should not be used for sheet flow longer than 100 feet.
- In watersheds with storm sewers, carefully identify the appropriate hydraulic flow path to estimate t_c.
- A culvert or bridge can act as a reservoir outlet if there is significant storage, protected by recorded easements, behind it. Detailed storage routing procedures should be used to determine the outflow through the culvert.

2.6.6 Triangular Hydrograph Equation

The triangular hydrograph is a practical representation of excess runoff with only one rise, one peak, and one recession. Its geometric makeup can be easily described mathematically, which makes it very useful in the process of estimating discharge rates, and produces results that are sufficiently accurate for most drainage facility designs. The SCS developed the following equation to estimate the peak rate of discharge for an increment of runoff:

$$\mathbf{q_p = (484\ A\ (q)) / (d/2 + T_L)} \qquad \textbf{(2.10)}$$

Where: q_p= peak rate of discharge (cfs)
A = area (mi^2)
q = storm runoff volume during time interval (in.)
d = rainfall time increment
T_L= watershed lag time ($T_L = 0.6\ t_c$) (hr)
T_p= time to peak ($d/2 + T_L$) (hr)
T_b= time of base ($2.67T_p$) (hr)

2.7 Simplified SCS Method

2.7.1 Overview

The following SCS procedures were taken from the SCS Technical Release 55 (TR-55) which presents simplified procedures to calculate storm runoff volume, peak rate of discharges and hydrographs. These procedures are applicable to small drainage areas and include provisions to account for urbanization. The following procedures outline the use of the SCS-TR 55 method.

2.7.2 Peak Discharges

The SCS peak discharge method is applicable for estimating the peak runoff rate from watersheds with a homogeneous land use. The following method is based on the results of computer analyses performed using TR-20, "Computer Program for Project Formulation - Hydrology" (SCS 1983).

The peak discharge equation is:

$$\mathbf{Q_p = q_u A Q F_p} \tag{2.11}$$

Where: Q_p = peak discharge (cfs)
q_u = unit peak discharge (cfs/mi^2/in)
A = drainage area (mi^2)
Q = runoff (in)
F_p = pond and swamp adjustment factor

The input requirements for this method are as follows:

1. T_c - hours
2. Drainage area - mi^2
3. Type II rainfall distribution
4. 24-hour design rainfall
5. CN value
6. Pond and Swamp adjustment factor (If pond and swamp areas are spread throughout the watershed and are not considered in the T_c computation, an adjustment is needed.)

2.7.3 Computations

Computations for the peak discharge method proceed as follows:

1. The 24-hour rainfall depth is determined from Table 2-3.

2. The runoff curve number, CN, is estimated from Table 2-7 and direct runoff, Q, is estimated from Figure 2-3. Determine if urban modifications of CN are appropriate.

3. The CN value is used to determine the initial abstraction, I_a, from Table 2-9, and the ratio I_a/P is then computed. (P = accumulated 24-hour rainfall or potential maximum runoff.)

4. The watershed time of concentration is computed using the procedures in Section 2.6.5 and is used with the ratio I_a/P to obtain the unit peak discharge, q_u, from Figure 2-7. If the ratio I_a/P lies outside the range shown in Figure 2-7, either the limiting values or another peak discharge method should be used.

5. The pond and swamp adjustment factor, F_p, is estimated from below:

Pond & Swamp Areas (%*)	F_p
0	1.00
0.2	0.97
1.0	0.87
3.0	0.75
5.0	0.72

*Percent of entire drainage basin

6. The peak runoff rate is computed using equation 2.11.

2.7.4 Limitations

The accuracy of the peak discharge method is subject to specific limitations, including the following.

1. The watershed must be hydrologically homogeneous and describable by a single CN value.

2. The watershed may have only one main stream, or if more than one, the individual branches must have nearly equal time of concentrations.

Table 2-9 I_a Values For Runoff Curve Numbers

Curve Number	I_a (in)	Curve Number	I_a (in)
40	3.000	70	0.857
41	2.878	71	0.817
42	2.762	72	0.778

43	2.651	73	0.740
44	2.545	74	0.703
45	2.444	75	0.667
46	2.348	76	0.632
47	2.255	77	0.597
48	2.167	78	0.564
49	2.082	79	0.532
50	2.000	80	0.500
51	1.922	81	0.469
52	1.846	82	0.439
53	1.774	83	0.410
54	1.704	84	0.381
55	1.636	85	0.353
56	1.571	86	0.326
57	1.509	87	0.299
58	1.448	88	0.273
59	1.390	89	0.247
60	1.333	90	0.222
61	1.279	91	0.198
62	1.226	92	0.174
63	1.175	93	0.151
64	1.125	94	0.128
65	1.077	95	0.105
66	1.030	96	0.083
67	0.985	97	0.062
68	0.941	98	0.041
69	0.899		

3. Hydrologic routing cannot be considered.

4. The pond and swamp adjustment factor, F_p, applies only to areas located away from the main flow path.

5. Accuracy is reduced if the ratio I_a/P is outside the range given in Figure 2-7.

6. Weighted CN value must be greater than or equal to 40 and less than or equal to 98.

7. The same procedure should be used to estimate existing and developed time of concentration when computing existing and developed peak discharge.

8. The watershed time of concentration must be between 0.1 and 10 hours.

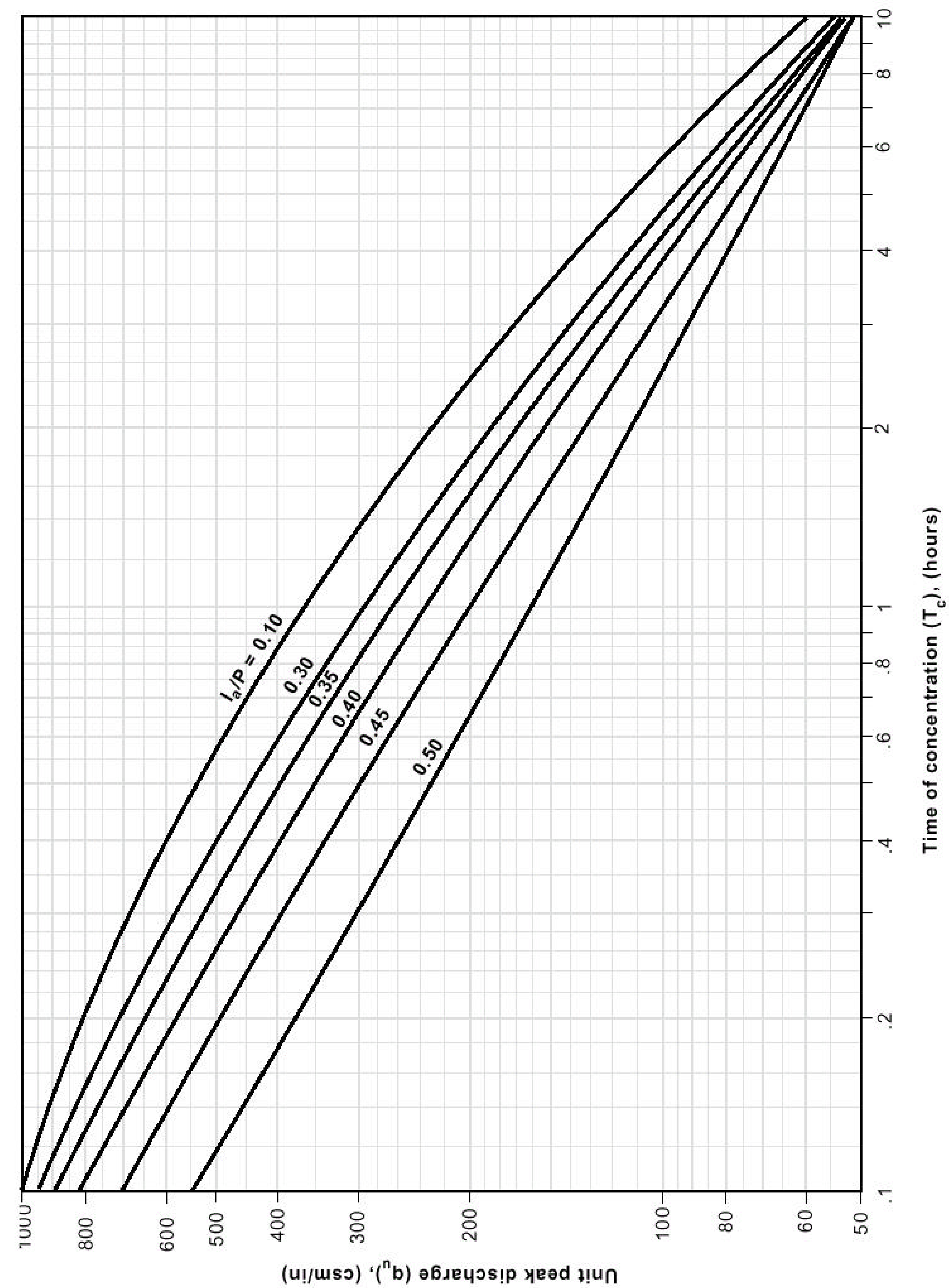


Figure 2-7 SCS Type II Unit Peak Discharge Graph

2.7.5 Example Problem

Compute the 100-year peak discharge for a 50-acre wooded watershed which will be developed as follows:

1. Forest land - good cover (hydrologic soil group B) = 8 ac.
2. Forest land - good cover (hydrologic soil group C) = 10 ac.
3. Town house residential (hydrologic soil group C) = 22 ac.
4. Industrial development (hydrological soil group C) = 10 ac.

Other data include: percentage of pond and swamp area = 0.2%.

Computations

1. Calculate rainfall excess:

- The 100-year, 24-hour rainfall is 7.9 inches.
- Composite weighted runoff coefficient is:

Dev. #	Area	% Total	Cn	Composite Cn
1	8 ac.	0.16	55	8.8
2	10 ac.	0.20	70	14.0
3	22 ac.	0.44	90	40.0
4	10 ac.	0.20	91	18.2
Total	50 ac.	1.00	81	

Assume all impervious areas are directly connected so urban modifications are not justified.

* From Figure 2-3, Q = 5.7 inches

2. Calculate time of concentration

- The hydrologic flow path for this watershed = 1,890 ft.

Segment	Type of Flow	Length	Slope (%)
1	Overland n = 0.24	40 ft.	2.0 %
2	Shallow channel	750 ft.	1.7 %
3	Main channel*	1100 ft.	0.50 %

* For the main channel, n = .06 (estimated), width = 10 feet, depth = 2 feet, rectangular channel. R = Cross-Sectional Area/Wetted Perimeter = 20/14 = 1.43 feet

- Segment 1 - Travel time from equation 2.6 with P_2 = 4.1 in.

$$T_t = [0.207(0.24 \text{ X } 40)^{.8}] / (0.02)^{.4}$$

T_t = 6.05 minutes

- Segment 2 - Travel time from Figure 2-6 or equation 2.7
  V = 2.1 ft/sec (from equation 2.7)
  T_t = 750 / 60 (2.1) = 5.95 minutes

- Segment 3 - Using equation 2.9
  V = (1.49/.06) (1.43)$^{.67}$ (.005)$^{.5}$ = 2.23 ft/sec
  T_t = 1100 / 60 (2.23) = 8.22 minutes

- T_c = 6.05 + 5.95 + 8.22 = 20.2 minutes (0.34 hours)

3. Calculate I_a/P for Composite Cn = 77, I_a = .597 (Table 2-9)
   I_a/P = (.597 / 7.9) = .08

   (Note: Use I_a/P = .10 – minimum given in Figure 2-7)

4. Estimate unit discharge q_u from Figure 2-7 = 650 cfs

5. Calculate peak discharge with F_p = 0.97 using equation 2.11
   Q_{25} = 650 (50/640) (5.4) (0.97) = 266 cfs.

2.7.6 Hydrograph Generation

In addition to estimating the peak discharge, the SCS method can be used to estimate the entire hydrograph from a drainage area. The Soil Conservation Service has developed a Tabular Hydrograph procedure that can be used to generate the hydrograph for small drainage areas (less than 2000 acres). The Tabular Hydrograph procedure uses unit discharge hydrographs that have been generated for a series of time of concentrations. In addition, SCS has developed hydrograph procedures to be used to generate composite flood hydrographs. For the development of a hydrograph from a homogeneous developed drainage area and drainage areas which are not homogeneous, where hydrographs need to be generated from sub-areas and then routed and combined at a point downstream, the engineer is referred to the procedures outlined by the SCS in the 1986 version of TR-55 available from the National Technical Information Service in Springfield, Virginia 22161. The catalog number for TR-55, "Urban Hydrology for Small Watersheds," is PB87-101580.

Since most local engineers use computer programs to generate hydrograph analysis using the SCS Method, detailed hand procedures for hydrograph generation are not presented in this manual. The above reference gives the details of hydrograph generated for those who want a hand-calculated procedure.

2.8 DeKalb County Dimensionless Hydrograph

One of the deficiencies of the Rational Method is that is produces only a peak rate of runoff and not a complete flood hydrograph. A method is presented here for fabricating a flood hydrograph based on the Rational Formula and the unit hydrograph theory.

The method assumes that the land uses in the watershed are homogeneously distributed. If a large portion of a residential watershed is either all commercial or woodland, or another non-residential land use, the watershed should be subdivided into homogeneous units and hydrographs prepared using methods other than the Rational Formula. The method is accepted by DeKalb County for calculating hydrographs from drainage less than 10 acres.

A design storm can be developed from the rainfall intensity data given in Table 2-3 by computing incremental rainfall intensities and arranging them to form a storm pattern. The sequence of occurrence of the blocks of rainfall intensity will be to place the most intense increment at the center of the storm and to place the next highest increments on alternative side of the peak, to form a pyramid rainfall patters, as shown in Figure 2-8.

By following this procedure for several durations, two dimensionless hydrographs were developed based on the frequency-duration-intensity data used in DeKalb County.

After the peak discharge rate for a watershed is calculated by the Rational Formula or SCS Method, the dimensionless hydrographs shown in Figure 2-9 can be used to obtain a flood hydrograph. If the dimensionless hydrographs are used, the volume of precipitation excess may not agree exactly with the volume calculated using the runoff coefficient since the curves in Figure 2-9 are average curves. The procedure can be used to develop a hydrograph for the particular time of concentration desired to avoid this discrepancy introduced by averaging.

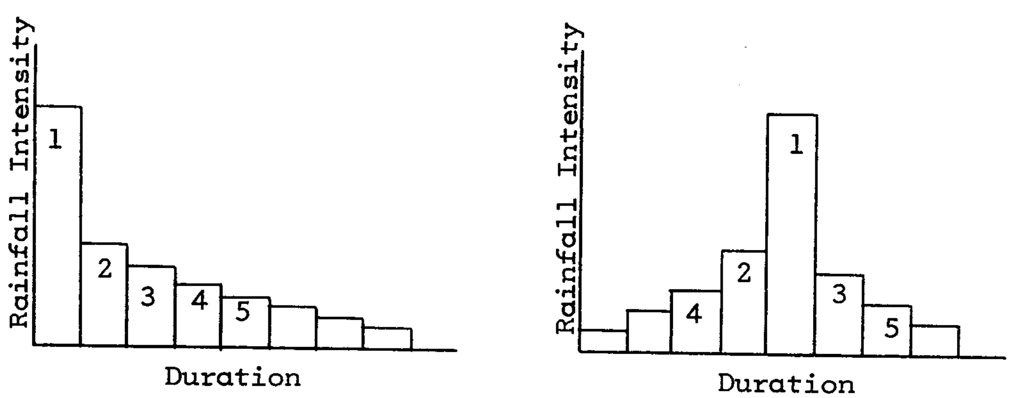


Figure 2-8 Rainfall Distribution for DeKalb County Dimensionless Hydrograph

t/T_c	1	2	3	4	5	6	7	8	9	10
Hydrograph A	0.16	0.19	0.27	0.45	1.00	0.34	0.27	0.19	0.12	0.0
Hydrograph B	0.04	0.08	0.16	0.32	1.00	0.30	0.11	0.05	0.03	0.0

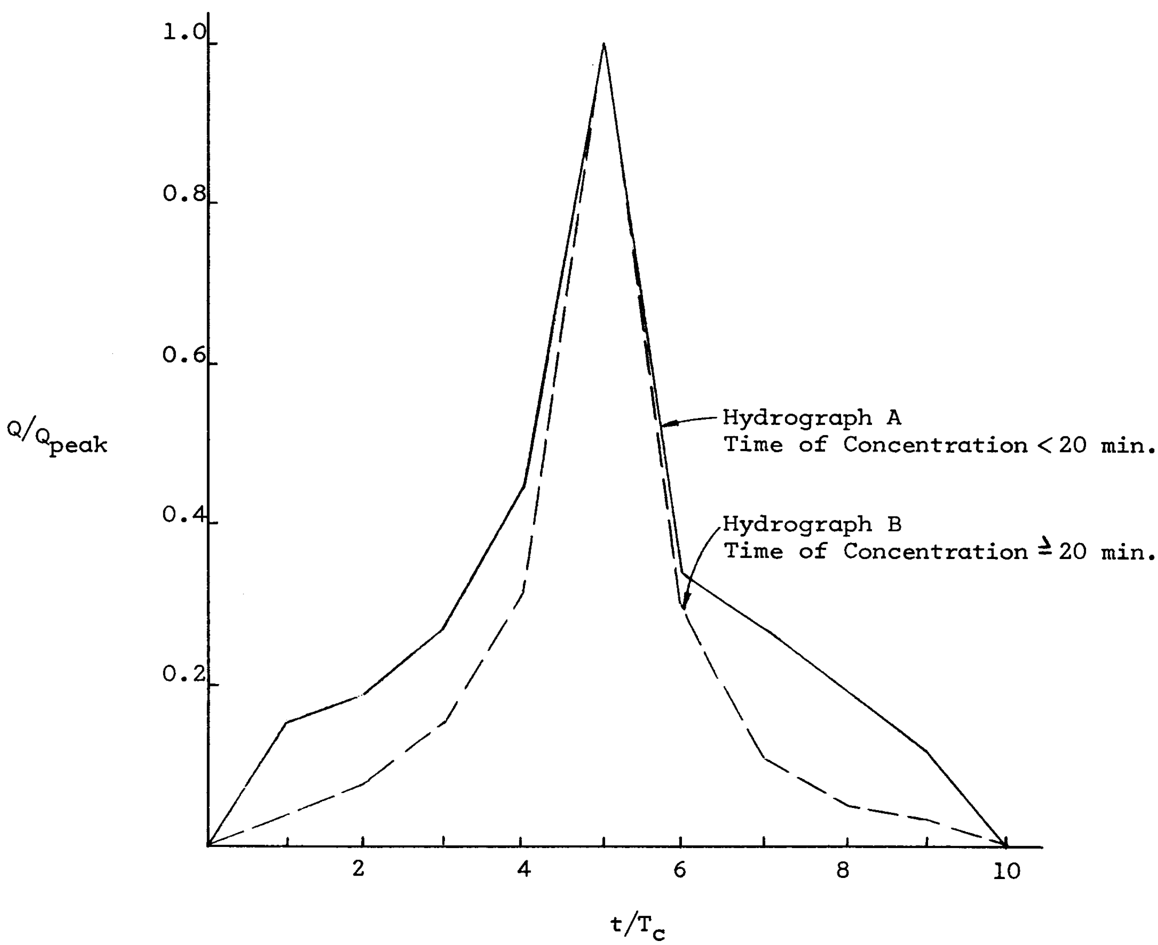


Figure 2-9 DeKalb County Dimensionless Hydrograph

Example Problem

Given the following data develop a runoff hydrograph using the DeKalb County Dimensionless Hydrograph.

Q_5 = 28 cfs
Time of Concentration = 17 minutes
Use Hydrograph A in Figure 2-9.

T/T_c	t (minutes)	Q/Q peak	Q (cfs)
0	0	0	0
1	17	0.16	4.5
2	34	0.19	5.3
3	51	0.27	7.6
4	68	0.45	12.6
5	85	1.00	28.0
6	102	0.34	9.5
7	119	0.27	7.6
8	136	0.19	5.3
9	153	0.12	3.4
10	170	0	0

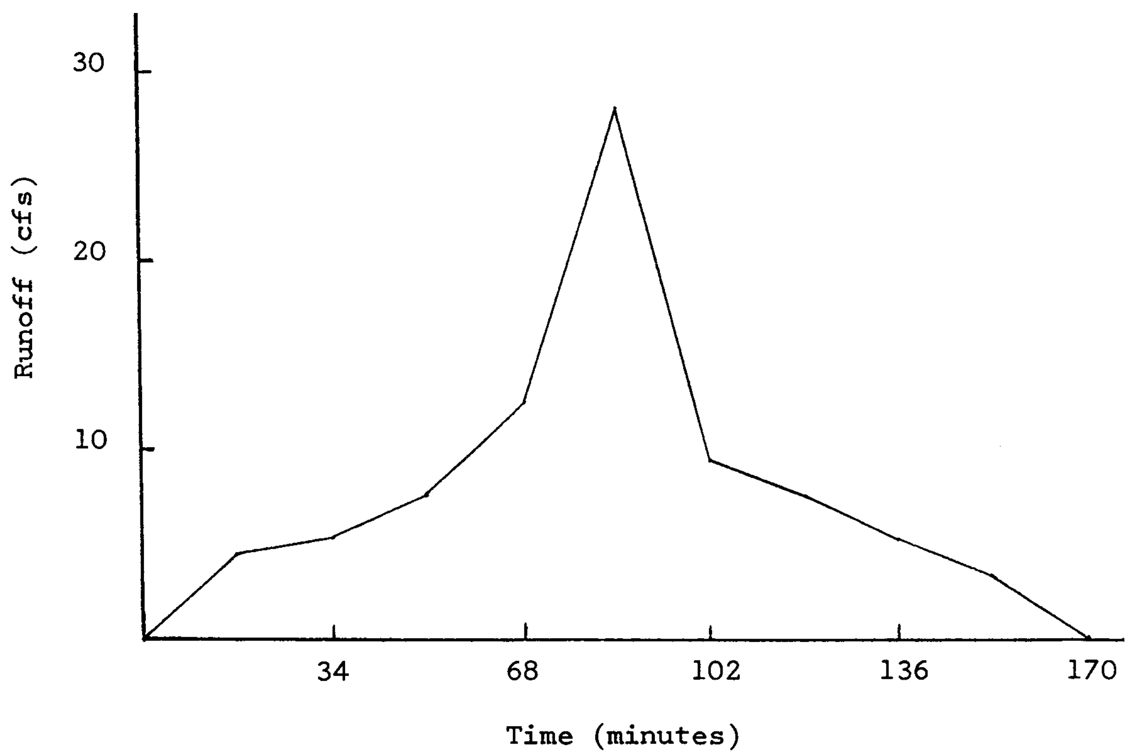


Figure 2-10 Example Problem Hydrograph

2.9 U.S. Geological Survey Peak Flow And Hydrograph Method

2.9.1 Introduction

For the past 20 years the U.S. Geological Survey has been collecting rain and streamflow data at various sites within the Atlanta Metropolitan Area and throughout the State of Georgia. The data from these efforts have been used to calibrate a U.S. Geological Survey rainfall-runoff model for use within the Atlanta Area. The U.S. Geological Survey Model was then used to develop peak discharge regression equations for the 2-, 5-, 10-, 25-, 50-, 100-, 200- and 500-year floods. In addition, the USGS used the statewide data-base to develop a dimensionless hydrograph which can be used to simulate flood hydrographs from rural and urban streams within DeKalb County.

2.9.2 Peak Discharge Equations

For a complete description of the USGS regression equations presented below, consult the USGS publication "Flood-Frequency Relations For Urban Streams In Georgia - 1994 Update, Water-Resources Investigation Report 95-4017. Following are the USGS regression equations for use in DeKalb County. Note that drainage areas in Region 1 flow to the Chattahoochee River while areas in Region 2 flow to the south away from the Chattahoochee River.

Frequency	Equation	
	Region 1	Region 2
2-year	$Q_2 = 167A^{0.73}TIA^{0.31}$	$Q_2 = 145A^{0.70}TIA^{0.31}$
5-year	$Q_5 = 301A^{0.71}TIA^{0.26}$	$Q_5 = 258A^{0.69}TIA^{0.26}$
10-year	$Q_{10} = 405A^{0.70}TIA^{0.21}$	$Q_{10} = 351A^{0.70}TIA^{0.21}$
25-year	$Q_{25} = 527A^{0.70}TIA^{0.20}$	$Q_{25} = 452A^{0.70}TIA^{0.20}$
50-year	$Q_{50} = 643A^{0.69}TIA0^{.18}$	$Q_{50} = 548A^{0.70}TIA0^{.18}$
100-year	$Q_{100} = 762A^{0.69}TIA^{0.17}$	$Q_{100} = 644A^{0.70}TIA^{0.17}$
200-year	$Q_{200} = 892A^{0.68}TIA0^{.16}$	$Q_{200} = 747A^{0.70}TIA0^{.16}$
500-year	$Q_{500} = 1063A^{0.68}TIA^{0.14}$	$Q_{500} = 888A^{0.70}TIA^{0.14}$

For these equations: A = drainage area, mi^2 TIA = total impervious area, %

2.9.3 Limitations

Following are the limitations of the variables within the peak discharge equations. These equations should not be used on drainage areas which have physical characteristics outside the limits listed below.

Physical Characteristics	Minimum	Maximum	Units
A - Drainage Area	0.04	19.1	mi^2
TIA - Total Impervious Area	1.00	62	percent

2.9.4 Hydrographs

The USGS has developed a dimensionless hydrograph for Georgia streams having drainage areas of less than 500 mi^2. This dimensionless hydrograph can be used to simulate flood hydrographs for rural and urban streams within DeKalb County. For a complete description of the USGS dimensionless hydrograph consult the USGS publication "Simulation Of Flood Hydrographs For Georgia Streams", Water-Resources Investigation Report 86-4004. Following are the time and discharge ratios for the dimensionless hydrograph for DeKalb County.

Time Ratio (t/T_L)	Discharge Ratio (Q/Q_p)	Time Ratio (t/T_L)	Discharge Ratio (Q/Q_p)
0.25	0.12	1.35	0.62
0.30	0.16	1.40	0.56
0.35	0.21	1.45	0.51
0.40	0.26	1.50	0.47
0.45	0.33	1.55	0.43
0.50	0.40	1.60	0.39
0.55	0.49	1.65	0.36
0.60	0.58	1.70	0.33
0.65	0.67	1.75	0.30
0.70	0.76	1.80	0.28
0.75	0.84	1.85	0.26
0.80	0.90	1.90	0.24
0.85	0.95	1.95	0.22
0.90	0.98	2.00	0.20
0.95	1.00	2.05	0.19
1.00	0.99	2.10	0.17
1.05	0.96	2.15	0.16
1.10	0.92	2.20	0.15
1.15	0.86	2.25	0.14
1.20	0.80	2.30	0.13
1.25	0.74	2.35	0.12
1.30	0.68	2.40	0.11

The lagtime equation that should be used in DeKalb County to use the dimensionless hydrograph is:

North of the Fall Line (rural):

$$\mathbf{T_L = 4.64A^{0.49}S^{-0.21}} \qquad \textbf{(2.12)}$$

South of the Fall Line (rural):

$$\mathbf{T_L = 13.6A^{0.43}S^{-0.31}} \qquad \textbf{(2.13)}$$

Regions 1 and 2 (urban):

$$\mathbf{T_L = 7.86A^{0.35}TIA^{-0.22}S^{-0.31}} \qquad \textbf{(2.14)}$$

Where: T_L = lagtime (hours)
A = drainage area (mi^2)
S = main channel slope (ft/mi) – based on the elevations at the 10% and 85%

locations within the drainage area.
TIA = total impervious area (percent)

2.9.5 Limitations

Following are the limitations of the variables within the lagtime equations. The lagtime equation should not be used for drainage areas that have physical characteristics outside these limits.

Physical Characteristics	Minimum	Maximum	Units
North of the Fall Line (rural)			
A - Drainage Area	0.3	500	mi^2
S - Main Channel Slope	5.0	200	feet per mile
South of the Fall Line (rural)			
A - Drainage Area	0.2	500	mi^2
S - Main Channel Slope	1.3	60	feet per mile
Regions 1 and 2 (urban)			
A - Drainage Area	0.04	19.1	mi^2
S - Main Channel Slope	9.4	772.0	feet per mile
TIA - Total Impervious Area	1.0	61.6	percent

2.9.6 Rural Basins

The USGS has recently revised the equation for estimating peak discharges for rural basins. For a complete discussion of the development of these equations consult the USGS publication "Techniques For Estimating Magnitude And Frequency Of Floods In Rural Basins Of Georgia", Water-Resources Investigations Report 93-4016. For these rural equations, the USGS has two regions (Region 1 and 2) that will affect the use of the equations within DeKalb County. Region 1 includes all those drainage areas that eventually flow into the Chattahoochee River. Following are the equations used to calculate peak discharges for rural basins in DeKalb County.

Frequency	Equations Region 1	Equations Region 2
Q_2	$207A^{.654}$	$182A^{.622}$
Q_5	$357A^{.632}$	$311A^{.616}$
Q_{10}	$482A^{.619}$	$411A^{.613}$
Q_{25}	$666A^{.605}$	$552A^{.610}$
Q_{50}	$827A^{.595}$	$669A^{.607}$
Q_{100}	$1010A^{.584}$	$794A^{.605}$
Q_{200}	$1220A^{.575}$	$931A^{.603}$
Q_{500}	$1530A^{.563}$	$1130A^{.601}$

A - Drainage Area in mi^2

2.9.7 Limitations

Following are the limitations associated with the rural basin equations given above.

Physical Characteristics	Minimum	Maximum	Units
Region 1			
A - Drainage Area	0.17	730	mi^2
Region 2			
A - Drainage Area	0.10	3,000	mi^2

2.9.8 Example Problem

For the 100-year flood, calculate the peak discharge for rural and developed conditions for the following drainage area located in Region 1.

Drainage Area = 175 acres = 0.273 mi^2
Total Impervious Area (TIA) = 32%

100-year Rural Peak Discharge

$Q_{100} = 1010A^{.584} = 1010(.273)^{.584} = 473$ cfs

100-year Developed Peak Flow

$Q_{100} = 762A^{.69}TIA^{.17}$

$Q_{100} = 762(.273)^{.69}(32)^{.17} = 561$ cfs

2.10 Downstream Hydrologic/Hydraulic Analysis (10% Analysis)

It is recognized in DeKalb County that the effects of urban development and stormwater management facilities can extend downstream beyond the discharge point of a development. To account for this in the design of stormwater management facilities the County has established the following policy.

> In determining downstream effects from stormwater management structures, BMPs, and the development, engineering studies shall extend downstream to a point where the proposed development represents less than ten (10) percent of the total watershed. The results of the extended downstream point analysis (10 percent point) shall be included in the engineering studies submitted to the County.

Typical steps in the application of the ten percent downstream analysis rule would include the following.

1. A visual inspection of the downstream drainage system to the 10 percent point shall be done in the field. The results of this inspection would be to document all conditions within the downstream drainage system which would cause a significant impact on the hydraulic characteristics associated with the development site discharge hydrograph moving through the system (e.g., major constrictions, undersized stormwater management facilities, existing

development located near the drainage system).

2. If the visual inspection indicates in the engineer's/designer's professional opinion that the design storm hydrograph from the development site can be discharged through the downstream drainage system without causing drainage or flooding problems or adversely impacting existing development, then this would conclude the 10 percent analysis study. These findings would be documented in a report that would be included in the engineering studies submitted to the County.

3. If a visual analysis indicates that further analysis is needed then the engineer/designer will determine what hydrologic/hydraulic analysis is needed to document the conclusions that will be included in the final report (e.g., backwater analysis, hydrograph routing, survey information). For all analysis, existing storage facilities that in the judgment of the engineer/designer will have a significant impact on the hydrologic/hydraulic analysis should be accounted for in the impact analysis.

4. If the hydrologic/hydraulic analysis indicated that drainage or flooding problems will be caused or increased by the discharge hydrograph from the development site, measures shall be included in the development site to mitigate the adverse impacts (e.g., on-site extended detention, obtain flow easements from downstream developments, eliminate constrictions in downstream drainage system).

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CHAPTER

HYDROLOGY 2