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2.0 Hydrology

Each development designed with the following hydrologic methods needs to have a hydrology study submitted with the corresponding plans in Spalding County. The basic methodology presented below was taken from several sources including Urban Hydrology for Small Watersheds Technical Release 55 and the Georgia Stormwater Management Manual.

2.1 Introduction to Hydrologic Methods

Hydrology is the scientific study of the properties, distribution, and effects of water on the earth's surface, in the soil and underlying rocks, and in the atmosphere. The areas focused on in this chapter of hydrology are estimating rainfall that falls on the site, peak flow rates, and time distributions of stormwater runoff. The breakdown and calculation of these factors are basic essentials to the design of stormwater management facilities, such as storm drainage systems and structural stormwater controls. During hydrologic analysis, there are a variety of factors that affect the nature of stormwater runoff from a development site. Some of the parameters that need to be contemplated include:

- Rainfall amount and storm distribution
- Drainage area size, shape and orientation
- Ground cover and soil type
- Slopes of terrain and stream channel(s)
- Antecedent moisture condition
- Storage potential (floodplains, ponds, wetlands, reservoirs, channels, etc.)
- Watershed development potential
- Characteristics of the local drainage system

There are many empirical hydrologic methods that can be used to estimate stormwater runoff for a site or drainage sub basin; however, the following methods presented in this section have been selected to support hydrologic site analysis for the design methods and procedures included in the Spalding County Stormwater Manual:

1. Rational Method
2. SCS Unit Hydrograph Method

These hydrologic methods were chosen based upon an affirmation of their accuracy in replicating local hydrologic estimates for a variety of design storms throughout Spalding County and the availability of equations, nomographs, and computer programs to support the methods.

Table 2.1-1 lists the hydrologic methods and the circumstances for their use in various analysis and design applications. Table 2.1-2 provides some limitations on the use of several methods.

Table 2-1 Applications of the Recommended Hydrologic Methods

Method	Manual Section	Rational Method	SCS Method	USGS Equations
Channel Protection Volume (C_{pv})	1.3		✓	
Overbank Flood Protection (Q_{p25})	1.2		✓	✓
Extreme Flood Protection (Q_f)	1.2		✓	✓
Storage Facilities	2.2		✓	✓
Outlet Structures	6.5		✓	✓
Gutter Flow and Inlets	3.3	✓		
Storm Drain Pipes	3.7	✓	✓	✓
Culverts	4.0	✓	✓	✓
Small Ditches	4.4	✓	✓	✓
Open Channels	5.0		✓	✓
Energy Dissipation	7.0		✓	✓

Table 2-2 Constraints on Using 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 site or subdivision storm sewer systems. Not to be used for storage design.
SCS[2]	0 – 2000 acres*	Method can be used for estimating peak flows and hydrographs for all design applications.

[1] Size limitations refer to the drainage basin for the stormwater management facility (e.g., culvert, inlet).
[2] There are many readily available programs (such as HEC-1) that utilize this methodology
* 2,000-acre upper size limit applies to single basin simplified peak flow only.

In general:

• The SCS Method is advocated for all small and large sites and design applications.

• The Rational Method is recommended for small highly impervious drainage areas such as parking lots and roadways draining into inlets and gutters, not for storage design.

• The USGS regression equations are recommended for drainage areas with characteristics within the ranges given for the equations. The USGS equations should be used with caution when there are significant storage areas within the drainage basin or where other drainage characteristics indicate that general regression equations might not be appropriate.

Several computer programs are acceptable and available for calculating the Rational and SCS

Note: It must be realized that any hydrologic analysis is only an approximation. The relationship between the amount of precipitation on a drainage basin and the amount of runoff from the basin is complex and too little data are available on the factors influencing the rainfall-runoff relationship to expect exact solutions.

2.2 Symbols and Definitions

To provide consistency within this section and throughout this Manual, the symbols used in Table 2.2-1 will be used. These symbols were chosen because of their wide use in technical publications. In some cases, the same symbol is used in existing publications for more than once definition. Where this occurs in this section, the symbol will be defined where it occurs in the text or the equations.

Table 2-3 Symbols and Definitions

Symbol	Definition	Units
A or a	Drainage area	acres
Af	Channel flow area	ft^2
B	Channel bottom width	ft
Bf	Baseflow	acre-feet
C	Runoff coefficient	-
Cf	Frequency factor	-
CN	SCS-runoff curve number	-
CPv	Channel Protection Volume	acre-feet
D	Depth of flow	ft
d	Time interval	hours
E	Evaporation	ft
Et	Evapotranspiration	ft
F	Pond and swamp adjustment factor	-
Gh	Hydraulic gradient	-
I or i	Runoff intensity	in/hr
I	Percent of impervious cover	%
I	Infiltration	ft

Ia	Initial abstraction from total rainfall	in
kh	Infiltration rate	ft/day
L	Flow length	ft
n	Manning roughness coefficient	-
Of	Overflow	acre-feet
P	Accumulated rainfall	in
P2	2-year, 24-hour rainfall	in
Pw	Wetted perimeter	ft
PF	Peaking factor	-
Q	Rate of runoff	cfs (or inches)
Qd	Developed runoff for the design storm	in
Qf	Extreme Flood Protection Volume	acre-feet
Qi	Peak inflow discharge	cfs
Qo	Peak outflow discharge	cfs
Qp	Peak rate of discharge	cfs
Qp25	Overbank Flood Protection Volume	acre-feet
Qwq	Water Quality peak rate of discharge	cfs
q	Storm runoff during a time interval	in
qu	Unit peak discharge	cfs (or cfs/mi²/inch)
R or r	Hydraulic radius	ft
Ro	Runoff	acre-feet
Rv	Runoff Coefficient	-
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
TL or T	Lag time	hours
Tp	Time to peak	hr
Tt	Travel time	hours
t	Time	min
Tc or tc	Time of concentration	min
TIA	Total impervious area	%
V	Velocity	ft/s
V	Pond volume	acre-feet
Vr	Runoff volume	acre-feet

Vs	Storage volume	acre-feet
WQv	Water Quality Volume	acre-feet

2.3 Design Frequency and Rainfall

The first step in any hydrologic analysis is an estimation of the rainfall that will fall on the site for a given time period. The amount of rainfall can be quantified with the following characteristics:

Duration (hours) – Length of time over which rainfall (storm event) occurs
Depth (inches) – Total amount of rainfall occurring during the storm duration
Intensity (inches per hour) – Depth divided by the duration

The Frequency of a rainfall event is the recurrence interval of storms having the same duration and volume (depth). This can be expressed either in terms of *exceedence probability* or *return period*.

Exceedence Probability – Probability that a storm event having the specified duration and volume will be exceeded in one given time period, typically 1 year

Return Period – Average length of time between events that have the same duration and volume

Thus, if a storm event with a specified duration and volume has a 1% chance of occurring in any given year, then it has an exceedence probability of 0.01 and a return period of 100 years.

Rainfall intensities for 16 locations across Georgia are provided in Figure 2-1 and should be used for all hydrologic analysis at the given locations. The values in these tables were derived in the following way:

- Initial values were derived from TP40 (Hershfield, 1961) and HYDRO 35 (NOAA, 1977) with the 60-minute and shorter values coming from HYDRO 35.
- Intensity values for smaller than the 2-year storm were extrapolated through a series of plots.
- All values were plotted and smoothed to ensure continuity between the two different sources and to catch any errors. The values for 60 minutes and less were fit using an equation of the form:

$$\mathbf{I = B / ((t + D) \wedge E)} \qquad \textbf{(2.1)}$$

where *I* is the rainfall intensity measured in units of inches per hour and *t* is the storm duration measured in units of minutes. B, *D*, and *E* are fitting parameters found at the top of Table 2-4. The tables are applicable to storFm durations up to and including 1 hour. This equation allows for automated calculation of rainfall values for the Rational Method without having to look values up in tables or interpolate them from charts. The time of

concentration is then substituted for **t** in Equation 2.2. The user can either use the values given in the tables or use the equations to calculate rainfall intensity values for times up to and including 1 hour.

Figure 2-2 shows an example Intensity-Duration-Frequency (IDF) Curve for Griffin, Georgia, for the seven storms (1-year through 100-year). These curves are plots of the tabular values. No values are given for times less than 5 minutes.

Values for areas other than those cities provided (Figure 2-2) can be interpolated. Figure 2-3 (included as the 10-year, 24-hour values from TP40) shows that the rainfall values vary south to north with generally constant values in a "V" pattern from east to west in central and South Georgia. This trend is accurate except in the far northeast corner of the state where higher elevations create an anomaly due to the orographic lifting. The anomaly does not extend south from the far northeast counties; therefore it is not correct to interpolate from this area and it should be ignored in areas outside of northeast counties. For these counties local values should be used.

Depending on the facility that is being designed or analyzed, different design frequencies will be required. In addition to the design frequency, the operations of all major drainage facilities in Spalding County shall be checked using the 100-year storm event to ensure there are no unexpected floods and to help reduce stormwater quantity and increase stormwater water quality.

Storage facilities shall be designed using the 2-, 5-, 10-, 25-, 50-, and 100-year storm events. An emergency overflow device shall be designed to pass the 100-year peak developed inflow without overtopping the dam. There should be at least one foot of free board between the 100-year water surface elevation in the emergency spillway and the top of the dam.

Figure 2-1 Location of Rain Fall Data Sites

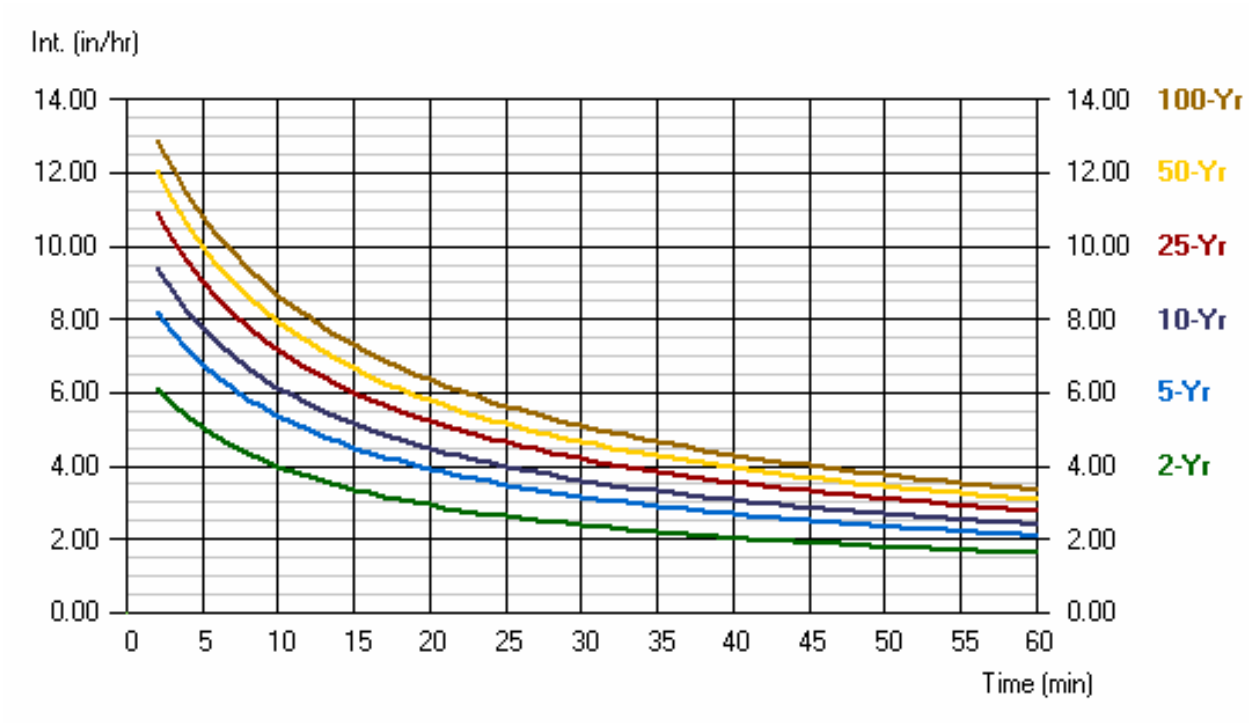


Figure 2-2 Hydrograph IDF Curves for Peachtree City, Georgia.

In Table 2.3 are the rainfall intensities for Peachtree City, Georgia taken from the Georgia Stormwater Management Manual Appendix A Table a-10. The table was chosen based on Spalding County's geographic location to Peachtree City.

Table 2- 4 Rainfall Intensity and Rainfall for Peachtree City, Georgia

		Return Period						
	IDF Curve Coefficients	1	2	5	10	25	50	100
	E	0.767	0.818	0.777	0.747	0.719	0.724	0.711
	B	38.81	57.93	61.46	61.89	64.80	74.70	77.93
	D	8	11	12	12	12	13	13
Hours	Minutes	Rainfall Intensity						
0.08	5	5.43	5.99	6.80	7.45	8.45	9.21	9.99
	6	5.13	5.70	6.51	7.14	8.11	8.85	9.61
	7	4.86	5.44	6.24	6.86	7.80	8.53	9.27
	8	4.63	5.20	6.00	6.60	7.52	8.23	8.95
	9	4.42	4.99	5.77	6.36	7.26	7.96	8.66
	10	4.23	4.79	5.57	6.15	7.02	7.71	8.39
	11	4.06	4.62	5.38	5.95	6.80	7.48	8.14
	12	3.90	4.45	5.20	5.76	6.59	7.26	7.91
	13	3.76	4.30	5.04	5.59	6.40	7.05	7.69
	14	3.63	4.16	4.89	5.43	6.22	6.86	7.49
0.25	15	3.50	4.03	4.75	5.28	6.06	6.69	7.30
	16	3.39	3.90	4.62	5.13	5.90	6.52	7.12
	17	3.29	3.79	4.49	5.00	5.75	6.36	6.95
	18	3.19	3.68	4.38	4.88	5.61	6.21	6.79
	19	3.10	3.58	4.27	4.76	5.48	6.07	6.64
	20	3.01	3.49	4.16	4.65	5.36	5.94	6.49
	21	2.93	3.40	4.06	4.54	5.24	5.81	6.36
	22	2.86	3.31	3.97	4.44	5.13	5.69	6.23
	23	2.79	3.23	3.88	4.35	5.03	5.57	6.10
	24	2.72	3.16	3.80	4.25	4.92	5.46	5.99
	25	2.66	3.08	3.72	4.17	4.83	5.36	5.87
	26	2.60	3.02	3.64	4.09	4.74	5.26	5.77
	27	2.54	2.95	3.57	4.01	4.65	5.16	5.66
	28	2.49	2.89	3.50	3.93	4.57	5.07	5.57
	29	2.43	2.83	3.43	3.86	4.49	4.98	5.47
0.50	30	2.38	2.77	3.37	3.79	4.41	4.90	5.38
	31	2.34	2.72	3.31	3.73	4.33	4.82	5.29
	32	2.29	2.67	3.25	3.66	4.26	4.74	5.21
	33	2.25	2.62	3.19	3.60	4.19	4.67	5.13
	34	2.21	2.57	3.14	3.54	4.13	4.59	5.05
	35	2.17	2.52	3.09	3.49	4.07	4.52	4.98
	36	2.13	2.48	3.04	3.43	4.00	4.46	4.90
	37	2.09	2.44	2.99	3.38	3.95	4.39	4.83
	38	2.06	2.40	2.94	3.33	3.89	4.33	4.77
	39	2.03	2.36	2.90	3.28	3.83	4.27	4.70
	40	1.99	2.32	2.85	3.23	3.78	4.21	4.64

	41	1.96	2.28	2.81	3.19	3.73	4.15	4.58
	42	1.93	2.25	2.77	3.14	3.68	4.10	4.52
	43	1.90	2.21	2.73	3.10	3.63	4.05	4.46
	44	1.87	2.18	2.69	3.06	3.58	4.00	4.40
0.75	45	1.85	2.15	2.66	3.02	3.54	3.95	4.35
	46	1.82	2.12	2.62	2.98	3.49	3.90	4.30
	47	1.80	2.09	2.59	2.94	3.45	3.85	4.25
	48	1.77	2.06	2.55	2.90	3.41	3.80	4.20
	49	1.75	2.03	2.52	2.87	3.37	3.76	4.15
	50	1.72	2.00	2.49	2.83	3.33	3.72	4.10
	51	1.70	1.98	2.46	2.80	3.29	3.67	4.06
	52	1.68	1.95	2.43	2.77	3.26	3.63	4.01
	53	1.66	1.93	2.40	2.74	3.22	3.59	3.97
	54	1.64	1.90	2.37	2.71	3.18	3.55	3.93
	55	1.62	1.88	2.34	2.68	3.15	3.52	3.88
	56	1.60	1.86	2.32	2.65	3.12	3.48	3.84
	57	1.58	1.83	2.29	2.62	3.08	3.44	3.81
	58	1.56	1.81	2.27	2.59	3.05	3.41	3.77
	59	1.54	1.79	2.24	2.56	3.02	3.37	3.73
1	60	1.53	1.77	2.22	2.54	2.99	3.34	3.69
2	120	0.97	1.19	1.44	1.60	1.85	2.07	2.24
3	180	0.69	0.82	1.02	1.16	1.33	1.47	1.62
6	360	0.40	0.49	0.61	0.70	0.81	0.91	0.98
12	720	0.24	0.29	0.36	0.42	0.48	0.53	0.58
24	1440	0.14	0.17	0.21	0.24	0.28	0.30	0.33

In Table 2-4 are the rainfalls for Peachtree City, Georgia derived using Table 2-4/

Table 2-5 Rainfall Intensity and Rainfall for Griffin, Georgia

Storm Duration		Rainfall (inches)					
Hours	Minutes	2-year	5-year	10-year	25-year	50-year	100-year
24	1440	4.08	5.04	5.76	6.72	7.2	7.92

2.4 Rational Method

2.4.1 Introduction

An important formula for determining the peak runoff rate is the Rational Formula. It is characterized by:

- Consideration of the entire drainage area as a single unit
- Estimation of flow at the most downstream point only
- The assumption that rainfall is uniformly distributed over the drainage area and is constant over time

The Rational Formula follows the assumption that:

- The predicted peak discharge (Q) has the same probability of occurrence (return period) as the used rainfall intensity (I)
- The runoff coefficient (C) is constant during the storm event

When using the Rational Method some precautions should be considered:

- In determining the C value (runoff coefficient based on land use) for the drainage area, hydrologic analysis should take into account any future changes in land use that might occur during the service life of the proposed facility.

- Since the Rational Method uses a composite C and a single t_c value for the entire drainage area, if the distribution of land uses within the drainage basin will affect the results of hydrologic analysis (e.g., if the impervious areas are segregated from the pervious areas), then basin should be divided into sub-drainage basins.

- The charts, graphs, and tables included in this section are given to assist the engineer in applying the Rational Method. The engineer should use sound 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 Application

The Rational Method can be used to estimate stormwater runoff peak flows for the design of gutter flows, drainage inlets, storm drain pipe, culverts and small ditches. It is most applicable to small, highly impervious areas. The recommended maximum drainage area that should be used with the Rational Method is 25 acres.

The Rational Method should not be used for storage design or any other application where a more detailed routing procedure is required. However, due to the popularity of the Modified Rational method among Georgia practitioners for design of small detention facilities, a method has been included in Section 2.2.

The Rational Method should also not be used for calculating peak flows downstream of bridges, culverts or storm sewers that may act as restrictions and impact the peak rate of discharge.

2.4.3 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 average rainfall intensity for a duration equal to the time of concentration, t_c (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:

$$Q = CIA \qquad (2.2)$$

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)

The coefficients given in Table 2-6 are applicable for storms of all storms.

Table 2-6 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
Unimproved areas (forest)	0.15
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
Streets:	
Asphalt and Concrete	0.95
Brick	0.85
Drives, walks, and roofs	0.95
Gravel areas	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

2.4.4 Infrequent Storms

Less frequent, higher intensity storms may require modification of the coefficient because infiltration and other losses have a proportionally smaller effect on runoff (Wright-McLaughlin Engineers, 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:

$$Q = C_f CIA \qquad (2.3)$$

The C_f values that can be used are listed in Table 2-7. The product of C_f times C shall not exceed 1.0.

Table 2-7 Frequency Factors for Rational Formula

Recurrence Interval (years)	C_f
10 or less	1.0
25	1.1
50	1.2
100	1.25

2.4.5 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). The time of concentration consists of an overland flow time to the point where the runoff is concentrated or enters a defined drainage feature (e.g., open channel) plus the time of flow in a closed conduit or open channel to the design point.

Figure 2-4 can be used to estimate overland flow time. 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 (C) is determined by the procedure described in a subsequent section of this chapter.

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 in Figure 2-5. Note: time of concentration cannot be less than 5 minutes.

Another method that can be used to determine the overland flow portion of the time of concentration is the "Kinematic Wave Nomograph" (Figure 2-6). The kinematic wave method incorporates several variables including rainfall intensity and Manning's "n". In using the nomograph, the engineer has two unknowns starting the computations: the time of concentration and the rainfall intensity. A value for the rainfall intensity "I" must be assumed. The travel time is determined iteratively.

If one has determined the length, slope and roughness coefficient, and selected a rainfall intensity table, the steps to use Figure 2-6 are as follows:

(Step 1) Assume a rainfall intensity.
(Step 2) Use Figure 2-6 (or the equation given in the figure) to obtain the first estimate of time of concentration.
(Step 3) Using the time of concentration obtained from Step 2, use the appropriate rainfall intensity table in Appendix A and find the rainfall intensity corresponding to the computed time of concentration. If this rainfall intensity corresponds with the assumed intensity, the problem is solved. If not, proceed to Step 4.
(Step 4) Assume a new rainfall intensity that is between that assumed in Step 1 and that determined in Step 3.
(Step 5) Repeat Steps 1 through 3 until there is good agreement between the assumed rainfall intensity and that obtained from the rainfall intensity tables.

Generally, the time of concentration for overland flow is only a part of the overall design problem.

Often one encounters swale flow, confined channel flow, and closed conduit flow-times that must be added as part of the overall time of concentration. When this situation is encountered, it is best to compute the confined flow-times as the first step in the overall determination of the time of concentration. This will give the designer a rough estimate of the time involved for the overland flow, which will give a better first start on the rainfall intensity assumption. For example, if the flow time in a channel is 15 minutes and the overland flow time from the ridge line to the channel is 10 minutes, then the total time of concentration is 25 minutes.

Other methods and charts may be used to calculate overland flow time if approved by the local review authority.

Two common errors should be avoided when calculating time of concentration. 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. 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 50 feet for impervious areas should be done only after careful consideration.

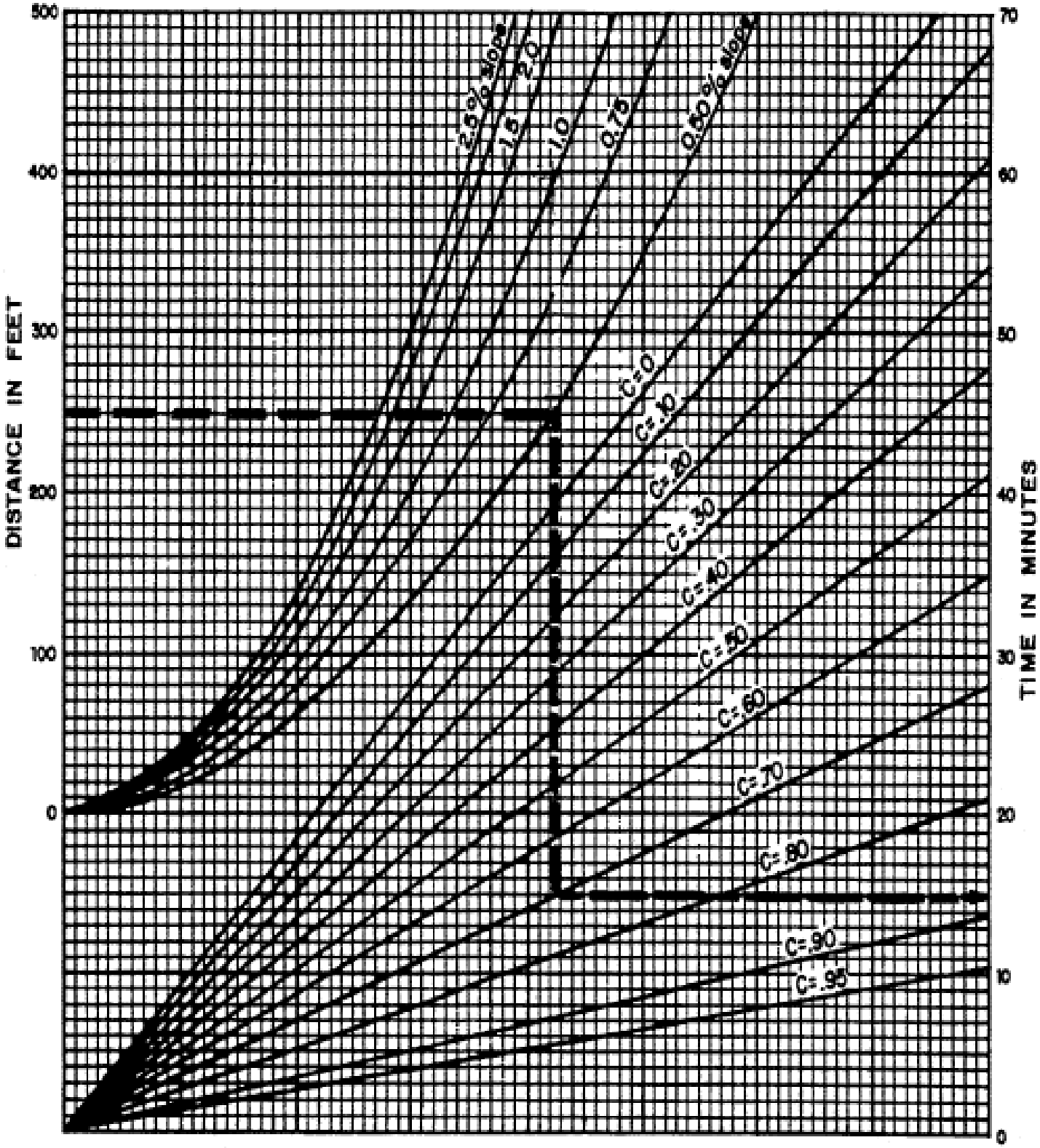


Figure 2-3 Rational Formula-Overland Time of Flow Nomograph
Source: Airport Drainage, Federal Aviation Administration, 1965)

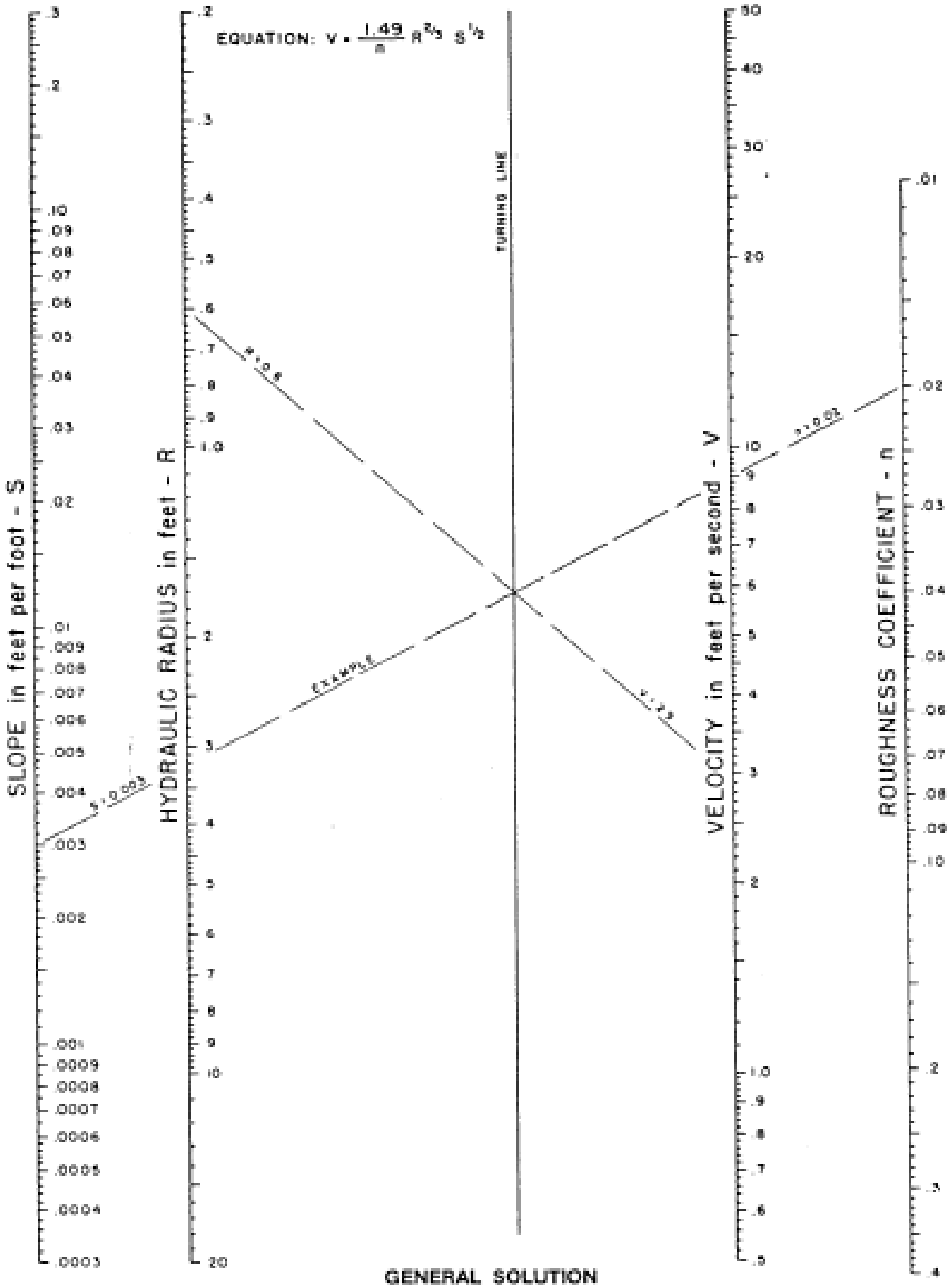
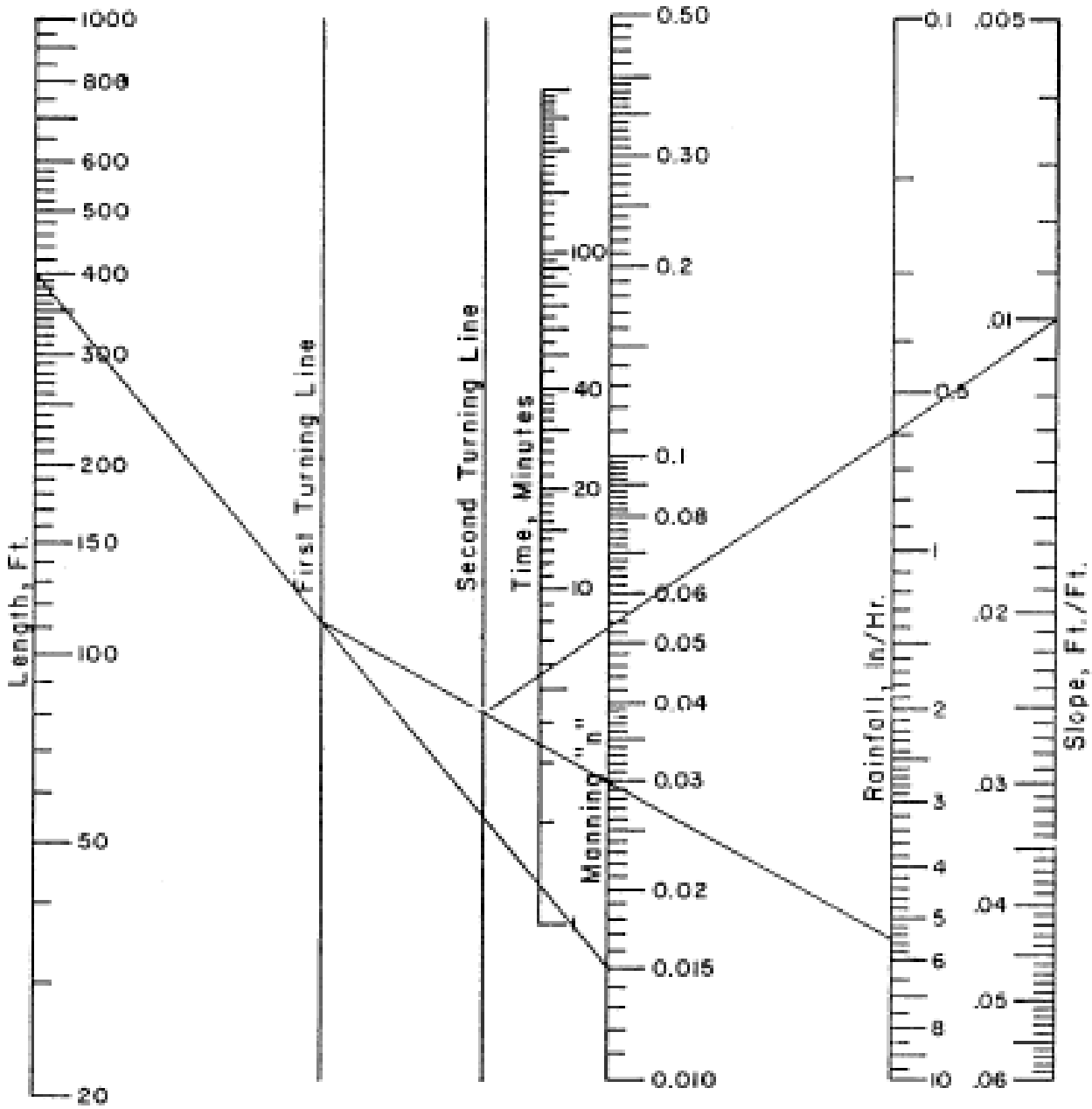


Figure 2-4 Manning's Equation Nomograph
(Source: USDOT, FHWA, HDS-3 (1961))

Equation solved by nomograph:

$$t_c\,(\text{Sec}) = 56\,\frac{L_o^{.6}\;n^{.6}}{i^{.4}\;S_o^{.3}}$$



Example:

L_o = 400 ft.
n = 0.015
i = 5.5 in./hr.
S_o = 0.01
t = 5.5 min.

ONE INCH is 25.4mm
ONE FOOT is 0.3048m

Figure 2-5 Kinematic Wave Nomograph
(Source: Manual For Erosion and Sediment Control In Georgia, 2001)

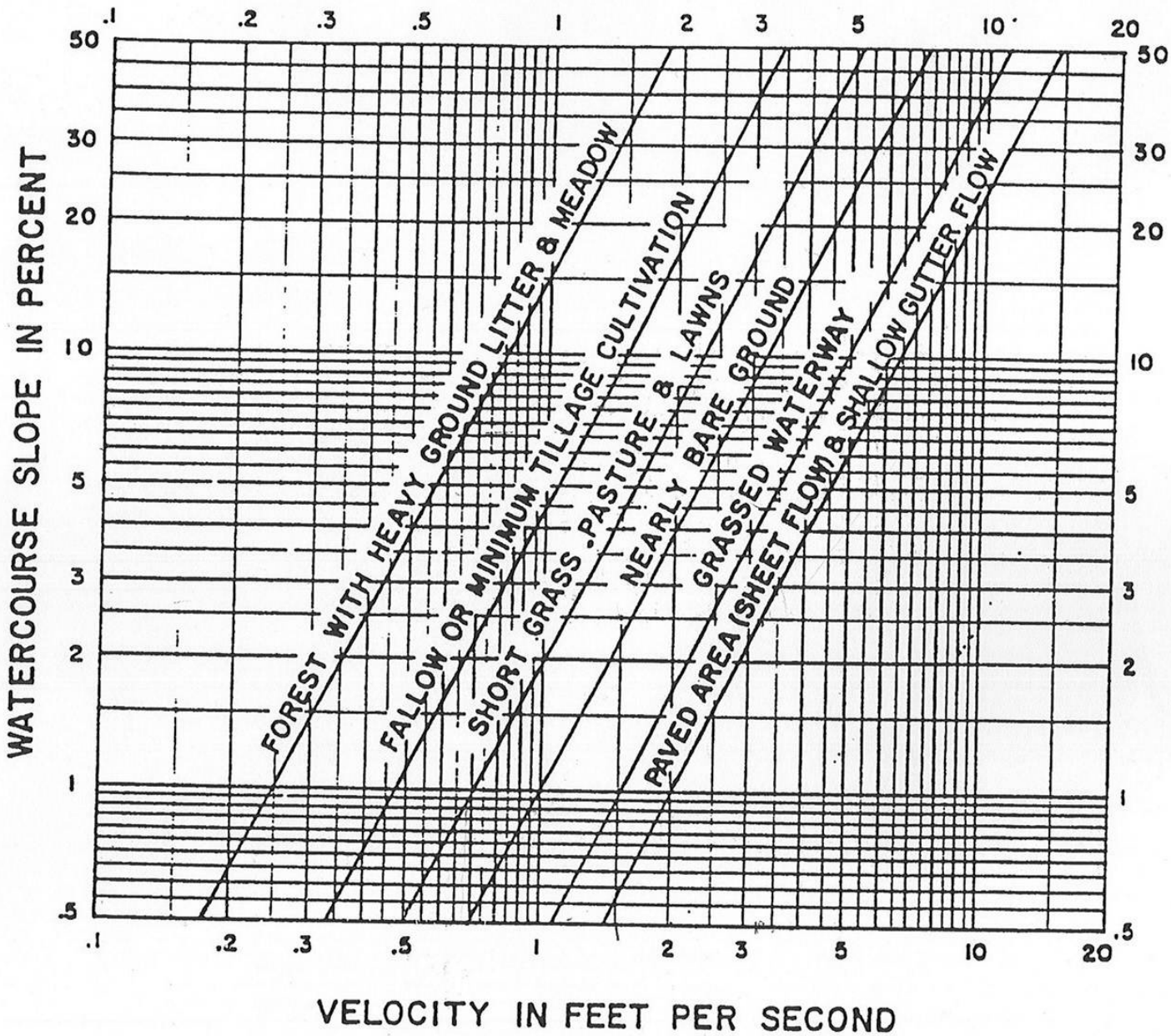


Figure 2-6 Average Velocities for Estimating Flow Time

Urban Hydrology for Small Watersheds
Techincal Release No. 55
Soil Conservation Service
U.S. Department of Agriculture
January 1975

2.4.6 Rainfall Intensity

The rainfall intensity (I) is the average rainfall rate measured in inches per hour 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 given in the rainfall tables in Section 2.4.5, or through the use of Equation 2.1.

2.4.7 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 2-6 gives the recommended runoff coefficients for the Rational Method.

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.

It may be that using only the impervious area from a highly impervious site (and the corresponding high C factor and shorter time of concentration) will yield a higher peak runoff value than by using the whole site. This should be checked particularly in areas where the overland portion is grassy (yielding a long t_c) to avoid underestimating peak runoff.

2.4.8 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 2-6 by using percentages of different land uses. In addition, more detailed composites can be made with coefficients for different surface types such as rooftops, asphalt, and concrete streets and sidewalks. 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.

2.4.9 Example Problem

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

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

Site Data

From a topographic map of the City of Roswell and a field survey, the area of the drainage basin upstream from the point in question is found to be 25 acres. In addition the following data were measured:

Average overland slope = 2.0 %
Length of overland flow = 80 ft
Length of main basin channel = 2,000 ft
Slope of channel - 0.015 ft/ft = 1.5 %
Roughness coefficient (n) of channel was estimated to be 0.080
From existing land use maps, land use for the drainage basin was estimated to be:
- Residential (single family) – 30 %
- Graded - sandy soil, 3 % slope – 20 %
- Street – asphalt – 10 %
- Lawn – sandy soil, 2-7 % slope – 40 %

From existing land use maps, the land use for the overland flow area at the head of the basin was estimated to be: Lawn - sandy soil, 2 % slope

Overland Flow

A runoff coefficient (C) for the overland flow area is determined from Table 2-7 to be 0.10.

Time of Concentration

From Figure 2-4 with an overland flow length of 80 ft, slope of 2 % and a C of 0.10, the overland flow time is 13 min. Channel flow velocity is determined from Figure 2-5to be 3.6 ft/s (n = 0.080, R = 2.00 (from channel dimensions) and S = 0.015). Therefore,

$$\text{Flow Time} = \frac{2{,}000 \text{ feet}}{(3.6 \text{ ft/s})/(60 \text{ s/min})} = 9.3 \text{ minutes}$$

and t_c = 13 + 9.3 = 22.3 min (use 23 min)

Rainfall Intensity

From Table 2.3-1 and using Equation 2.1, using a duration equal to 23 minutes and the appropriate equation coefficients,

I_{100} (100-yr return period) = 5.90 in/hr

Runoff Coefficient

A weighted runoff coefficient (C) for the total drainage area is determined below by utilizing the values from Table 2-8.

Table 2-8 Weighted Runoff Coefficents

Land Use	Percent of Total Land Area	Runoff Coefficient	Weighted RunoffCoefficient*
Residential (single family)	0.3	0.5	0.15
Graded area	0.2	0.3	0.06
Street (asphalt)	0.1	0.95	0.095
Lawn (sandy soil, 2-7 %)	0.4	0.15	0.06

Total Weighted Runoff Coefficient =0 .365
*Column 3 equals column 1 multiplied by column 2.

Peak Runoff

The estimate of peak runoff for a 100-yr design storm for the given basin is:

$$Q_{100} = C_f CIA = (1.25)(0.365)(5.90)(25) = 67.30 \text{ cfs}$$

2.5 SCS Hydrologic Method

2.5.1 Introduction

The Soil Conservation Service* (SCS) 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, Hydrology.

A typical application of the SCS method includes the following basic steps:

(1) Determination of curve numbers that 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, because of the location of Spalding County, total and excess rainfall amounts are determined. Note: See Figure 2.-8 for the geographic boundaries for the different SCS rainfall distributions.
(4) Using the unit hydrograph approach, the hydrograph of direct runoff from the drainage basin can be developed.

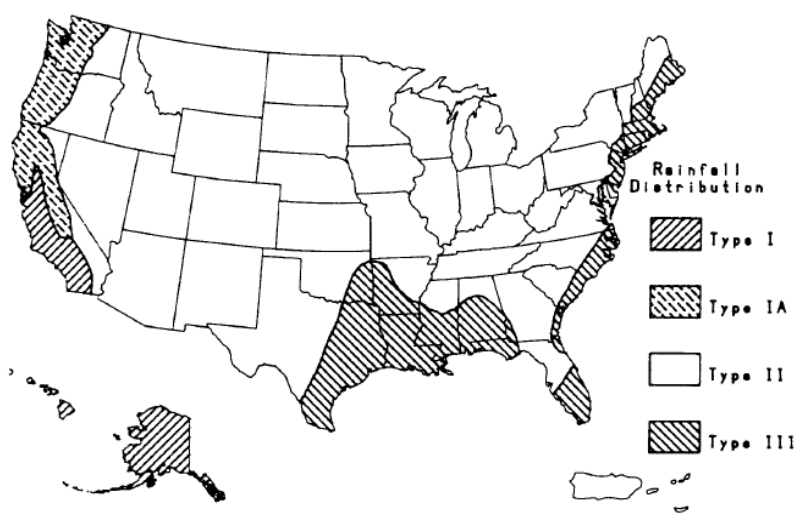


Figure 2-7 Geographic Boundaries for SCS Rainfall Distributions

2.5.2 Application

The SCS method can be used for both the estimation of stormwater runoff peak rates and the generation of hydrographs for the routing of stormwater flows. The simplified

method of subsection 2.5.6 can be used for drainage areas up to 2,000 acres. Thus, the SCS method can be used for most design applications, including storage facilities and outlet structures, storm drain systems, culverts, small drainage ditches and open channels, and energy dissipators.

2.5.3 Equations and Concepts

The hydrograph of outflow from a drainage basin is the sum of the elemental hydrographs from all the sub-areas of the basin, modified by the effects of transit time through the basin and storage in the stream channels. Since the physical characteristics of the basin including shape, size and slope are constant, the unit hydrograph approach assumes that there is considerable similarity in the shape of hydrographs from storms of similar rainfall characteristics. Thus, the unit hydrograph is a typical hydrograph for the basin with a runoff volume under the hydrograph equal to one (1.0) inch from a storm of specified duration. For a storm of the same duration but with a different amount of runoff, the hydrograph of direct runoff can be expected to have the same time base as the unit hydrograph and ordinates of flow proportional to the runoff volume. Therefore, a storm that produces 2 inches of runoff would have a hydrograph with a flow equal to twice the flow of the unit hydrograph. With 0.5 inches of runoff, the flow of the hydrograph would be one-half of the flow of the unit hydrograph.

The following discussion outlines the equations and basin concepts used in the SCS method.

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, combine hydrographs from different sub-basins as applicable, and/or route flows to points of interest.

Rainfall - The SCS method applicable to Spalding County is based on a storm event that has a Type II time distribution. These distributions are used to distribute the 24-hour volume of rainfall for the different storm frequencies (figure 2-8).

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:

$$\mathbf{Q = \frac{(P - Ia)^2}{(P - Ia) + S}} \qquad \mathbf{(2.4)}$$

Where: Q = accumulated direct runoff (in)
P = accumulated rainfall (potential maximum runoff) (in)
Ia = initial abstraction including surface storage, interception, evaporation, and infiltration prior to runoff (in)
S = potential maximum soil retention (in)

An empirical relationship used in the SCS method for estimating Ia is:

$$I_a = 0.2S \qquad (2.5)$$

This is an average value that could be adjusted for flatter areas with more depressions if there are calibration data to substantiate the adjustment.

Substituting 0.2S for I_a in Equation 2.4, the equation becomes:

$$Q = \frac{(P - 0.2S)^2}{(P + 0.8S)} \qquad (2.6)$$

Where: S = (1000/CN)-10 where CN = SCS curve number

Acceptable values for P can be found in Table 2-9.

Table 2-9 Runoff Curve Numbers[1]

Storm Duration		Rainfall (inches)					
Hours	**Minutes**	**2-year**	**5-year**	**10-year**	**25-year**	**50-year**	**100-year**
24	1440	4.08	5.04	5.76	6.72	7.2	7.92

Figure 2-9 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.

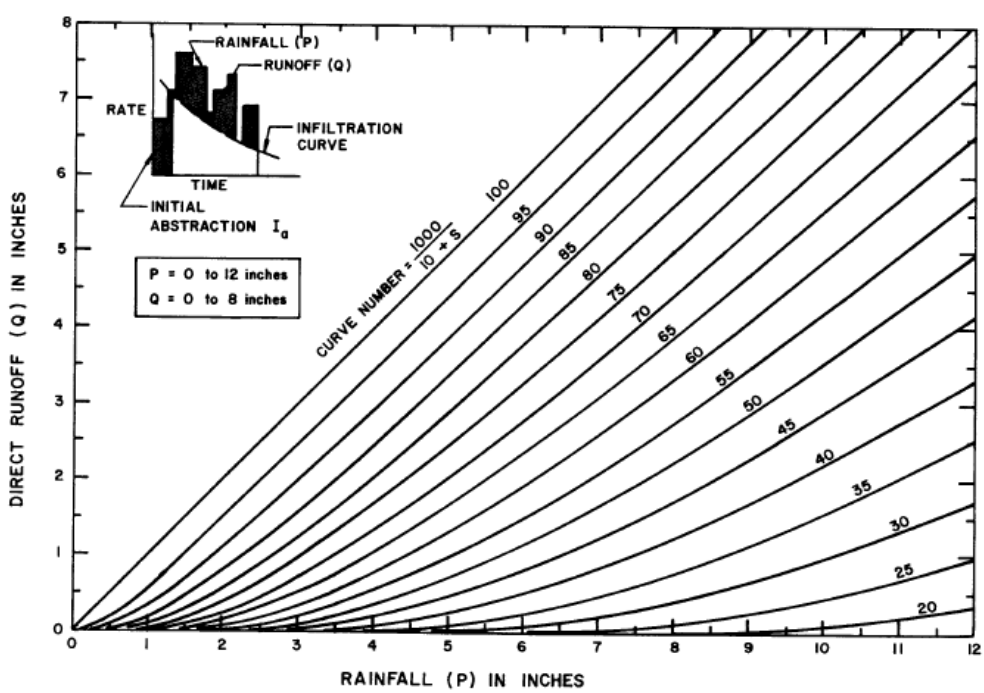


Figure 2-8 SCS Solution of the Runoff Equation
(Source: SCS, TR-55, Second Edition, June 1986)

Equation 2.6 can be rearranged so that the curve number can be estimated if rainfall and runoff volume are known. The equation then becomes (Pitt, 1994):

$$\mathbf{CN = 1000/[10 + 5P + 10Q - 10(Q^2 + 1.25QP)^{1/2}]} \qquad \mathbf{(2.7)}$$

2.5.4 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 the runoff potential. Soil properties influence the relationship between runoff and rainfall since soils have differing rates of infiltration. Based on infiltration rates, the 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 throughout the State of Georgia and their hydrologic classification can be found in the publication *Urban Hydrology for Small Watersheds, 2nd Edition, Technical Release Number 55, 1986*. Soil Survey maps can be obtained from local SCS offices for use in estimating soil type.

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 most hydrologic analysis, except in the design of state-regulated Category I dams where AMC III may be required. Areas with high water table

conditions may want to consider using AMC III antecedent soil moisture conditions. This should be considered a calibration parameter for modeling against real calibration data. Table 2.5-1 gives recommended curve number values for a range of different land uses.

Table 2-10 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 Ia = 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.

When a drainage area has more than one land use, a composite curve number can be calculated and used in the analysis. It should be noted that when composite curve numbers are used, the analysis does not take into account the location of the specific land uses but sees the drainage area as a uniform land use represented by the composite curve number.

Composite curve numbers for a drainage area can be calculated by using the weighted method as presented below.

Composite Curve Number Calculation Example

Land Use	Percent of Total Land Area	Curve Number	Weighted Curve Number (% area x CN)
Residential 1/8 acre Soil group B	0.80	0.85	0.68
Meadow Good condition Soil group C	0.20	0.71	0.14

Total Weighted Curve Number = 0.68 + 0.14 = 0.82

The different land uses within the basin should reflect a uniform hydrologic group represented by a single curve number. Any number of land uses can be included, but if their spatial distribution is important to the hydrologic analysis, then sub-basins should be developed and separate hydrographs developed and routed to the study point.

2.5.5 Urban Modifications of the SCS Method

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 developed 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 2.5-1 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, but allowing runoff to flow as sheet flow over significant pervious areas.

The following discussion will give some guidance for adjusting curve numbers for different types of impervious areas.

Connected Impervious Areas

The CNs provided in Table 2-9 for various land cover types 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.

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 2.5-1 are not applicable, use Figure 2-10 to compute a composite CN. For example, Table 2.5-1 gives a CN of 70 for a 1/2-acre lot in hydrologic soil group B, with an assumed impervious area of 25%. However, if the lot has 20% impervious area and a pervious area CN of 61, the composite CN obtained from Figure 2-10 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 2-11 if total impervious area is less than 30% or (2) use Figure 2-10 if the total impervious area is equal to or greater than 30%, because the absorptive capacity of the remaining pervious areas will not significantly affect runoff.

When impervious area is less than 30%, obtain the composite CN by entering the right half of Figure 2-11 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 1/2-acre lot with 20% total impervious area (75% of which is unconnected) and pervious CN of 61, the composite CN from Figure 2-11 is 66. If all of the impervious area is connected, the resulting CN (from Figure 10) would be 68.

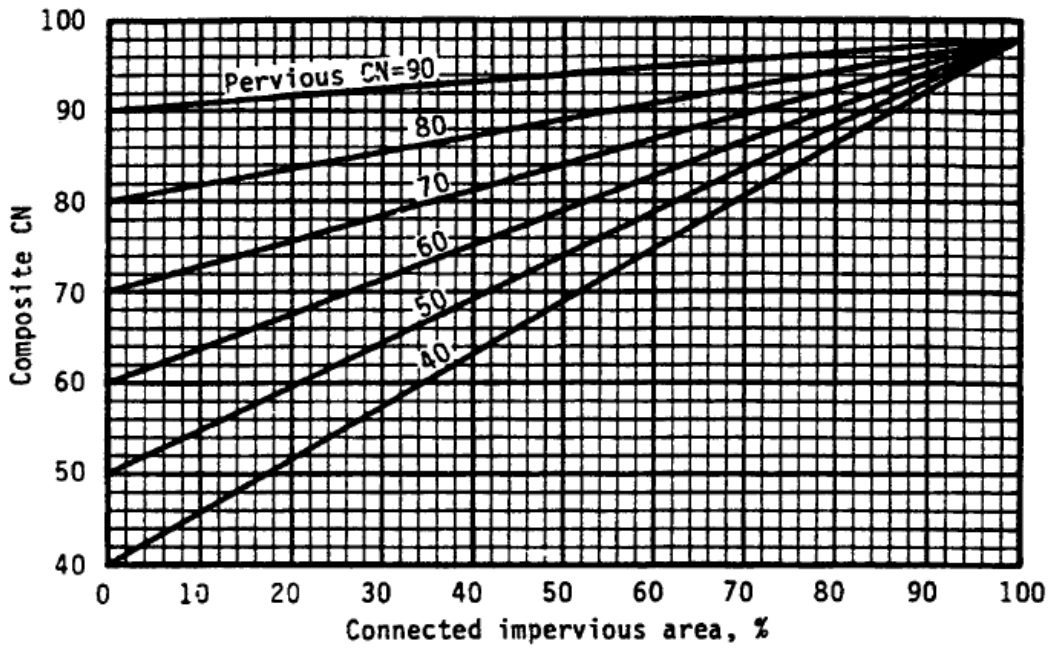


Figure 2-9 Composite CN with Connected Impervious Area
(Source: SCS, TR-55, Second Edition, June 1986)

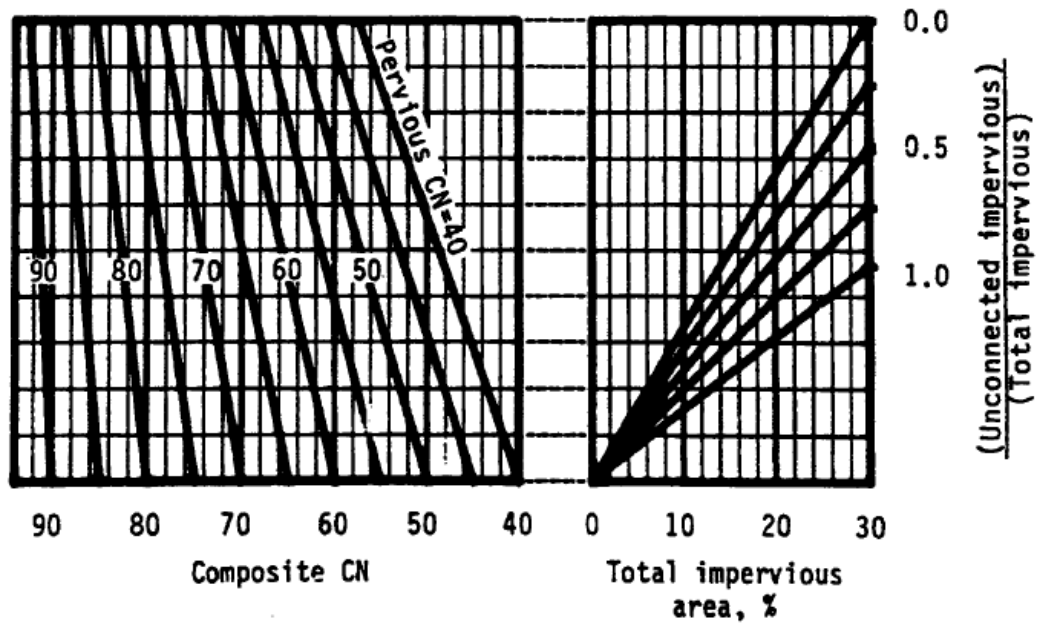


Figure 2-10 Composite CN with Unconnected Impervious Area (Total Impervious Area Less Than 30%)
(Source: SCS, TR-55, Second Edition, June 1986)

2.5.6 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 (USDA, 1986).

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:

$$\mathbf{T_t = \frac{L}{60V}} \qquad \textbf{(2.8)}$$

Where: T_t = travel time (min)
L = flow length (ft)
V = average velocity (ft/s)
60 = conversion factor from seconds to minutes

Sheet Flow

Sheet flow can be calculated using the following formula:

$$\mathbf{T_t = \frac{0.42\,(nL)^{0.8}}{(P_2)^{0.5}\,(S)^{0.4}}} \qquad \textbf{(2.9)}$$

Where: T_t = travel time (min)
n = Manning roughness coefficient (see Table 2.5-2)
L = flow length (ft),
P_2 = 2-year, 24-hour rainfall
S = land slope (ft/ft)

Table 2-11 Roughness Coefficients (Manning's n) for Sheet Flow[1]

Surface Description	n
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.)

Shallow Concentrated Flow

After a maximum of 50 to 100 feet, sheet flow usually becomes shallow concentrated flow. The average velocity for this flow can be determined from Figure 2-12, 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 2-12, or the following equations. These equations can also be used for slopes less than 0.005 ft/ft.

Unpaved $$V = 16.1345(S)^{0.5} \quad (2.10)$$

Paved $$V = 20.3282(S)^{0.5} \quad (2.11)$$

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

After determining average velocity using Figure 2-12 or Equations 2.10 or 2.11, use Equation 2.8 to estimate travel time for the shallow concentrated flow segment.

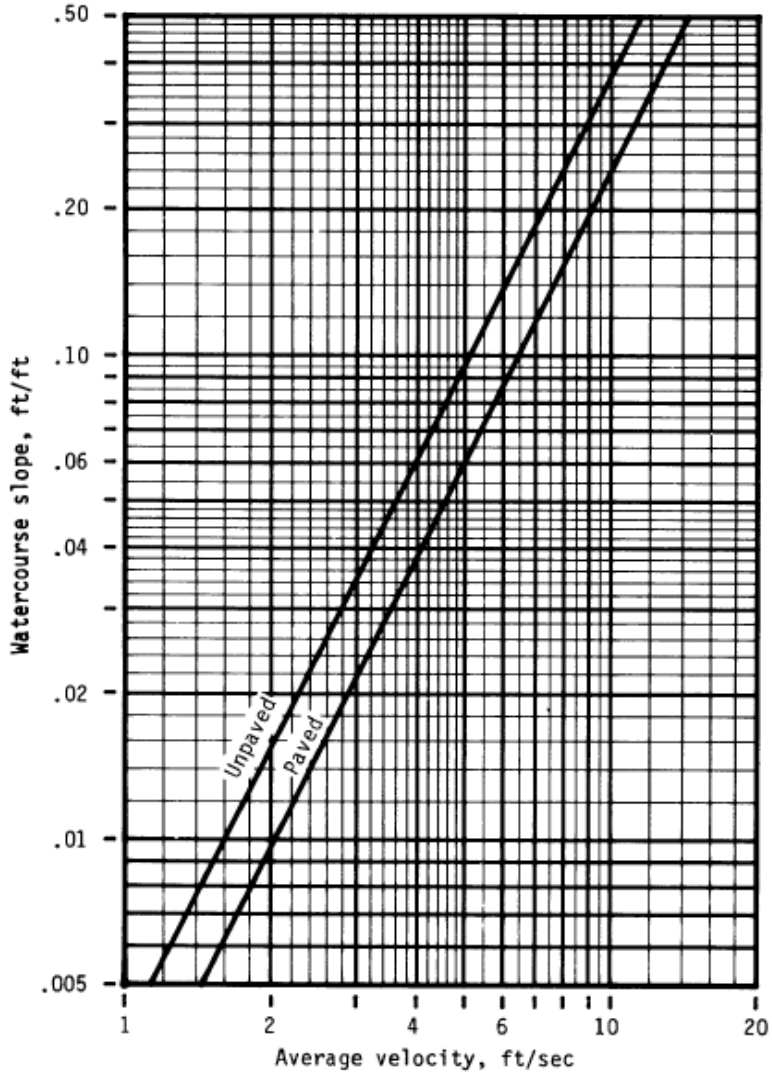


Figure 2-11 Average Velocities- Shallow Concentrated Flow
(Source: SCS, TR-55, Second Edition, June 1986)

Open Channels

Velocity in channels should be calculated from the Manning equation. Open channels are assumed to begin where surveyed cross section information has been obtained, where channels are visible on aerial photographs, where channels have been identified by the local municipality, 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 for travel time calculations is usually determined for bank-full elevation assuming low vegetation winter conditions.

Manning's equation is $$V = \frac{1.49\,(R)^{2/3}\,(S)^{1/2}}{n} \qquad (2.12)$$

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 2.12, T_t for the channel segment can be estimated using Equation 2.8.

Limitations

- Equations in this section should not be used for sheet flow longer than 50 feet for impervious land uses.

- In watersheds with storm sewers, carefully identify the appropriate hydraulic flow path to estimate t_c.

- A culvert or bridge can act as detention structure if there is significant storage behind it. Detailed storage routing procedures should be used to determine the outflow through the culvert or bridge.

2.5.7 Triangular Hydrograph Equation

The triangular hydrograph equation 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:

$$q_p = (484*A*q) / ((d/2) + T_L) \qquad (2.13)$$

Where: q_p = peak rate of discharge (cfs)
q = storm runoff volume during time interval (in.) (obtained from Equation 2.10)
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 ($2.67\ T_p$) (hr)

2.6 Simplified SCS Method

2.6.1 Overview

The following SCS procedures were taken from the SCS Technical Release 55 (USDA, 1986) which presents simplified procedures to calculate storm runoff volume and peak rate of discharges. These procedures are applicable to small drainage areas (typically less than 2,000 acres) with homogeneous land uses that can be described by a single CN value. The peak discharge equation is:

$$\mathbf{Qp = q_u AQFp} \qquad \mathbf{(2.14)}$$

Where: Qp = peak discharge (cfs)
qu = unit peak discharge (cfs/mi2/in)
A = drainage area (mi2)
Q = runoff (in)
Fp = pond and swamp adjustment factor

The input requirements for this method are as follows:

- tc – hours
- Drainage area – mi^2
- Type II rainfall distribution
- 24-hour design rainfall
- CN value
- 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.)

Computations for the peak discharge method proceed as follows:

(1) The 24-hour rainfall depth is determined from the rainfall tables in Appendix A for the selected location and return frequency.

(2) The runoff curve number, CN, is estimated from Table 2.5-1 and direct runoff, Qp, is calculated using Equation 2.13.

(3) The CN value is used to determine the initial abstraction, Ia, from Table 2.5-3, and the ratio Ia/P is then computed (P = accumulated 24-hour rainfall).

(4) The watershed time of concentration is computed using the procedures in subsection 2.5.6 and is used with the ratio Ia/P to obtain the unit peak discharge, qup, from Figure-13 for the Type II rainfall distribution. If the ratio Ia/P lies outside the range shown in the figures, either the limiting values or another peak discharge method should be used. Note: Figures 2-13 and 2-14 are based on a peaking factor of 484. If a peaking factor of 300 is needed,

these figures are not applicable and the simplified SCS method should not be used.

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

Table 2.5-3 Pond and Swamp Adjustment Factor, F_p

Pond and 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 2.13.

Table 2-12 Ia Values for Runoff Curve Numbers

Curve Ia	number (in)	Curve Ia	number (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		

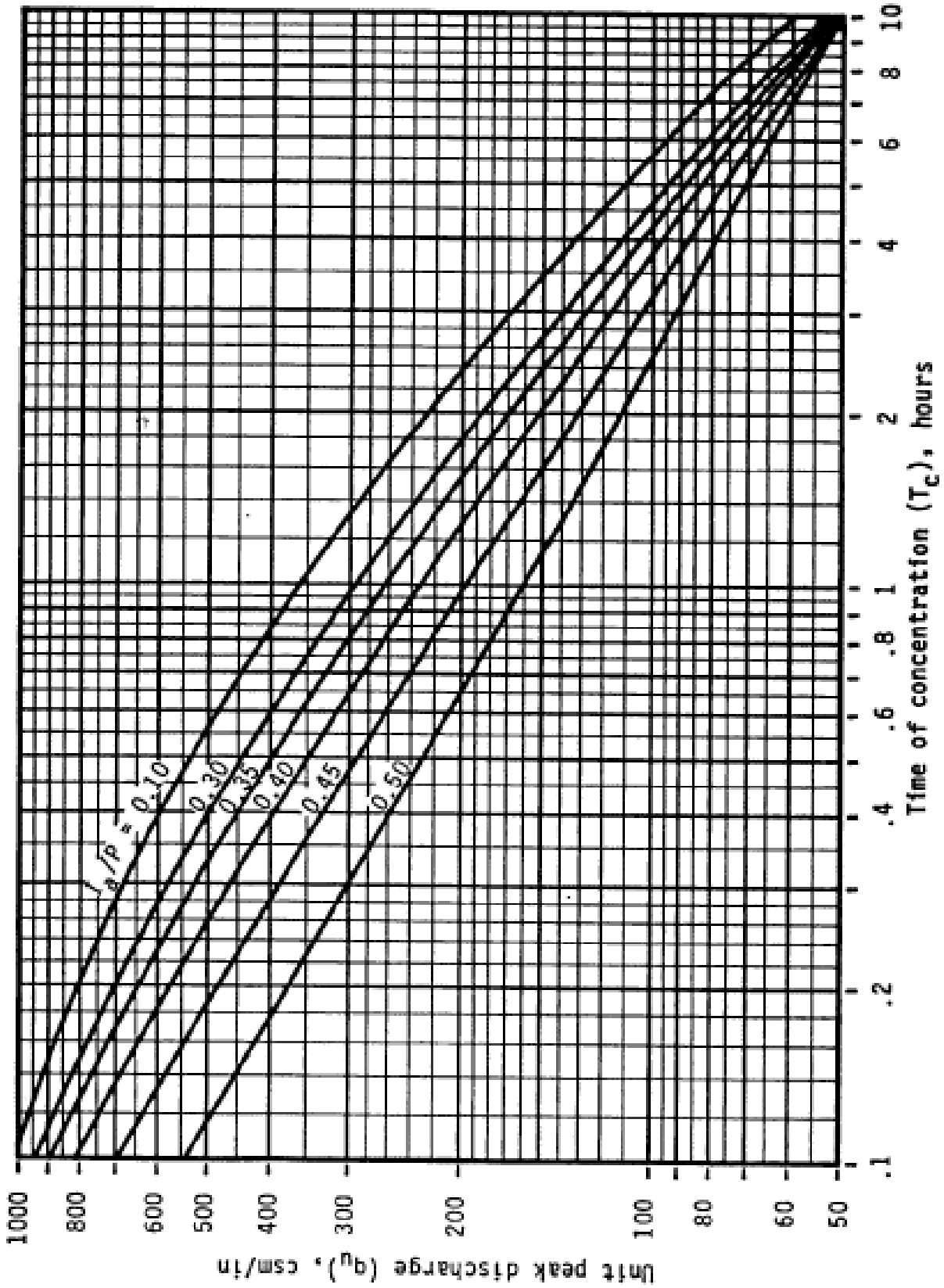


Figure 2-12 SCS Type II Unit Peak Discharge Graph
(Source: SCS, TR-55, Second Edition, June 1986)

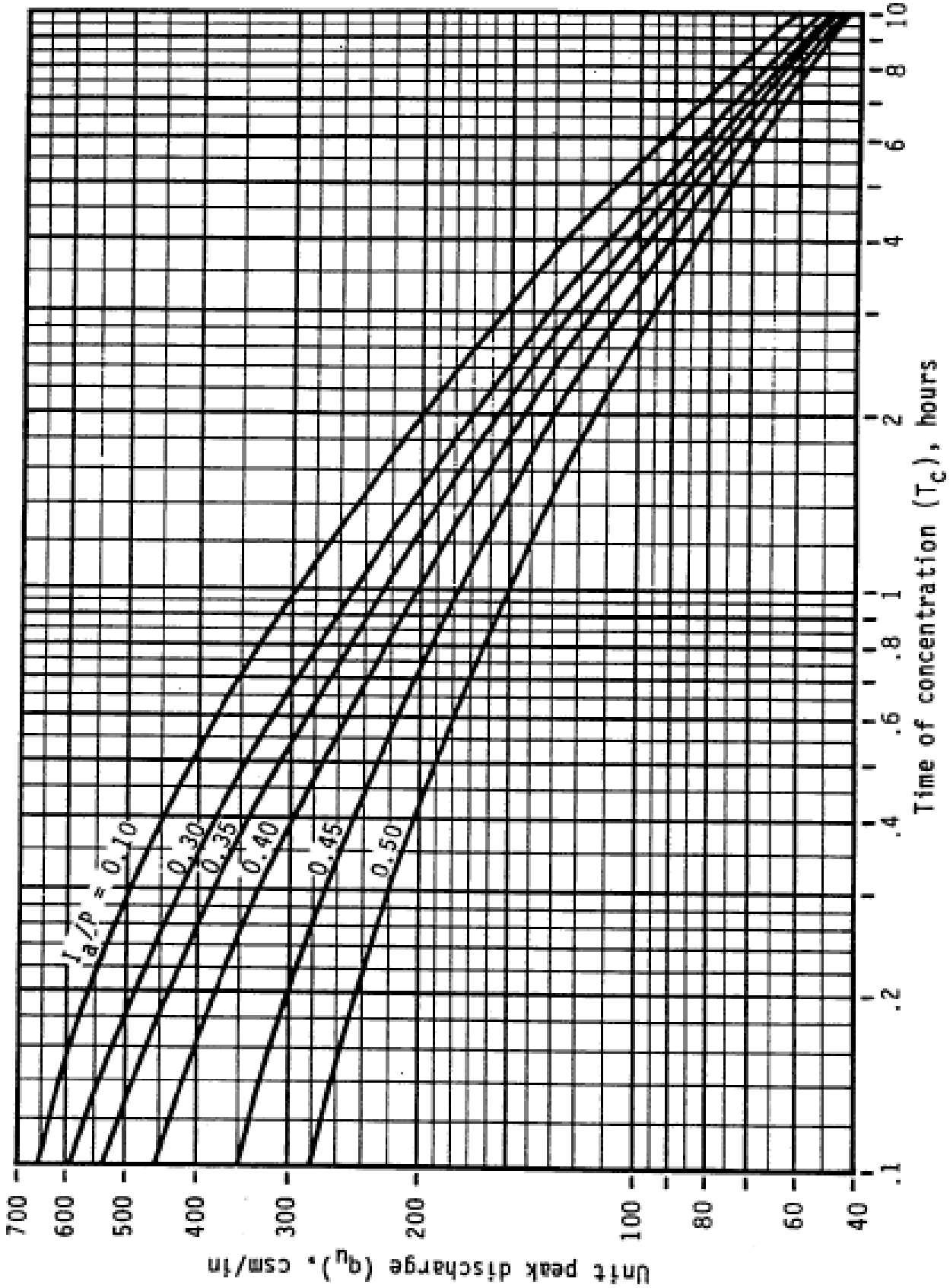


Figure 2-13 SCS Type III Unit Peak Discharge Graph

2.6.2 Example Problem

Compute the 100-year peak discharge for a 75-acre wooded watershed located in Spalding County, which will be developed as follows:

- Forest land - good cover (hydrologic soil group B) = 20 ac
- Forest land - good cover (hydrologic soil group D) = 10 ac
- 1/2 acre residential (hydrologic soil group C) = 30 ac
- Industrial development (hydrological soil group B) = 15 ac

Other data include the following: Total impervious area = 20 acres, % of pond / swamp area = 0

Computations

(1) Calculate rainfall excess:

- The 100-year, 24-hour rainfall is 8.00 inches (From Table 2.3-2).
- Composite weighted runoff coefficient is:

Dev. #	Area	% Total	CN	Composite CN
1	20 ac.	0.267	55	14.685
2	10 ac.	0.133	77	10.241
3	30 ac.	0.40	80	32
4	15 ac.	0.20	88	17.6
Total	75 ac.	1.00		74.5 = 75

* from Equation 2.6, Q_{100} (100-year) = 5.04 inches

(2) Calculate time of concentration

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

Segment	Type of Flow	Length (ft)	Slope (%)
1	Overland, sheet flow (Bermuda Grass n = 0.41)	80	2.5
2	Shallow channel	850	1.5
3	Main channel* (Not maintained, weeds and Brush, clear bottom, brush Sides, normal conditions)	1300	1.0

* For the main channel, n = 0.05 (estimated), width = 12 feet, depth = 2.5 feet, rectangular channel

Segment 1 - Travel time from Equation 2.9 with P_2 = 3.80 inches (Table 2.3-2)

$T_t = [0.42 (0.05 * 80)^{0.8}] / [(3.80)^{0.5} (0.025)^{0.4}] =$ 2.86 minutes

Segment 2 - Travel time from Figure 2-12 or Equation 2.10

$V = 16.1345 (0.015)^{0.5} = 1.98$ ft/sec (from Equation 2.10)

T_t = 850 ft / (60 sec/min) * (1.98 ft/sec) = 7.15 minutes (from Equation 2.8)

Segment 3 - Using Equation 2.12, $R = A/P_w = (12 * 2.5) / (12 + 2.5 + 2.5) = 1.76$ ft

V = [(1.49/0.05) (1.76) 2/3 (0.01) 1/2]/ 0.05 = 4.34 ft/sec

T_t = 1300 ft / 60 (sec/min) * (4.34 ft/sec) = 4.99 minutes

t_c = 2.86 + 7.15 + 4.99 = 15 minutes (0.25 hours)

(3) Calculate I_a/P for C_n = 75 (Table 2.5-1), I_a = 0.667 (Table 2.5-4)

I_a/P = (0.667 / 8.00) = 0.083 (Note: Use I_a/P = 0.10 to facilitate use of Figure-13. Straight line interpolation could also be used.)

(4) Unit discharge q_u (100-year) from Figure-13 = 725 csm/in

(5) Calculate peak discharge with F_p = 1 using Table 2.5-3 and using Equation 2.13

Q_{100} = (650 csm/in)(75 acres / (640 acre/mi²))(5.04 in)(1.0) = 428 cfs

2.7 Water Quality- Volume and Peak Flow/Discharge

2.7.1 Overview

The following procedure is a variation of the methodology presented in SCS Technical Release 55 (TR-55) to calculate storm runoff volume, peak rate of discharges and hydrographs. This procedure is applicable to small storm events. The following procedures outline the use of this variation of the SCS-TR 55 method. Conventional SCS Methods underestimate the volume and rate of runoff for rainfall events less than 2". This discrepancy in estimating runoff and discharge rates can lead to situations where a significant amount of runoff by-passes the filtering treatment practice due to an inadequately sized diversion structure or leads to the design of undersized grass channels.

2.7.2 Water Quality Volume Calculation

The Water Quality Volume (WQ_v) is the treatment volume required to remove a significant percentage of the stormwater pollution load, defined in this Manual as an 80% removal of the average annual post-development total suspended solids (TSS) load. This is achieved by intercepting and treating a portion of the runoff from all storms and all the runoff from 85% of the storms that occur on average during the course of a year.

The water quality treatment volume is calculated by multiplying the 85th percentile annual rainfall event by the volumetric runoff coefficient (R_v) and the site area. R_v is defined as:

$$\mathbf{R_v = 0.05 + 0.009(I)} \qquad \textbf{(2.15)}$$

Where: I = percent of impervious cover (%)

For the state of Georgia, the average 85th percentile annual rainfall event is 1.2 inches. Therefore, WQ_v is calculated using the following formula:

$$\mathbf{WQ_v = \frac{1.2\ R_v\ A}{12}} \qquad \textbf{(2.16)}$$

Where: WQ_v = water quality volume (acre-feet)
R_v = volumetric runoff coefficient
A = total drainage area (acres)

WQ_v can be expressed in inches simply as $1.2(R_v) = Q_{wv}$ **(2.17)**

2.7.3 Water Quality Volume -Peak Flow/Discharge Calculation

The peak rate of discharge for the water quality design storm is needed for the sizing of off-line diversion structures, such as for sand filters and infiltration trenches. An arbitrary storm would need to be chosen using the Rational Method, and conventional

SCS methods have been found to underestimate the volume and rate of runoff for rainfall events less than 2 inches. This discrepancy in estimating runoff and discharge rates can lead to situations where a significant amount of runoff by-passes the treatment practice due to an inadequately sized diversion structure and leads to the design of undersized bypass channels.

The following procedure can be used to estimate peak discharges for small storm events. It relies on the Water Quality Volume and the simplified peak flow estimating method above. A brief description of the calculation procedure is presented below.

1. Using WQ_v, a corresponding Curve Number (CN) is computed utilizing the following equation:

$$\mathbf{CN = 1000/[10 + 5P + 10Q_{wv} - 10(Q_{wv}^2 + 1.25\, Q_{wv}P)^{1/2}]} \quad \mathbf{(2.7)}$$

   Where, P = rainfall, in inches (use 1.2 inches for the Water Quality Storm in Spalding County)
   and Q_{wv} = Water Quality Volume, in inches ($1.2R_v$)

2. Once a CN is computed, the time of concentration (t_c) is computed (based on the methods described in this section).

3. Using the computed CN, t_c and drainage area (A), in acres; the peak discharge (Q_{wq}) for the water quality storm event is computed using a slight modification of the Simplified SCS Peak Runoff Rate Estimation technique of subsection 2.5.7. Use appropriate rainfall distribution type (Type II for Spalding County).
   - Read initial abstraction (I_a), compute I_a/P
   - Read the unit peak discharge (q_u) for appropriate t_c
   - Using WQ_v, compute the peak discharge (Q_{wq})

$$\mathbf{Q_{wq} = q_u * A * Q_{wv}} \quad \mathbf{(2.18)}$$

   where Q_{wq} = the water quality peak discharge (cfs)
   q_u = the unit peak discharge (cfs/mi²/inch)
   A = drainage area (mi²)
   Q_{wv} = Water Quality Volume, in inches ($1.2R_v$)

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.
3. Hydrologic routing cannot be considered.
4. Accuracy is reduced if the ratio I_a/P is outside the range given in Figure-13.
5. The weighted CN value must be greater than or equal to 40 and less than or equal to 98.

2.7.5 Example Problem

Using the data and information from the example problem in subsection 2.6.2 calculate the water quality volume and the water quality peak flow.

<u>Calculate water quality volume (WQ_v)</u>

Compute volumetric runoff coefficient, R_v
$R_v = 0.05 + (I)(0.009) = 0.05 + (0.009)(20/75 \times 100\%) = 0.29$

Compute water quality volume, WQ_v
$WQ_v = 1.2(R_v)(A)/12 = 1.2(0.29)(75)/12 = 2.175$ acre-feet

<u>Calculate water quality peak flow</u>

Compute runoff volume in inches, Q_{wv}:
$Q_{wv} = 1.2\ R_v = 1.2 * 0.29 = 0.348$ inches

Computer curve number:

$$CN = 1000/[10 + 5P + 10Q_{wv} - 10(Q_{wv}^2 + 1.25\ Q_{wv}\ P)^{1/2}$$

$$CN = 1000/[10 + (5*1.2) + (10*0.348) - 10(0.348^2 + (1.25*0.348*1.2))^{1/2}]$$
$$= 87$$

$t_c = 0.25$ (computed in previous example)

$S = 1000/CN – 10 = 1000/87 – 10 = 1.49$ inches (from Equation 2.6)
$0.2S = I_a = 0.298$ inches (from Equation 2.5)
$I_a/P = 0.298/1.2 = 0.248$

Find q_u:
From Figure-13 for $I_a/P = 0.248$ $q_u = 675$ cfs/mi²/in

Compute water quality peak flow:
$Q_{wq} = q_u * A * Q_{wv} = (675$ cfs/mi²/in$) * (75$ acre / (640 acre/mi²)$) * (0.348$ in$) =$ 27.5 cfs

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