Some Comments

on the

Rational Formula

by

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THE RATIONAL FORMULA

Introduction

Many new methodologies involving complex computer programs have been proposed in recent years for the planning and design of urban stormwater management systems. These systems include storm sewers, detention areas and overflow facilities and take into account both the quantity and quality of urban runoff. However, until these emerging methods come into more general use, the smaller storm sewers in the system will continue to be designed using the rational method. Thus, a review of its orgins and present-day interpretations is in order so that designers are reminded of what it is and of what it is not, of its limitations and its many interpretations.

The rational formula consists of four variables: a runoff coefficient, rainfall intensity, drainage area and time of concentration. The definitions of these variables have been expressed in various ways and these have led to some widely-held misconceptions. These misconceptions and the assumptions and limitations of the rational formula are each discussed in turn.

The two variables subject to the widest intrepretation are the runoff coefficient and the time of concentration. Presently, a designer can select values for C for a watershed which differ by two or three times from each other simply by using tables recommended by various agencies and texts. A new formula is proposed for the runoff coefficient which should reduce the present variability in the estimates of C. The same variability exists for estimates of the time of concentration, t_c. Using the same data and presently available equations, estimates of t_c can range from 5 to 35 minutes.

Each of the four variables is discussed in turn and comments are made on the usefulness and shortcomings of several of the tables, equations and figures presently used to estimate these variables. Following this, some examples of the use of the rational formula are given along with some advice on how the rational method should be applied to the design of storm sewers.

As originally conceived, the rational formula yields only a peak discharge rate. However, some engineers also use the rational formula to develop a hydrograph for detention basin design. Two such methods are discussed along with the problems and uncertainties inherent in using the rational formula for hydrograph development. Examples of these two methodologies are given and are compared with the results obtained from using the method contained in TR55 of the Soil Conservation Service.[1]

History of the Formula

The rational formula had its beginnings about 130 years ago. In 1851, T. J. Mulvaney, an Irish engineer, published a paper entitled "On the Use of the Self-registering Rain and Flood Guages in Making Observations on the Relation of Rainfall and Flood Discharges in a Given Catchment" in the Transactions of the Institution of Civil Engineers, Ireland.[2] Though not stated as such, the underlying principles of the rational formula, including the concept of the time of concentration, were definitely implied in his paper.

However, this paper was largely ignored and not until 1889 did the rational formula begin to come into general use. In that year Emil Kuichling, the city engineer of Rochester, New York, presented a paper entitled "The Relation Between the Rainfall and the Discharge of Sewers in Populous Districts" before the American Society of Civil Engineers.[3] He indicated that "in drainage areas of moderate size, the heaviest discharge always occurs when the rain lasts long enough at its maximum intensity to enable all portions of the area to contribute to the flow."

He concluded

"that there must be some definite relation between these fluctuations of discharge and the intensity of the rain, also between the magnitude of the drainage area and the time required for the floods to appear and subside."

The rational formula was introduced into England in 1906 by David Ernest Lloyd-Davies in his paper "The Elimination of Storm-Water from Sewerage Systems" before the Institution of Civil Engineers.[4] Thus, in England, the rational formula is known as the Lloyd-Davies formula.

In the next few decades several writers sought to estimate the time of concentration (t_c), runoff coefficient (C) and rainfall intensity (i) more accurately. Some success was achieved with rainfall intensity through the development of intensity-duration-frequency (I-D-F) curves. However, work on t_c and C met with much less success. In the last 40 years, there have been few if any improvements in the use of the rational formula; what has occurred is a proliferation of methods to estimate the various factors in the form of equations, graphs and tables. This movement towards simplicity has resulted generally in some widely-held misconceptions and mediocrity in the use of the formula.

Hydrologic Cycle

Any formula or methodology which estimates a peak discharge rate and/or flood hydrograph must, to a greater or lesser extent, incorporate the several portions of the hydrologic cycle. Thus, a review is necessary to determine to what extent the rational formula meets this test.

The hydrologic cycle consists of an unending sequence of events. Water vapor in the atmosphere is lifted by some mechanism and then falls to the earth's surface as one of several forms of precipitation. In the rational formula, we are only concerned with precipitation which falls as rain. Some rain is intercepted by foliage and structures before it reaches the earth's surface. That which reaches the ground first gets everything wet and then begins to fill the innumerable surface depressions. Only after this depression storage volume is satisfied and if the rainfall intensity is greater than the infiltration rate at that point in time does surface runoff begin. This surface runoff flows overland, then in channels of ever-increasing size until the runoff reaches the ocean. Evaporation from land and water surfaces adds water vapor to the atmosphere and the cycle continues.

At some point in some channel we can measure a runoff hydrograph, the peak of which is estimated by the rational formula. Two other portions of the hydrologic cycle, evapotranspiration and groundwater flow, play an insignificant role in the short time spans, small drainage areas and channels with which the rational formula is concerned and can be neglected.

Definitions of Variables

The rational formula is usually expressed as

$$Q_T = Ci_TA \tag{1}$$

where Q_T is the estimate of the peak rate of runoff in cubic feet per second for some recurrence interval, T; C is the fraction of rainfall, expressed as a dimensionless decimal, that appears as surface runoff from the tributary area (the ratio of surface runoff to rainfall); i_T is the average rainfall intensity in inches per hour during a period of time equal to t_c for some recurrence interval, T; A is the watershed area in acres tributary to the point of design; and t_c is the rainfall intensity averaging time in minutes, usually referred to as the time of concentration, equal to the time required for water to flow from the hydraulically most distant point in the watershed to the point of design.

Precipitation in the hydrologic cycle is included in the rational formula by using the average rainfall intensity over some time period. By default, all other portions of the hydrologic cycle must be contained in the runoff coefficient, C. Therefore, C includes interception, depression storage, infiltration, evaporation and groundwater flow. The variables needed to estimate C should include soil type, land use, degree of imperviousness, watershed slope, surface roughness, antecedent moisture condition, duration of rainfall and the intensity of rainfall as reflected by the recurrence interval. The fewer of these variables used to estimate C, the less accurately will the rational formula reflect the hydrologic cycle.

The peak discharge rate is assumed to vary directly with the magnitude of the drainage area. This assumption makes the equation essentially dimensionally accurate since 1.0 acre-inch per hour is equal to 1.0083 cubic feet per second.

The next logical step would be to discuss each of the variables in the rational formula in detail. However, by first discussing some of the assumptions, limitations and misconceptions of the rational formula, it is hoped that the reader will have a better appreciation for the ensuing discussion of the above variables.

Assumptions and Misconceptions

Assumptions and misconceptions are grouped together because an assumption used in the rational formula might in itself be a misconception or could be a conclusion based on some misconception. Several assumptions are listed below with each followed by a brief discussion.

The peak rate of runoff at some point is a direct function of the tributary drainage and the average rainfall intensity during the time of concentration to that point. This is the rational formula stated in words and is the basis (the basic assumption) of Kuichling's 1889 paper.[3] Sufficient data, both rainfall and runoff records, have not been available to either prove or disprove this hypothesis.

The method assumes that the frequency (recurrence interval) of the peak discharge rate is the same as the frequency of the average rainfall intensity. This is not always the case due to watershed-related variations. However, this assumption is used in many methodologies for estimating peak flows or runoff hydrographs.

The runoff frequency curve is parallel to the rainfall frequency curve. This implies that the same value of the runoff coefficient C is used for all recurrence intervals. However, work done by Schaake, Geyer and Knapp indicates that the two curves tend to converge at the rarer frequency rainfall events.[5]

Each of the variables (C,i,A) is independent of each other and each is estimated separately. This is one of the major misconceptions. There is some interdependency among the variables. Present procedure is to estimate each variable separately from an equation, graph, map or table. A close look at these aids indicates, in most cases, a lack of recognition of any interdependency between these variables.

The time of concentration t_c is the time required for water to flow from the hydraulically most remote point in the watershed to the point of design. Rather than an assumption, the foregoing statement is usually given as the definition of t_c. However, Schaake, Geyer and Knapp have stated that there is no known way to determine t_c, either from measurements in the field during storms or from records of rainfall and runoff and[5]

> "except for steady state conditions, which rarely, if ever, are reached during a thunderstorm, there is no good reason to believe that the time of flow from the farthest point in a drainage area should necessarily be the best rainfall averaging time to use in the Rational Method."

The rainfall intensity remains constant during the time period equal to t_c. Based on rainfall records, this assumption is true for short periods of time, such as a few minutes. However, as the time period increases, this assumption becomes less and less realistic.

The above assumption has led to another assumption: the definition of i in the rational formula. A common definition of i is the rainfall intensity in inches per hour of a storm whose "duration" is equal to the time of concentration of the basin. This definition evolved from current practice or current practice evolved from this definition.

"Duration" has been placed in parentheses because the interpretation placed on "duration" has led to the worst misconception of all. The common interpretation is that the duration of the storm is equal to t_c. This assumption is totally false and misleading. It is, of course, theoretically possible, since rainfall is a random event; however, the much more common case is that the total storm duration is considerably longer than t_c. Of equal importance is the concept that t_c (rainfall intensity averaging time) can occur during any segment of the total storm duration – at the beginning, before, during or after the middle portion or near the end.

This concept also has implications for the runoff coefficient C and how well the rational formula mirrors the hydrologic cycle. If t_c occurs at the beginning of the storm, then the antecedent moisture conditions become important. If t_c occurs near the end of a long storm, then the ground may be saturated and the depressions already filled with water when t_c begins.

Another assumption and misconception is that the area to be used is the total area tributary to the point of design. Kuichling recognized this possibility when he stated that[3]

> "the conclusion is accordingly irresistible that the rates of rainfall adopted in computing the dimensions of a main sewer must correspond to the time required for the concentration of the drainage waters from the whole tributary area when small, or from so much thereof as will produce an absolute maximum discharge when the area is very large."

Time of concentration formulas estimate t_c. Unfortunately, many times this assumption is just not true. T_c consists of an inlet time plus flow time. Inlet time consists of the time required for water flowing overland to reach established surface drainage channels, such as ditches and street gutters, plus travel time through them to the point of inlet to a storm sewer. Flow time is the time of flow through the storm sewer to the point of design. Even though many equations purportedly yield t_c, some estimate only overland flow time or inlet time.

The rational method assumes that runoff is linearly related to rainfall. If rainfall is doubled; runoff is doubled. This is not really accurate, for many variables interact.

One last major misconception is that the runoff coefficient C is a constant. C is a variable and during the design of a storm sewer system, it should take on several different values for the various pipe segments, rather than retain a constant value throughout the entire design, even though the land use remains the same.

Limitations of the Formula

The most outstanding limitation is that the only product of the method is a peak discharge. The method provides only an estimate of a single point on the runoff hydrograph.

Another limitation is that the results are usually not replicable from user to user. There are considerable variations in interpretation and methodology in the use of the formula. The simplistic approach permits and requires a wide latitude of subjective judgement in its application. Each firm or agency has its favorite t_c formula, its favorite table for determining C, its own method for determining the tributary area and its own set of criteria for determining which recurrence interval is to be used in certain situations.

The average rainfall intensities used in the method bear no time sequence relation to the actual rainfall pattern during a storm. The intensity - duration - frequency (I-D-F) curves prepared by the Weather Bureau are not time sequence curves of precipitation. The maximums of the several durations as used in the method are not necessarily in their original sequential order; and the resulting tabulations of maximums ordered by size or duration may bear little resemblence to the original storm pattern. In many, if not most, cases, the intensities on the same frequency curve for various durations are not from the same storm.

The method assumes that the rainfall intensity is uniform over the entire watershed during the "duration" of the storm. This assumption is true only for small watersheds and time periods, thus limiting the use of the rational formula to small watersheds. Whether "small" means 20 acres or 200 acres is still being discussed.

The method also assumes that the runoff rate reaches a maximum at a time equal to t_c. This assumption is true only when equilibrium conditions exist, which seldom occur during a thunderstorm, except over small areas, again limiting the usefulness of the rational formula.

Discussion of the Variables

With the preceding as background, each of the four variables in the rational formula is discussed in turn: runoff coefficient, rainfall intensity, rainfall intensity averaging time and tributary drainage area. While the rainfall intensity averaging time t_c does not appear in the formula, it must be estimated in order to estimate the rainfall intensity.

Runoff Coefficient C

Various writers have used one or more variables to estimate C. A compilation of these variables yields the following list.

1. percentage of impervious surface

2. character of soil (soil type)

3. duration of rainfall

4. intensity of rainfall

5. shape of tributary drainage area

6. antecedent moisture conditions

7. slope of watershed

8. design frequency (recurrence interval)

9. nature of the surface (land use)

10. surface storage (pondage)

11. interception

12. roof drainage - is it connected directly to the storm sewer, directed to a driveway or directed onto a pervious surface?

Variation of C with i_T. As indicated above, some writers state that C varies with rainfall intensity. As the rainfall intensity increases, the value of C also increases. This is logical since after interception and depression storage are satisfied and the infiltration rate has been reduced to some constant minimum value, any increase in the rainfall rate must be accompanied by an increase in the rate of runoff. From the first portion of the hydrologic cycle, the following equation can be written.

$$P = F + I_a + SRO \tag{2}$$

where P is precipitation, F is infiltration, I_a is initial abstraction which includes interception and depression storage and SRO is surface runoff, all measured in inches. Also,

$$P = i_T \times \text{time} \tag{3}$$

For simplicity, assume the following conditions: the soil is saturated prior to the beginning of the storm, the minimum infiltration capacity of the soil is 1.27 cm/hr (0.5 in./hr), the initial abstraction is 1.27 cm/hr (0.5 in.) the storm duration is 1.0 hr and the watershed contains no impervious area. The surface runoff for various rainfall intensities and resulting values of C are shown in Table 1, Variation of C with i_T. These results are based on the following equations.

$$SRO = P - F - I_a \tag{4}$$

$$C = SRO/P \tag{5}$$

Note that the values of C range from 0.00 to 0.83, hardly a constant value, as shown in Figure 1, Variation of C with i_T.

TABLE 1

VARIATION OF C WITH i_T

Average Intensity in./hr.	P in.	F in.	I_a in.	SRO in.	C
0.0	0.0	0.0	0.0	0.0	0.00
0.5	0.5	0.5	0.0	0.0	0.00
1.0	1.0	0.5	0.5	0.0	0.00
1.5	1.5	0.5	0.5	0.5	0.33
2.0	2.0	0.5	0.5	1.0	0.50
2.5	2.5	0.5	0.5	1.5	0.60
3.0	3.0	0.5	0.5	2.0	0.67
3.5	3.5	0.5	0.5	2.5	0.71
4.0	4.0	0.5	0.5	3.0	0.75
4.5	4.5	0.5	0.5	3.5	0.78
5.0	5.0	0.5	0.5	4.0	0.80
5.5	5.5	0.5	0.5	4.5	0.82
6.0	6.0	0.5	0.5	5.0	0.83

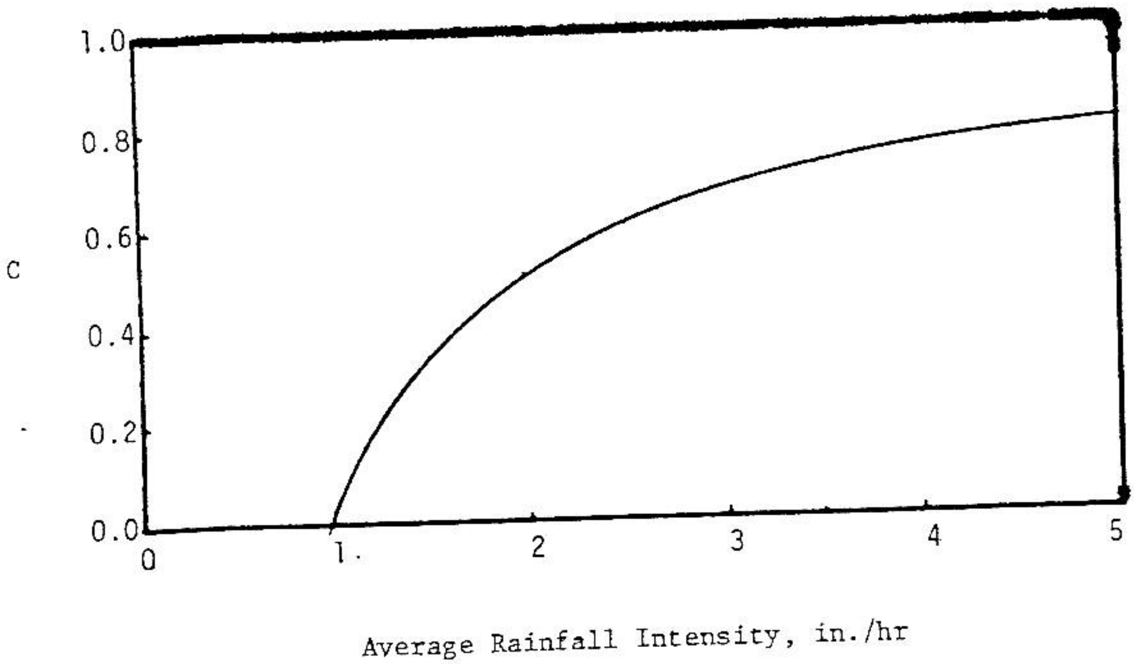

FIGURE 1 VARIATION OF C WITH i_T

Tables of C values published by various authors. Several tables have been published which enable users to estimate a value for C. The values can range from zero to 1.0, or more if rain falls on frozen ground, from no runoff to all rainfall becoming runoff. In the following tables, note that some include a range of values, but no directions are given to indicate what other parameters should be used to determine if the user should be at the low or high end of the range for his or her particular watershed. Note also the number and types of variables used in the tables. Table 2, Runoff Coefficients for Various Areas, was taken from a 1970 Concrete Pipe Design Manual.[6] Table 3, Coefficients of Runoff to be Used in the Rational Formula, was obtained from a highway engineering text by Ritter and Paquette.[7] Table 4, Runoff Coefficient C, came from a 1958 Concrete Pipe Handbook.[8]

TABLE 2

RUNOFF COEFFICIENTS FOR VARIOUS AREAS

Description of Areas	Runoff Coefficients
Business:	
Downtown areas	0.70 to 0.95
Neighborhood areas	0.50 to 0.70
Residential:	
Single-family areas	0.30 to 0.50
Multi units, detached	0.40 to 0.60
Multi units, attached	0.60 to 0.70
Residential (suburban)	0.25 to 0.40
Apartment dwelling areas	0.50 to 0.70
Industrial:	
Light areas	0.50 to 0.80
Heavy areas	0.60 to 0.90
Parks, cemetaries	0.10 to 0.25
Playgrounds	0.20 to 0.35
Railroad yard areas	0.20 to 0.40
Unimproved areas	0.10 to 0.30

TABLE 3

COEFFICIENTS OF RUNOFF TO BE
USED IN THE RATIONAL FORMULA

Type of Drainage Areas	Runoff Coefficients
Concrete and bituminous pavements	0.70 to 0.95
Gravel or macadam surfaces	0.40 to 0.70
Impervious soil	0.40 to 0.65
Impervious soils, with turf*	0.30 to 0.55
Slightly pervious soils*	0.15 to 0.40
Pervious soils*	0.05 to 0.10
Wooded areas (depending on slope and cover)	0.05 to 0.20

*For slopes from 1 to 2 percent.

TABLE 4

RUNOFF COEFFICIENT C

Type of Surfaces	C Values
USE FOR A CULVERT DESIGN	
Impervious surfaces	0.90 - 0.95
Steep barren surfaces	0.80 - 0.90
Rolling barren surfaces	0.60 - 0.80
Flat barren surfaces	0.50 - 0.70
Rolling meadow	0.40 - 0.65
Deciduous timberland	0.35 - 0.60
Conifer timberland	0.25 - 0.50
Orchard	0.15 - 0.40
Rolling farmland	0.15 - 0.40
Flat farmland	0.10 - 0.30
USE FOR AN AIRPORT DRAINAGE DESIGN	
Watertight roof surfaces	0.75 - 0.95
Asphalt runway pavements	0.80 - 0.95
Concrete runway pavements	0.70 - 0.90
Gravel or macadam pavements	0.35 - 0.75
Impervious soils (heavy)*	0.40 - 0.64
Impervious soils w/ turf*	0.30 - 0.55
Slightly pervious soils*	0.15 - 0.40
Slightly pervious soils w/ turf*	0.10 - 0.30
Moderately pervious soils*	0.05 - 0.20
Moderately pervious soils w/ turf*	0.00 - 0.10
USE FOR A STORM SEWER IN AN URBAN AREA	
Watertight surfaces, roofs & pavements	0.70 - 0.90
Block pavements w/ open joints	0.50 - 0.70
Macadam pavements	0.25 - 0.60
Gravel surfaces	0.15 - 0.30
Parks, cultivated lands, lawns, etc., dependent on slopes and character of soil	0.05 - 0.30
Wooded areas	0.01 - 0.02

*For slopes from 1 to 2 percent

Table 5, Average Runoff Coefficient for Use in the Rational Formula, is the table of runoff coefficients which appears in Manual No. 37 of the ASCE.[9] A footnote to this table indicates that these coefficients are applicable for storms of 5 to 10 year frequencies. Less frequent, higher intensity storms will require the use of higher coefficients because infiltration and other losses have a proportionally smaller effect on runoff. The coefficients are based on the assumption that the design storm does not occur when the ground is frozen. However, no instructions are given in the table as to how much higher the coefficients should be when a 25-, 50- or 100-yr storm is used for design.

Table 6, Runoff Coefficients for Use in the Rational Formula, was taken from the drainage manual of Erie and Niagara Counties in New York.[10] Table 7, Rational Method Runoff Coefficients for Composite Analysis, was obtained from the drainage manual for the City of Austin, Texas.[11] Note that additional variables have been added to these two tables: slope, soil type and frequency of occurence. With the addition of these three new variables, the runoff coefficient obtained from either of these two tables should more nearly reflect the hydrologic cycle.

Rather than a table, Ordon has presented a figure, reproduced here as Figure 2, Runoff Coefficient vs. Rainfall Intensity, to estimate C.[12] In his figure, C varies with rainfall intensity and land use. The family of curves drawn by Ordon are similar to the curve shown in Figure 1. While his curves are intuitively correct, he gives no details on how they were derived, except to say that they are based on data assembled from the literature. Recurrence interval is reflected somewhat in the rainfall intensity, but soil type and slope do nor appear in his curves. In his article, he did comment that his curves were based on low permeability soils with a high potential for runoff.

TABLE 5

AVERAGE RUNOFF COEFFICIENT FOR
USE IN THE RATIONAL FORMULA

Description of Use	Runoff Coefficients
Business:	
Downtown areas	0.70 to 0.95
Neighborhood areas	0.50 to 0.70
Residential:	
Single family areas	0.30 to 0.50
Multi-units, detached	0.40 to 0.60
Multi-units, attached	0.60 to 0.70
Residential (suburban)	0.25 to 0.70
Apartment dwelling units	0.50 to 0.70
Industrial:	
Light areas	0.50 to 0.80
Heavy areas	0.60 to 0.90
Parks, cemetaries	0.10 to 0.25
Playgrounds	0.20 to 0.40
Railroad vard areas	0.20 to 0.40
Unimproved areas	0.10 to 0.30

It is often desirable to develop a composite runoff coefficient based on the percentage of different types of surface in the drainage area.

Character of Surface	Runoff Coefficients
Streets:	
Asphaltic	0.70 to 0.95
Concrete	0.80 to 0.95
Brick	0.70 to 0.85
Drives and walks	0.85 to 0.85
Roofs	0.75 to 0.95
Lawns; Sandy soil:	
Flat, 2%	0.05 to 0.10
Average, 2% to 7%	0.10 to 0.15
Steep, 7%	0.15 to 0.20
Lawns; Heavy soil:	
Flat, 2%	0.13 to 0.17
Average, 2% to 7%	0.18 to 0.22
Steep, 7%	0.25 to 0.35

TABLE 6

RUNOFF COEFFICIENTS FOR USE IN THE RATIONAL FORMULA

Land Use	Hydrologic Soil Group and Slope Range											
	A			B			C			D		
	0-2%	2-6%	6% +	0-2%	2-6%	6% +	0-2%	2-6%	6% +	0-2%	2-6%	6% +
Industrial	0.67[1] 0.85[2]	0.68 0.85	0.68 0.86	0.68 0.85	0.68 0.86	0.69 0.86	0.68 0.86	0.69 0.86	0.69 0.87	0.69 0.86	0.69 0.86	0.70 0.88
Commercial	0.71 0.88	0.71 0.89	0.72 0.89	0.71 0.89	0.72 0.89	0.72 0.89	0.72 0.89	0.72 0.89	0.72 0.90	0.72 0.89	0.72 0.89	0.72 0.90
High Density[3] Residential	0.47 0.58	0.49 0.60	0.50 0.61	0.48 0.59	0.50 0.61	0.52 0.64	0.49 0.60	0.51 0.62	0.54 0.66	0.51 0.62	0.53 0.64	0.56 0.69
Medium Density[4] Residential	0.25 0.33	0.28 0.37	0.31 0.40	0.27 0.35	0.30 0.39	0.35 0.44	0.30 0.38	0.33 0.42	0.38 0.49	0.33 0.41	0.36 0.45	0.42 0.54
Low Density[5] Residential	0.14 0.22	0.19 0.26	0.22 0.29	0.17 0.24	0.21 0.28	0.26 0.34	0.20 0.28	0.25 0.32	0.31 0.40	0.24 0.31	0.28 0.35	0.35 0.46
Agricultural	0.08 0.14	0.13 0.18	0.16 0.22	0.11 0.16	0.15 0.21	0.21 0.28	0.14 0.20	0.19 0.25	0.26 0.34	0.18 0.24	0.23 0.29	0.31 0.41
Open Space	0.05 0.11	0.10 0.16	0.14 0.20	0.08 0.14	0.13 0.19	0.19 0.26	0.12 0.18	0.17 0.23	0.24 0.32	0.16 0.22	0.21 0.27	0.28 0.39
Freeways and Expressways	0.57 0.70	0.59 0.71	0.60 0.72	0.58 0.71	0.60 0.72	0.61 0.74	0.59 0.72	0.61 0.73	0.63 0.76	0.60 0.73	0.62 0.75	0.64 0.78

1 Lower runoff coefficients for use with storm recurrence intervals less than 25 years

2 Higher runoff coefficients for use with storm recurrence intervals of 25 years or more

3 High Density Residential - greater than 15 dwelling units per acre

4 Medium Density Residential - 4 to 15 dwelling units per acre

5 Low Density Residential - 1 to 4 dwelling units per acre

TABLE 7

RATIONAL METHOD RUNOFF COEFFICIENTS FOR COMPOSITE ANALYSIS

Character of Surface	Runoff Coefficient Design Coefficient for Storm Frequency of		
	5 - 10 Years	25 Years	100 Years
Streets			
Asphaltic	.80	.88	.95
Concrete	.85	.93	.95
Drives and Walks, Concrete	.85	.93	.95
Roofs	.85	.93	.95
Lawns, Sandy Soil			
Flat, 2%	.07	.08	.09
Average, 2-7%	.12	.13	.15
Steep, 7%	.17	.19	.21
Lawns, Clay Soil			
Flat, 2%	.18	.20	.22
Average, 2-7%	.22	.24	.27
Steep, 7%	.30	.33	.37
Undeveloped Woods & Pasture			
Sandy Soil			
Flat, 2%	.12	.13	.15
Average, 2-7%	.20	.22	.25
Steep, 7%	.30	.33	.37
Clay Soil			
Flat, 2%	.30	.33	.37
Average, 2-7%	.40	.44	.50
Steep, 7%	.50	.55	.62

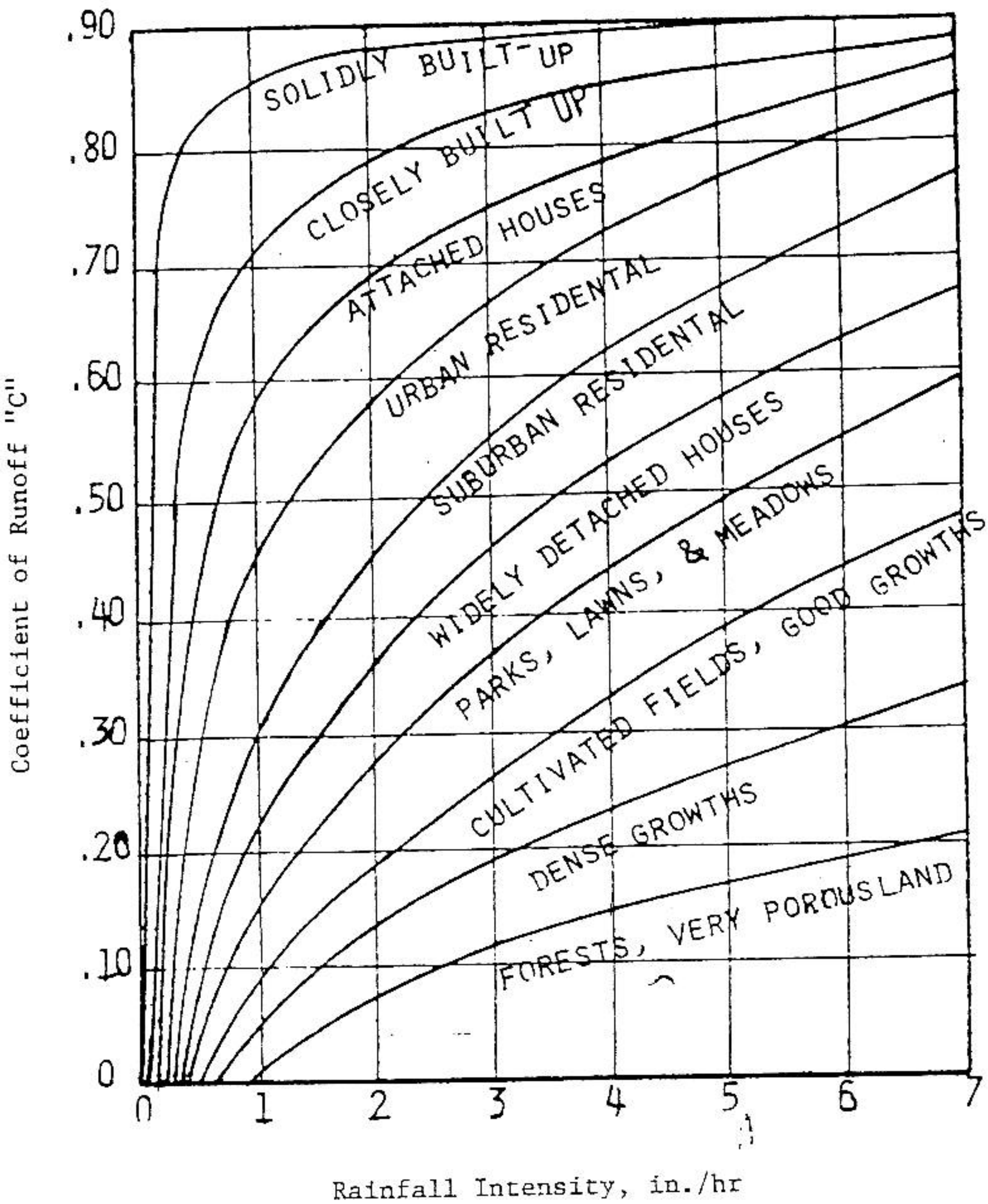

FIGURE 2 RUNOFF COEFFICIENT VS. RAINFALL INTENSITY

Rossmiller's equation for the runoff coefficient C. Each of the preceeding tables has one or more shortcomings: some do not include essential variables, some do not explain how to select a particular value from a given range of values, some do not include particular land uses. While some writers have tried to solve these deficiencies as shown in Tables 6 and 7, there is still a lack of agreement for a certain set of conditions.

A number of variables should be used to estimate C. These include land use, soil type, antecedent moisture condition, recurrence interval, imperviousness of the watershed, rainfall intensity, watershed slope and surface roughness. Each of the variables, acting in concert with some of the others, affects the portion of rainfall which will appear as runoff. As an aid to more uniform estimation, the following empirical equation is proposed for estimating the runoff coefficient C.

$$C = 7.2(10)^{-7} CN^{3} RI^{.05} ((.01CN)^{.6})^{-S^{.2}} (.001CN^{1.48})^{.15-.1I} ((Imp+1)/2)^{.7} \qquad (6)$$

where C is the runoff coefficient, a dimensionless decimal between zero and 1.00; CN is the SCS curve number, a dimensionless integer between zero and 100; RI is the recurrence interval in years; S is the average land slope of the watershed in percent, i.e., for a 4% slope, S = 4; I is the rainfall intensity in inches per hour; and Imp is the watershed imperviousness, a dimensionless decimal between zero and 1.00, i.e., for 20% imperviousness, Imp = 0.20. The SCS curve number is calculated from equation (7).

$$CN = 98Imp + X(1-Imp) \qquad (7)$$

where X is a dimensionless integer which varies with the SCS hydrologic soils group (HSG) as shown in Table 8, Variation of X with the SCS Soils Group.

TABLE 8

VARIATION OF X WITH THE SCS SOILS GROUP

HSG	X
A	39
B	61
C	74
D	80

The first two terms in equation (6) yield a basic runoff coefficient. The next three terms adjust this basic value for the effects of frequency, slope and rainfall intensity, respectively. As these variables increase, the value of C also increases. The form of the fourth and fifth terms takes into account the tendency for the effect of increased slope and rainfall intensity to be less and less as the runoff potential of the surface becomes greater and greater. The last term takes into account the surface roughness. As the imperviousness of the watershed increases, the surface becomes smoother, thus increasing the amount of runoff. Also, as imperviousness increases, more and more of this area becomes interconnected which allows more water to reach the point of design.

The formula yields values which range from 0.04 to 0.95 and is based on the assumption that the rain falls on ground which is not frozen.

Rainfall Intensity i_T

As stated before, a common definition of i_T has led to many misconceptions: the rainfall intensity in inches per hour of a storm whose duration is equal to the time of concentration of the watershed. This intensity is assumed to be uniform over the time period equal to t_c.

Current practice is to compute t_c by some method, then from an I-D-F curve prepared by the Weather Bureau for the design location, pick off a rainfall intensity for some desired frequency and a "duration" equal to t_c. What has been lost sight of in present-day use of the rational formula is that the intensities taken from an I-D-F curve are simply maximum average intensities over some time periods and bear no relation to sequential rainfall in an actual storm. Also, I-D-F curves yield <u>average</u> intensities. The actual intensities may have varied considerably during the "duration" shown on an I-D-F curve. This is due to the manner in which the I-D-F curves were derived. The following explanation of the development of I-D-F curves was taken from Hjelmfelt and Cassidy.[13]

1. Precipitation also varies with time within each particular storm, and the duration (total time during which rain falls) varies from storm to storm; therefore, analysis of precipitation at a point must involve both the amount (depth) of rain that falls and the elapsed time (duration) during which that amount fell. This is called intensity-duration analysis and proceeds in the following manner. The rainfall record from a recording rain gage is listed in Table 9, Precipitation Data in Inches. A particular duration is selected and the maximum rainfall for this time is determined. The maxima for all storms are listed in order of descending magnitude. Table 10, Frequency Analysis of Exceedence Values, is an example of an analysis of a 10-minute duration rainfall for Chicago, Illinois; column 1 is the order number m, column 2 is the rainfall in the most intense 10 minutes y, and column 3 is the return period assigned to each rainfall T_r. This is a partial-duration series; therefore, the return period is given by the formula $T_r = N/m$, N = years of record.

2. Next, the same type of analysis is carried out for a different duration, say 30 minutes. The 30-minute values may or may not include the 10-minute values of the preceding analysis. A frequency distribution is constructed from the 30-minute values, and the process is continued for other durations. The manner in which the precipitation data is reported has changed through the years, and modification of the record may be needed to put all the data on the same basis.

TABLE 9

PRECIPITATION DATA IN INCHES

Date	Year	Duration in Minutes													
		5	10	15	20	25	30	35	40	45	50	60	80	100	120
July 14	1913	0.16	0.29	0.40	0.50	0.59	0.67	0.74	0.79						
Aug. 7		0.31	0.37												
Aug. 7-8		0.30	0.44	0.56											
Aug 18		0.28	0.49	0.63	0.67										
Apr. 27	1914	0.27													
May 27		0.18	0.33	0.41	0.49										
Jun 4		0.21	0.35	0.40											
Jul 16		0.33	0.66	0.79	0.97	1.21	1.48	1.61							
Aug. 9		0.35	0.62	0.83	0.91										
Aug. 13		0.19	0.36	0.50	0.60	0.68									
Sept. 1		0.14	0.27	0.38	0.40										
May 15	1915	0.17	0.25	0.32	0.40	0.48									
Jun 12		0.18	0.31	0.46	0.56	0.76	0.82	0.89	0.92	0.98					
Data from July 7, 1915 through July 12, 1947 were listed and analyzed but are not shown here.															
Jul 13	1947	0.31	0.44	0.57	0.62	0.64	0.66	0.68	0.70	0.72	0.73	0.75	0.81	0.84	0.84
Aug. 29		0.36	0.60	0.72	0.77	0.81	0.83	0.85	0.87	0.89	0.91				
Sept 11		0.25	0.50	0.72	0.77	0.77	0.78								
Sept 21		0.13	0.23	0.33	0.42	0.50	0.57	0.62	0.67	0.72	0.77	0.85	0.98	1.13	1.24
Oct. 26		0.19	0.29	0.38	0.43	0.45	0.46	0.48	0.49	0.49	0.50	0.53	0.55	0.56	

TABLE 10

FREQUENCY ANALYSIS OF EXCEEDENCE VALUES

(10-min duration rainfall depth)

(1) m	(2) Rainfall in.	(3) T_r years
1	1.11	35.00
2	0.96	17.50
3	0.94	11.67
4	0.92	8.75
5	0.88	7.09
6	0.80	5.83
7	0.80	5.00
8	0.76	4.38
9	0.74	3.84
10	0.74	3.50
11	0.71	3.18
12	0.70	2.92
13	0.68	2.69
14	0.68	2.50
15	0.68	2.33
16	0.67	2.19
17	0.66	2.04
18	0.66	1.94
19	0.66	1.81
20	0.65	1.75
21	0.64	1.67
22	0.64	1.59
23	0.63	1.52
24	0.62	1.46
25	0.62	1.40
26	0.61	1.35
27	0.60	1.30
28	0.60	1.25
29	0.59	1.21
30	0.59	1.17
31	0.58	1.13
32	0.58	1.09
33	0.57	1.06
34	0.57	1.03
35	0.57	1.00

3. Finally, the rainfall depths determined above are converted to intensities. These rainfall intensities, for each duration and frequency, are plotted and smooth curves drawn through points of like frequency, thus producing the family of curves which are known as rainfall intensity-duration-frequency (I-D-F) curves, such as shown in Figure 3, Typical Rainfall I-D-F Curves.

Note that in the last step, the rainfall depths are converted to intensities. For m = 1 in Table 10, the depth of 1.11 in. converts to an intensity of 6.66 in./hr (1.11 x 60/10). The calculation yields an <u>average</u> intensity of 6.66 in./hr. No presumption is made that the rain fell at a uniform intensity during the 10-minute period. An examination of the pen trace on the original record would reveal whether or not the intensity was uniform. Actual intensities could have ranged from 4 to 10 in./hr, for example, during the 10-minute period. This possibility is obscured in the I-D-F curves and the false assumption is made that the rainfall intensity is uniform during the time period equal to t_c.

<u>Rainfall Intensity Averaging Time t_c</u>

In the preceeding discussion, t_c has been referred to as the time of concentration. This was done because of its greater familiarity to practicing engineers. However, henceforth t_c will be referred to as the rainfall intensity averaging time for the following reason: these words more accurately define the reason for and the use of this variable. T_c is merely a time period from which an average rainfall intensity is identified. T_c is not the total duration of a storm. T_c is simply a period of time within some total storm duration during which the maximum average rainfall intensity occurs. This time period could occur near the beginning, middle or end of a storm.

The rainfall intensity averaging time <u>is</u> tied to the magnitude of the drainage area because of the commonly used definition of t_c: the time required for water to flow from the hydraulically most distant point in the watershed to

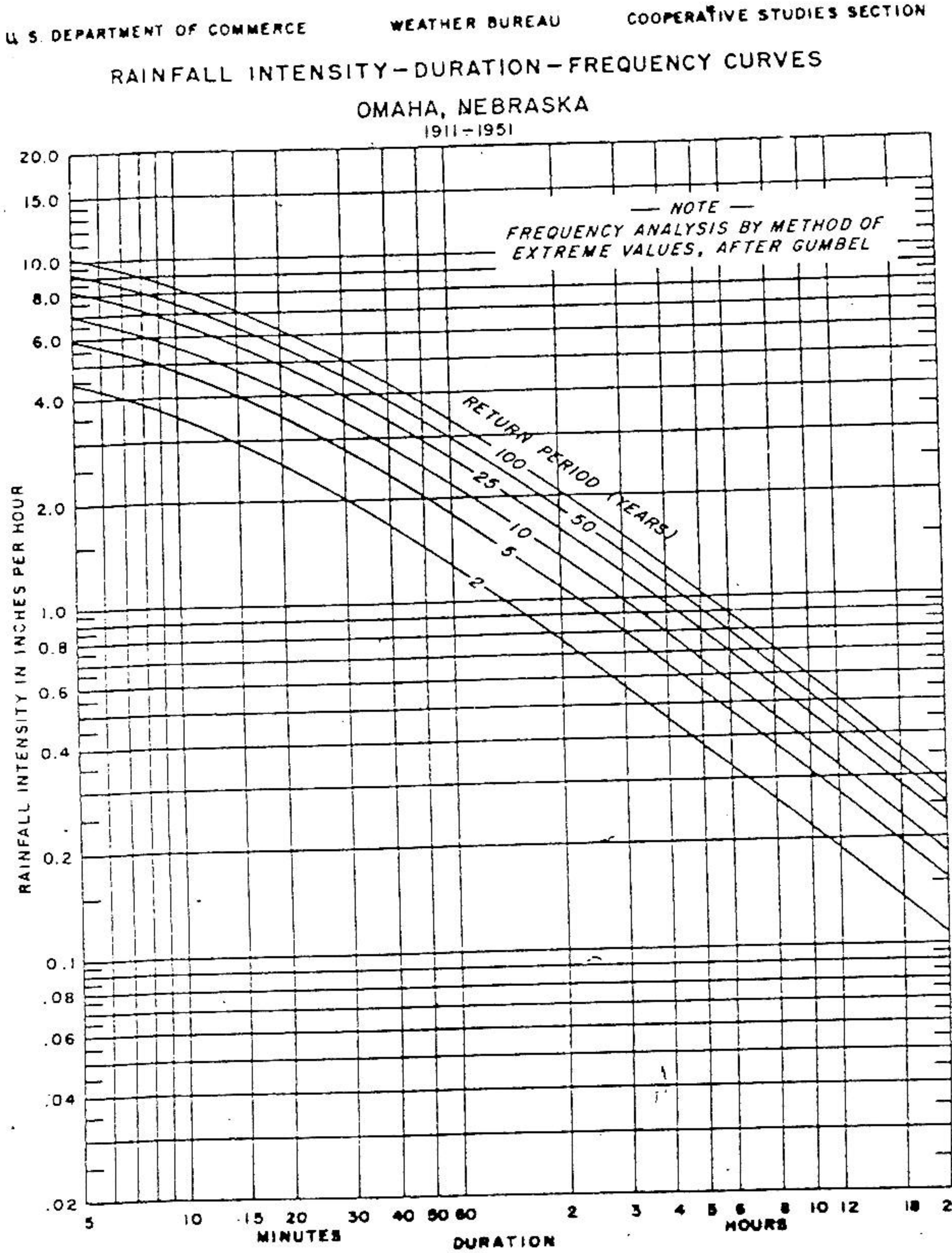

FIGURE 3 TYPICAL RAINFALL I-D-F CURVES

the point of design. A better definition would be: the time required for water to flow from the hydraulically most distant point of that tributary watershed which produces the greatest discharge to the point of design.

The rainfall intensity averaging time consists of an inlet time plus a flow time. Inlet time consists of the time required for water flowing overland to reach established surface drainage channels, such as ditches and street gutters, plus travel time through them to the point of inlet to a storm sewer. Flow time is the time of flow through the storm sewer to the point of design. The inlet time can be estimated from one of several formulas devised for this purpose and/or from Manning's formula. The flow time can be estimated from Manning's formula.

Various writers have used one or more variables to estimate the rainfall intensity averaging time. A compilation of these variables yields the following list.

1. surface slope
2. stream slope
3. length of surface flow
4. length of stream from the point of interest to the divide
5. antecedent rainfall
6. interception storage
7. depression storage
8. infiltration capacity of the soil
9. surface roughness
10. rainfall intensity
11. drainage area
12. imperviousness

Even though most of the following equations purportedly yield the rainfall intensity averaging time, some of them estimate overland flow time only, some estimate inlet time and some estimate t_c.

Kirpich.[14] In the past, this has been the most commonly used formula for estimating the rainfall intensity averaging time. The formula is

$$t_c = 0.0078(L/(S)^{0.5})^{0.77} \tag{8}$$

where t_c is in minutes, L is the length of travel in feet and S is the slope in feet per foot, the difference in elevation in feet between the most remote point and the outlet divided by the horizontal distance between these two points in feet.

The formula is based on a 1927 article by Ramser in the Journal of Agricultural Research.[15] Ramser obtained his data from six small watersheds ranging in size from 0.5 to 45.4 ha (1.2 to 112 acres) on the Murchison farm near Jackson, Tennessee. The slopes on the farm were steep, the soils were droughty and the timber cover ranged from zero to 56 percent.

Because of its data base, some references suggest that the following modifications be made to the estimated t_c.

1. Use the equation t_c for natural basins with well defined channels, for overland flow on bare earth and for mowed grass roadside channels.
2. For overland flow, grassed channels, multiply t_c by 2.
3. For overland flow, concrete or asphalt surfaces, multiply t_c by 0.4.
4. For concrete channels, multiply t_c by 0.2.

Several other equations which have the same general form as the Kirpich formula are listed below.

$$T_p = 0.00054(L/(S)^{0.5})^{0.64} \tag{9}$$

where T_p is the time to peak in hours.

$$t_c = 0.00587(L/(S)^{0.5})^{0.8} \tag{10}$$

$$t_c = 0.0046(L/(S)^{0.5})^{0.77} \tag{11}$$

$$t_c = 0.0078L^{1.153}/H^{0.387} \tag{12}$$

where H is the difference in elevation in feet between the most remote point and the outlet.

$$t_c = 3000n(L/(S)^{0.5})^{0.6} \tag{13}$$

where n is Manning's friction factor, a dimensionless decimal.

An equation commonly used throughout the United States was formulated by Rowe.[16] However, it is simply a manipulation of Kirpich's formula with t_c in hours and L in miles.

$$t_c = (11.9L^3/H)^{0.385} \tag{14}$$

The author's opinion is that the Kirpich formula, including all of the above variations, should only be used in rural areas. They should not be used to estimate the rainfall intensity averaging time in urban areas.

Drainage Area. Another whole series of equations is based on the drainage area to some power modified by a coefficient and, in some cases, by other factors as well. These equations are listed below.

$$t_c = 0.9A_w^{0.6} \tag{15}$$

where t_c is in hours and A_w is the watershed area in square miles.

$$t_r = 10A_w^{0.41} \tag{16}$$

where t_r is the time of rise in hours.

$$t_r = 25.3A_w^{-0.14} \tag{17}$$

$$t_p = 1.05A_w^{0.6} \tag{18}$$

where t_p is the lag time in hours.

$$t_p = 0.58A_w^{0.75} \tag{19}$$

$$t_p = 0.57A_w^{0.38} \tag{20}$$

$$t_c = 0.70A_w^{0.55} \tag{21}$$

$$t_c = 1.45A_w^{0.445} \tag{22}$$

$$t_c = 5 + 75A_w^{0.50} \tag{23}$$

where t_c is in minutes for equation (23).

$$t_c = 1.5A_w^{0.50} \tag{24}$$

where t_c is in minutes and A_w is in acres.

$$t_c = 2.7(A_w/sl)^{0.5} \tag{25}$$

where t_c is in hours, A_w is in square miles and sl is the average watershed slope in percent.

$$t_p = 106(A_w^{0.3}/sl(DD)^{0.5})^{0.61} \tag{26}$$

where A_w is the watershed area in acres and DD is the drainage density (total length of visible channels per unit area).

This author's opinion is that all of this series of equations should only be used in rural areas. None of these equations should be used to estimate the rainfall intensity averaging time in urban areas.

Federal DOT.[17] The Aviation Agency of the Federal DOT has suggested the following formula.

$$t_c = 1.8(1.1-C)D^{0.5}/S^{0.333} \tag{27}$$

where t_c is in minutes, C is the rational formula runoff coefficient, a dimensionless decimal, D is the distance in feet and S is the slope in percent.

SCS Upland Method.[18] The US Soil Conservation Service upland method considers the following types of flow: overland, through grassed waterways, over paved areas and through small upland gullies. The formula is

$$t_c = \sum_{i=1}^{n} L_i/V_i \tag{28}$$

where t_c is measured in seconds, L_i is the length of reach i in feet and V_i is the average velocity in reach i in feet per second. The SCS has prepared a figure, included here as Figure 4, Velocities for SCS Upland Method of Estimating t_c, for determining the velocity of flow based on slope in percent and type of flow path. This formula can be used to estimate both overland flow time and inlet time.

SCS Curve Number Method.[18] The SCS has developed this equation to span a broad set of conditions ranging from heavily forested watersheds with steep channels and a high percent of the runoff resulting from subsurface or inter-flow and meadows providing a high retardance to surface runoff, to smooth land surfaces and large paved parking lots. The formula is

$$t_c = L/0.6 \tag{29}$$

$$L = \ell^{0.8}(S+1)^{0.7}/1900Y^{0.5} \tag{30}$$

where L is the basin lag in hours, ℓ is the hydraulic length of the watershed in feet, Y is the average watershed land slope in percent and S is determined from equation (31).

$$S = (1000/CN') - 10 \tag{31}$$

where CN' = CN and CN is the SCS curve number, a dimensionless number between zero and 100. The SCS has prepared Figure 5, SCS Curve Number Method for Estimating Lag, to solve equation (30). This formula can also be used to

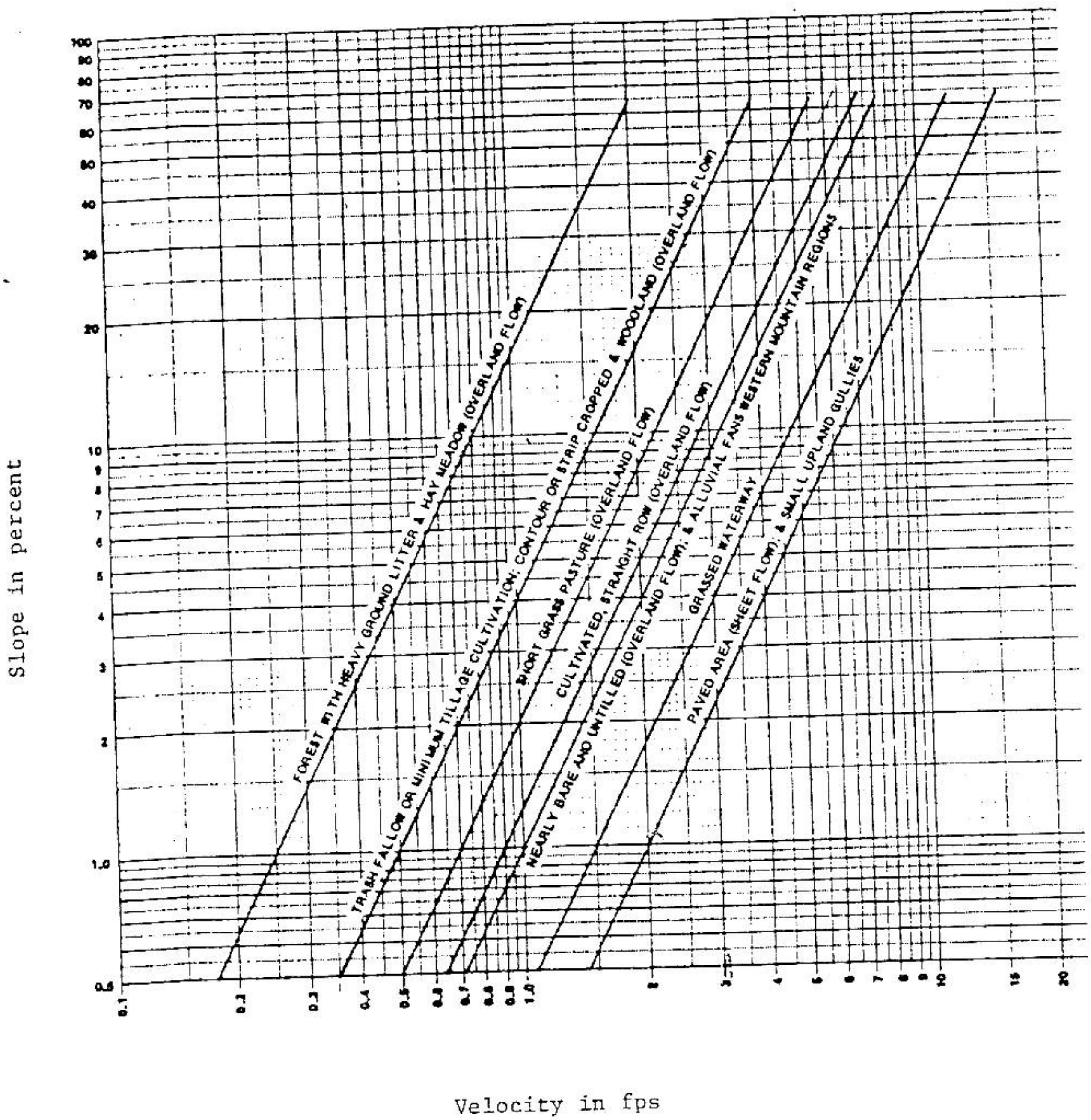

FIGURE 4 VELOCITIES FOR SCS UPLAND METHOD OF ESTIMATING t_c

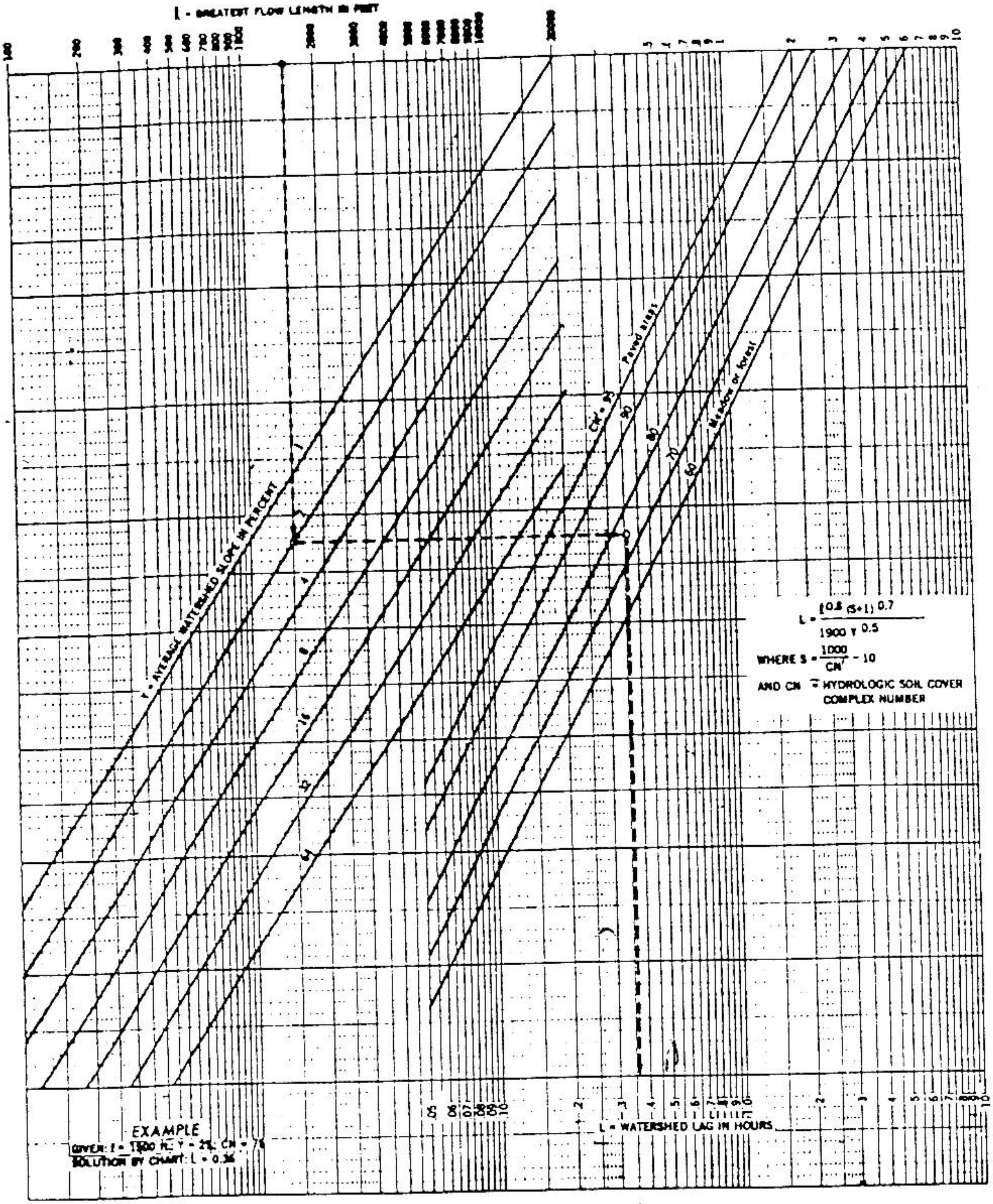

FIGURE 5 SCS CURVE NUMBER METHOD FOR ESTIMATING LAG (L)

estimate both overland flow time and inlet time.

Kerby.[19] This formula is based on Hathaway's 1945 article on the drainage of military airfields.[20] The formula is

$$t_c^{2.14} = 2NL/3S^{0.5} \tag{32}$$

or

$$t_c = 0.83(NL/S^{0.5})^{0.467} \tag{33}$$

where t_c is measured in minutes; L is the length in feet, measured from the extremity of the catchment area in a direction parallel to the slope until a defined channel is reached; S is the slope in feet per foot, the difference in elevation between the extreme point of the catchment area and the point in question, divided by the horizontal distance between the two points; and N is a coefficient of roughness taken from Table 11, Values of N in the Kerby formula.

TABLE 11

VALUES OF N IN THE KERBY FORMULA

C	N	Type of Surface
0.90	0.02	smooth impervious surfaces
0.50	0.10	smooth bare packed soil, free of stones
0.45	0.20	poor grass, cultivated row crops or moderately bare surfaces
0.40	0.40	pasture or average grass cover
0.35	0.60	deciduous timberland
0.30	0.80	conifer timberland, deciduous timberland with deep forest litter or dense grass cover

CHOW p. 498 ±

N appears to be roughly in the range $0.85 - C \geq N \geq 0.7 - C$

$N \cong 0.0125C^{-3.533}$ CORR = −0.9874

The t_c estimated by this equation is for overland flow only. Kerby states:[19]

> "If channelized flow occurs in a catchment area, the time of concentration will be the time of overland flow plus the time within the channel."

Kinematic Wave Formulation.[21] Ragan and Duru have developed the following equation using kinematic wave theory.

$$t_c = 0.93L^{0.6}n^{0.6}/i^{0.4}S^{0.3} \tag{34}$$

where t_c is in minutes, L is the length in feet, n is Manning's roughness coefficient, a dimensionless decimal, i is the rainfall intensity in inches per hour and S is the slope in feet per foot.

The equation should be used only for overland flow and works best for turbulent flow on a homogeneous surface. Since t_c is used with a rainfall I-D-F curve to determine i, and since i is a variable in the equation, the estimation of t_c in equation (34) is a trial and error process.

Author's Recommendation. This author recommends, based on our present knowledge, that Kerby's formula and/or either of the Soil Conservation Service's formulas be used to estimate overland flow time and inlet time. Inlet time and flow time can be estimated from Manning's equation.

Drainage Area A

The area to be used in the rational formula is the drainage area in acres tributary to the point of design which produces the maximum peak flow rate. Several questions must be answered in order to determine the tributary area from a map of some type.

1. Will the existing contour lines remain the same or will the area be regraded?
2. How are individual lots in a subdivision to be graded: from rear to front, half to rear and half to front, what?
3. Which way will water run along the rear of the lots and along the street gutters?
4. At intersections, does water turn the corner or flow across the intersec~~tion~~?
5. With higher intensity rainfalls, the 100-yr instead of the 5-yr, will the water flow the same ways at the intersections?

Some designers assume that the drainage area consists of only those impervious areas which are interconnected and lead to a storm sewer inlet. Some examples follow.

1. Streets
2. Driveways connected to streets
3. Parking areas connected to driveways which are connected to streets
4. Parking areas with inlets which are connected to pipes which lead to a storm sewer
5. Garage, house and building roofs whose downspouts direct runoff onto driveways which are connected to streets
6. Garage, house and building roofs whose downspouts direct runoff onto parking areas which are connected to driveways which are connected to streets
7. Garage, house and building roofs whose downspouts direct runoff onto parking areas with inlets which are connected to pipes which lead to a storm sewer.

These interconnected impervious areas may produce larger discharges because they have a shorter t_c (than the total tributary area which includes pervious surfaces) which yield higher rainfall intensities which more than compensate for the decrease in total drainage area. Therefore, the discharge is larger.

Use of the Formula for Storm Sewer Design

Stormwater runoff in urban areas is conveyed in two systems: the minor system and the major system. The minor system consists of street gutters, storm sewers and small open channels. The major system is utilized whenever the capacity of the minor system is exceeded. Flow paths in the major system can and do include streets, minor and major drainage swales, homes and basements, parking lots, shopping centers and other commercial areas, creeks and streams, industrial areas and rivers.

Traditionally, much time, effort and money have been spent on the design, and construction of the minor system. The same is not true of the major system. Traditionally, the minor system has been designed using the rational formula. The sequence of steps to be used in designing the storm sewer portion of the minor system should include all of the following.

First, prepare a map of the area to be drained which includes streets, lot lines, underground utilities and arrows which denote all directions of flow for each lot and street.

Second, sketch one or more tentative minor systems to drain the entire area which includes the streets and locations of minor swales and storm sewers. Also include the tentative locations of all inlets to the storm sewers.

Third, sketch in the drainage area tributary to each inlet for each alternative system layout. For each tributary area, record the following information: total area in acres, area of interconnected impervious area in acres, percent of impervious area, lengths of flow paths for both areas, average watershed land slopes for both areas, channel slopes for both areas, soil type(s), land use(s) and SCS curve numbers for both areas.

Fourth, select the recurrence interval(s) to be used in the design. This could be 2 or 5 years for residential areas, 5 or 10 years for commercial and industrial areas and major streets. The major system should be designed for the 100-year storm.

Fifth, estimate the rainfall intensity averaging times for each inlet for both the total tributary area and the interconnected impervious area only. Determine the rainfall intensity for each t_c.

Sixth, determine the runoff coefficient C for each inlet for both areas.

Seventh, determine the peak discharge rate for each inlet for both areas. Record the larger of the two.

Eighth, determine the depth of flow in each channel, swale or gutter at each inlet. Determine whether or not any depth causes the width of flow to exceed established criteria. If some do, then go back to step 2 and add additional inlets where necessary. Repeat steps 3 through 7 for each inlet that was changed and/or added.

Ninth, size the uppermost inlet and record the discharge which enters the inlet and the discharge which bypasses the inlet. Size each inlet in turn, moving in a downstream direction, using the discharge determined in step 7 plus the discharge which bypassed the upstream inlet. Record all discharges and inlet sizes and types.

Tenth, design the connector pipes between the inlets and the storm sewers. Record the tentative lengths, sizes, inlet and outlet elevations and head losses.

Eleventh, beginning at the upstream end, size each pipe segment in the storm sewer system according to local criteria. The discharge to be used is not the summation of the discharges entering the upstream inlets. Rather, new discharges must be determined for each pipe segment using the tributary area, composite C value and rainfall intensity corresponding to the total rainfall intensity averaging time to that point in the system. Both total area and interconnected inpervious area only values for C, i and A should be checked. Record the tentative lengths, sizes, inlet and outlet elevations for each pipe segment.

Twelveth, determine and record the minor head losses at all points in the system. These losses will occur at manholes, bends, junctions, inlets and transitions.

Thirteenth, beginning at the downstream end of the system with some known water surface elevation, determine the elevation of the hydraulic grade line at every point in the system. If local criteria for depth of the hydraulic grade line below street grade and/or inlets are violated at one or more points in the system, make whatever revisions are necessary in sizes, invert elevations and/or slopes of the affected segments of the storm sewer and/or connector pipes until all criteria are met.

Note that the rational formula is used four times, in steps 3 through 7 and in step 10, in the design process. It is used first to design the inlets and then again to design the storm sewer itself. The first two times through, A, t_c, i and C refer to each individual subwatershed; the third and fourth times, A, t_c, i and C refer to the composite effect of all subwatersheds upstream of the pipe segment currently being designed.

Use of the Formula for Determining Peak Discharge Rates

Figure 6, Subdivision Layout, shows a proposed subdivision. The lots are zoned for single-family residences, average about 1/4 acre in size and slope towards the streets at a 1.0 percent slope. The streets are 31 feet wide in a 60-foot right-of-way with 4-foot sidewalks on both sides of the street. Both Ring Circle and Irey Swing slope towards Dougal Drive at 1.0 percent grades. Carol Court also slopes towards Dougal Drive: a 0.4 percent slope from Sta. 0+00 to Sta. 2+00 and a 4.5 percent slope from Sta. 2+00 to Sta. 4+00. Most of Dougal Drive slopes towards the south; a high point occurs at Sta 8+70. Dougal Drive has a slope of 0.4 percent between Sta. 0+00 and Sta. 2+75 and a slope of 2.7 percent north of Sta. 2+75. Austin Avenue slopes downwards to the west at 1.5 percent. Each lot contains a 1,600 sq ft home, a 400 sq ft garage and a 200 sq ft patio; however, the downspouts from these structures drain on to grass.

Figure 6 also shows the proposed layout of the storm sewer system. The storm sewer itself is shown as a solid line, the catch basins as squares and the manholes as circles. An existing storm sewer in Austin Avenue is shown as a dashed line. With all lots sloping towards the streets, the area tributary to each catch basin is shown on Figure 7, Subdivision Layout Showing Area Tributary to Each Catch Basin. Also shown on Figure 7 are arrows indicating the flow paths used in determining the rainfall intensity averaging times for each subarea.

Table 12, Impervious and Total Area of Each Subarea in the Subdivision, lists the size of each subarea. Interconnected impervious area includes only the streets and driveways. Other impervious area includes homes, garages and patios. Total impervious area ranges from 33 percent to 44 percent and

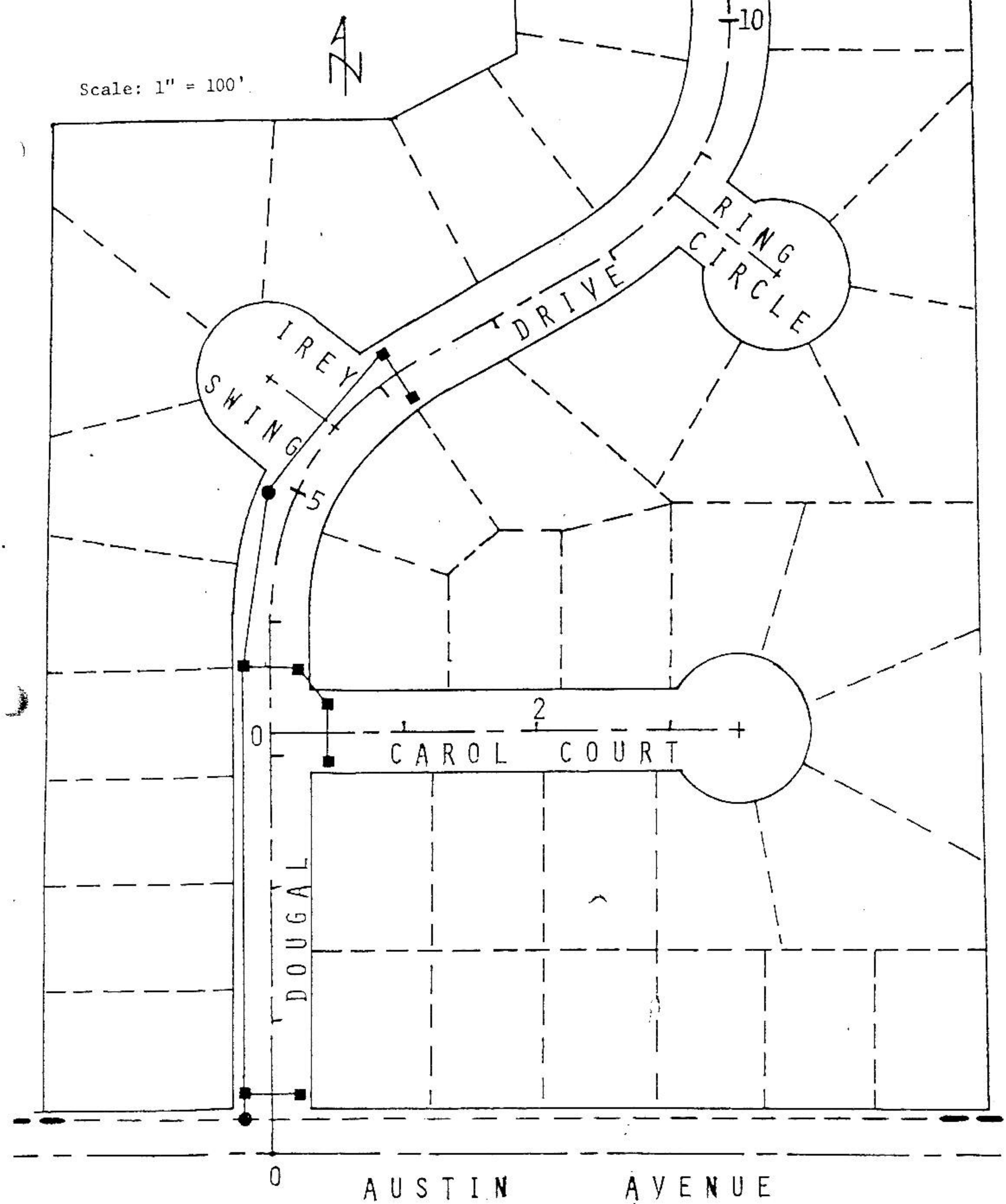

FIGURE 6 SUBDIVISION LAYOUT

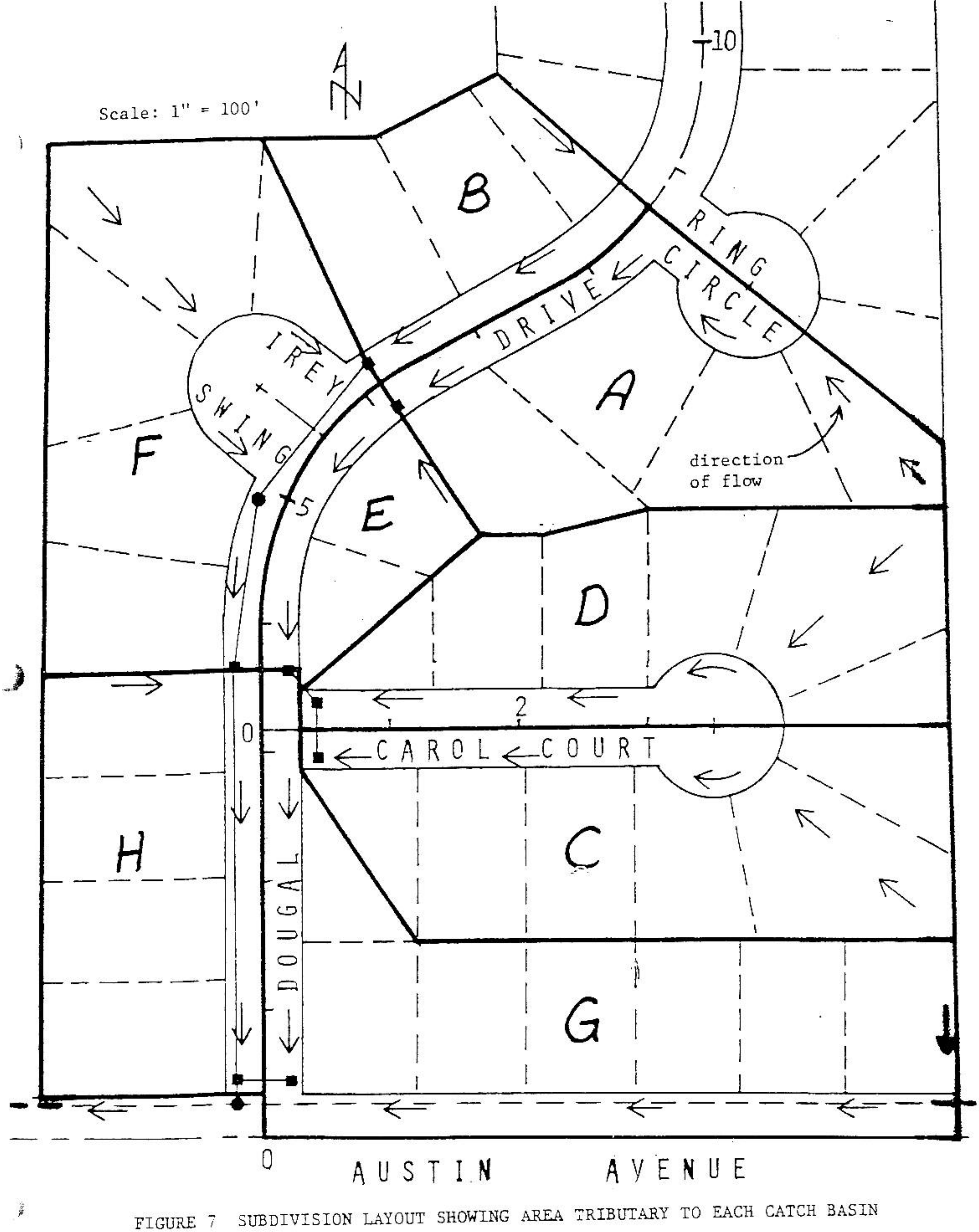

FIGURE 7 SUBDIVISION LAYOUT SHOWING AREA TRIBUTARY TO EACH CATCH BASIN

TABLE 12

IMPERVIOUS AND TOTAL AREA OF
EACH SUBAREA IN THE SUBDIVISION

Subarea	Impervious Area			Total Area	Imper-viousness
	Inter-Connected acres	Other acres	Total acres	acres	%
A	0.32	0.18	0.50	1.5	33
B	0.17	0.13	0.30	0.9	33
C	0.35	0.25	0.60	1.8	33
D	0.35	0.25	0.60	1.7	35
E	0.14	0.08	0.22	0.5	44
F	0.40	0.23	0.63	2.0	32
G	0.55	0.33	0.88	2.2	40
H	0.26	0.20	0.46	1.4	33
Total	2.54	1.65	4 19	12.0	

averages 35 percent for the 12-acre subdivision. With 32 lots in the subdivision, each lot averages about 0.4 acre, including the street rights-of-way.

Table 13, Rainfall Intensity Averaging Times for Interconnected Impervious Area Only for Each Subarea, contains the values needed to determine the rainfall intensity averaging times. This includes overland flow time in the driveways plus travel time in the gutter. Manning's "n" value for both the driveways and gutters is assumed to be 0.016. Driveway slopes are 1.0 percent; the street slopes were listed above, cross slopes are 2.0 percent. Lengths of flow were taken from Figure 7. Kerby's formula, equation (33), was used to determine overland flow on the driveways. Velocity of flow in the gutters was determined from equation (35). Average depth of gutter flow was assumed to be 3 inches.

$$V = 1.12S_o^{1/2}d^{2/3}/n \tag{35}$$

where V is the velocity of flow in the gutter in feet per second, S_o is the longitudinal street slope expressed as a decimal, d is the depth of flow in feet and n is Manning's roughness factor expressed as a decimal. In Table 13, t_o is overland flow time, t_g is gutter flow time and t_i is the total inlet time (rainfall intensity averaging time), all measured in minutes. A minimum inlet time of 5 minutes is used.

Table 14, Rainfall Intensity Averaging Times for Total Area of Each Subarea, contains the values needed to determine the rainfall intensity averaging times. This includes overland flow time on the grass plus travel time in the gutter. Kerby's formula and equation (35) were used to calculate the flow times. Note that in Table 14, t_i varies from 16 to 21 minutes while in Table 13, t_i is 5 minutes for all subareas. This results in a large

TABLE 13

RAINFALL INTENSITY AVERAGING TIMES FOR INTER-CONNECTED IMPERVIOUS AREA ONLY FOR EACH SUBAREA

Subarea	Driveway			Gutter Flow				Gutter Flow			t_i
	S_o ft/ft	L ft	t_o min	S_g ft/ft	L ft	t_g min		S_g ft/ft	L ft	t_g min	min
A	.01	45	2.3	.010	135	0.8		.027	255	0.9	5.0
B	.01	45	2.3	.027	255	0.9					5.0
C	.01	45	2.3	.045	205	0.6		.004	170	1.6	5.0
D	.01	45	2.3	.045	205	0.6		.004	170	1.6	5.0
E	.01	45	2.3	.027	220	0.8					5.0
F	.01	45	2.3	.010	90	0.5		.027	230	0.8	5.0
G	.01	45	2.3	.015	500	2.4	or	.004	300	2.8	5.0
H	.01	45	2.3	.004	300	2.8					5.0

TABLE 14

RAINFALL INTENSITY AVERAGING TIMES
FOR TOTAL AREA OF EACH SUBAREA

Subarea	Driveway			Gutter Flow			Gutter Flow			t_i
	S ft/ft	L ft	t_o min	S ft/ft	L ft	t_o min	S ft/ft	L ft	t_o min	min
A	.01	190	18.4	.010	135	0.8	.027	255	0.9	20
B	.01	145	16.2	.027	255	0.9				17
C	.01	210	19.3	.045	180	0.5	.004	170	1.6	21
D	.01	210	19.3	.045	180	0.5	.004	170	1.6	21
E	.01	125	15.2	.027	250	0.9				16
F	.01	215	19.5	.010	90	0.5	.027	230	0.8	21
G	.01	140	16.0	.015	500	2.4				18
H	.01	160	17.0	.004	320	3.0				20

difference in the rainfall intensities used in the rational formula.

Table 15, Values of C for Interconnected Impervious Area Only for Each Subarea, lists the values of the variables used to determine the runoff factor C for the interconnected inpervious area only for each subarea. These variables are used in equation (6) to determine C. The soils in the subdivision are in the SCS hydrologic soils group B. A 5-year recurrence interval is used to design these storm sewers in a residential area. C is constant at 0.78. Table 16, Values of C for Total Area of Each Subarea, lists the values needed to determine the runoff factor C for the total area of each subarea. C now varies from 0.30 to 0.37.

The peak flow rates for each subarea for the interconnected impervious area only and total area are determined in Table 17, Peak Inflow Rates for Interconnected Impervious Area Only for Each Subarea, and Table 18, Peak Inflow Rates for Total Area of Each Subarea, respectively. Coincidentally, the flow rates in each subarea are about equal in the two tables. Note that if the total impervious area of each subarea as listed in Table 12 had been used in Table 17, the peak flow rates would have increased an average of about 65 percent. Thus, by directing the roof downspouts onto grass, the peak flow rates have been reduced.

Storm Sewer

The discharges determined in Tables 17 and 18 are used to size the gutter inlets. In order to design the various segments of the storm sewer itself, a new set of discharges must be developed. The proposed storm sewer layout is shown in Figure 8, Subdivision and Storm Sewer Layout, with the catch basins and manholes labeled from 1 through 10. Each storm sewer segment is designed assuming that the total tributary drainage area upstream of that segment is contributing flow. For example, the area tributary to segment 1-2

TABLE 15

VALUES OF C FOR INTERCONNECTED IMPERVIOUS AREA ONLY FOR EACH SUBAREA

Subarea	CN	RI yr	S %	I_5 in./hr	Imp	C
A	98	5	2.0	5.7	1.00	0.78
B	98	5	2.5	5.7	1.00	0.78
C	98	5	2.0	5.7	1.00	0.78
D	98	5	2.0	5.7	1.00	0.78
E	98	5	2.2	5.7	1.00	0.78
F	98	5	1.6	5.7	1.00	0.78
G	98	5	1.0	5.7	1.00	0.78
H	98	5	0.6	5.7	1.00	0.78

TABLE 16

VALUES OF C FOR TOTAL AREA OF EACH SUBAREA

Subarea	CN	RI yr	S %	I_5 in./hr	Imp	C
A	73	5	1.0	3.4	0.33	0.31
B	73	5	1.0	3.7	0.33	0.31
C	73	5	1.0	3.3	0.33	0.30
D	74	5	1.0	3.3	0.35	0.32
E	77	5	1.0	3.8	0.44	0.37
F	73	5	1.0	3.3	0.32	0.30
G	76	5	1.0	3.6	0.40	0.35
H	73	5	1.0	3.4	0.33	0.31

TABLE 17

PEAK INFLOW RATES FOR INTERCONNECTED IMPERVIOUS AREA ONLY FOR EACH SUBAREA

Subarea	A acres	i_5 in./hr	C	Q_5 cfs
A	0.32	5.7	0.78	1.4
B	0.17	5.7	0.78	0.8
C	0.35	5.7	0.78	1.6
D	0.35	5.7	0.78	1.6
E	0.14	5.7	0.78	0.6
F	0.40	5.7	0.78	1.8
G	0.55	5.7	0.78	2.4
H	0.26	5.7	0.78	1.2

TABLE 18

PEAK INFLOW RATES FOR TOTAL AREA OF EACH SUBAREA

Subarea	A acres	i_5 in./hr	C	Q_5 cfs
A	1.5	3.4	0.31	1.6
B	0.9	3.7	0.31	1.0
C	1.8	3.3	0.30	1.8
D	1.7	3.3	0.32	1.8
E	0.5	3.8	0.37	0.7
F	2.0	3.3	0.30	2.0
G	2.2	3.6	0.35	2.8
H	1.4	3.4	0.31	1.5

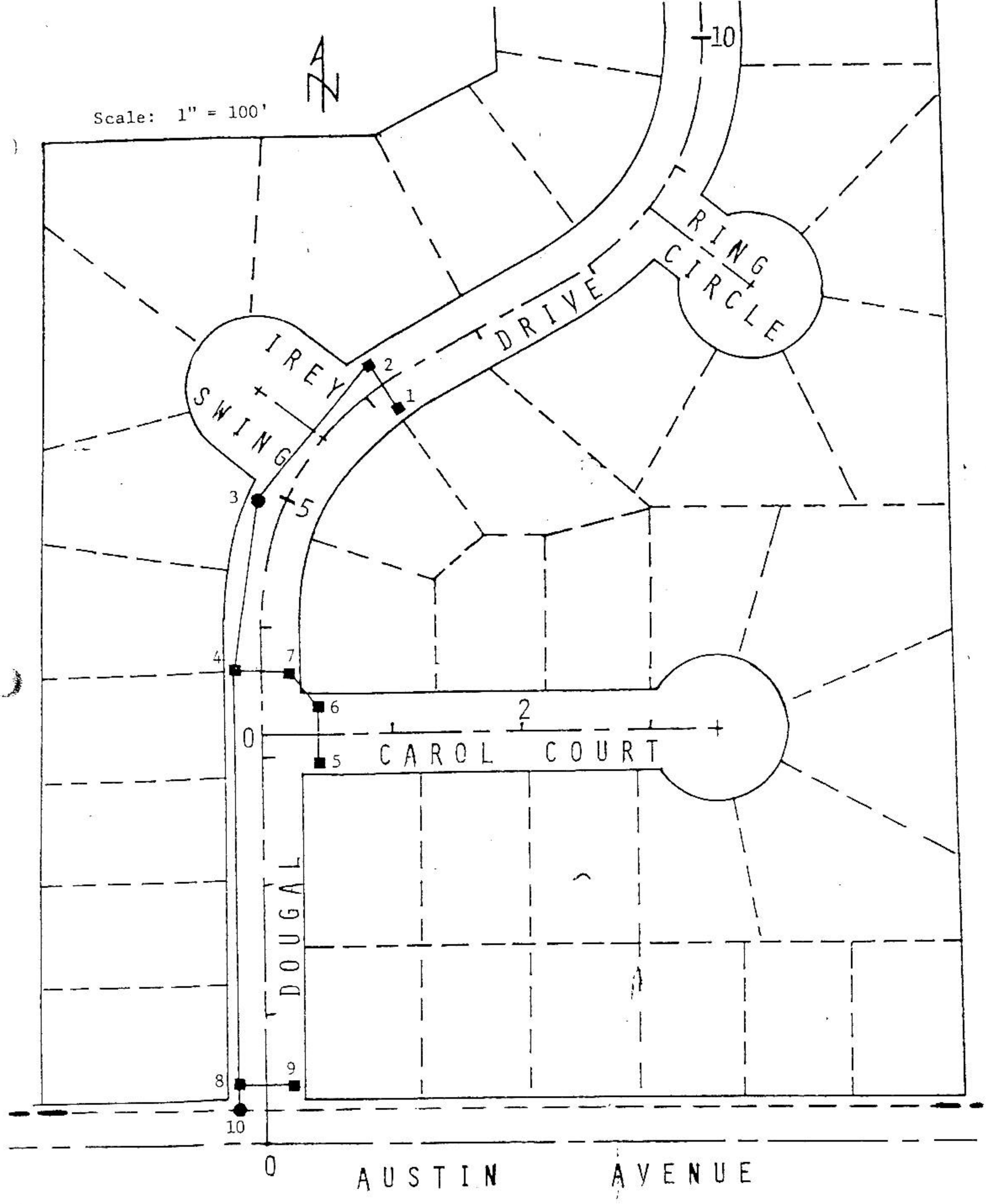

FIGURE 8 SUBDIVISION AND STORM SEWER LAYOUT

is subarea A; for segment 2-3, subareas A and B; and for segment 8-10, subareas A through H. In some cases the interconnected impervious area only may govern, so this possibility should also be checked.

The rainfall intensity averaging time for each segment is the total time to the upstream end of the segment: overland flow time plus travel time in either the gutter or the storm sewer, whichever is longer. For this example, travel time in the gutter is assumed to be longer and is taken from Table 14. T_c and i_T for each segment are developed in Table 19, Rainfall Intensity Averaging Times and Rainfall Intensities for Total Tributary Area to Each Storm Sewer Segment. A footnote to Table 19 indicates that certain pipe segments are connector pipes from catch basins to the main line (segments 1-2-3-4-8-10). T_cs for these connector pipes are based on overland and gutter flow times plus any flow time in upstream connector pipe(s).

Table 20, Values of C for Total Tributary Area to Each Storm Sewer Segment, lists the runoff coefficients for each segment. These C values are composite values for the total area upstream of that segment. Table 21, Peak Flow Rates for Total Tributary Area to Each Storm Sewer Segment, shows the peak flow calculations for each storm sewer segment. These discharges range from 1.6 to 11.2 cfs.

Some designers assume that the discharges shown in Table 21 are the ones to be used to design each segment of the storm sewer. This is true ONLY if all water arriving at the inlet(s) enters the inlet(s). The most usual case in actual practice is to allow some portion of the flow to bypass the inlet and be picked up at the next inlet. The cost of adding additional length of opening to insure that 100 percent of the flow is intercepted is usually excessive. Thus, each storm sewer segment is designed for only that water already in the upstream segment plus what enters at inlets and junctions. The remainder is picked up at some downstream inlet.

TABLE 19

RAINFALL INTENSITY AVERAGING TIMES AND RAINFALL INTENSITIES FOR TOTAL TRIBUTARY AREA TO EACH STORM SEWER SEGMENT

Segment	Subareas	Overland Flow Time min	Gutter Flow Time min	Gutter Flow Time min	t_c min	i_5 in./hr
1-2	A	18.4	0.8	0.9	20.0	3.4
2-3	A-B		0.5**		20.5	3.3
3-4	A-B				20.5	3.3
5-6*	C	19.3	0.5	1.6	21.4	3.2
6-7*	C-D		0.5**		21.9	3.1
7-4*	C-E		0.2**		22.1	3.1
4-8	A-F	20.5+0.8 or	22.1+0.2**		22.3	3.1
9-8*	G	16.0	2.4		18.4	3.5
8-10	A-H		3.0		25.3	3.0

* These segments are connector pipes to the main line and are designed based on t_c for those areas upstream of these segments.

** An assumed time of flow in the storm sewer segment immediately upstream.

TABLE 20

VALUES OF C FOR TOTAL TRIBUTARY AREA TO EACH STORM SEWER SEGMENT

Segment	Subareas	CN	RI yr	S %	i_5 in./hr	Imp	C
1-2	A	73	5	1.0	3.4	0.33	0.31
2-3	A-B	73	5	1.0	3.3	0.33	0.30
3-4	A-B	73	5	1.0	3.3	0.33	0.30
5-6	C	73	5	1.0	3.2	0.33	0.30
6-7	C-D	74	5	1.0	3.1	0.34	0.31
7-4	C-E	74	5	1.0	3.1	0.35	0.31
4-8	A-F	74	5	1.0	3.1	0.34	0.31
9-8	G	76	5	1.0	3.5	0.40	0.35
8-10	A-H	74	5	1.0	3.0	0.35	0.31

TABLE 21

PEAK FLOW RATES FOR TOTAL TRIBUTARY AREA TO EACH STORM SEWER SEGMENT

Segment	Subareas	A acres	i_5 in./hr	C	Q_5 cfs
1-2	A	1.5	3.4	0.31	1.6
2-3	A-B	2.4	3.3	0.30	2.4
3-4	A-B	2.4	3.3	0.30	2.4
5-6*	C	1.8	3.2	0.30	1.7
6-7*	C-D	3.5	3.1	0.31	3.4
7-4*	C-E	4.0	3.1	0.31	3.8
4-8	A-F	8.4	3.1	0.31	8.1
9-8*	G	2.2	3.5	0.35	2.7
8-10	A-H	12.0	3.0	0.31	11.2

* These segments are connector pipes to the main line and are designed based on t_c for those areas upstream of these segments.

The peak flow rates for the storm sewer were also determined using only the interconnected impervious areas. Table 22, Rainfall Intensity Averaging Times and Rainfall Intensities for Interconnected Impervious Area Only to Each Storm Sewer Segment, lists t_c and i_T for this case. T_cs now range from 5.0 to 9.1 min as opposed to 20.0 to 25.3 min when the entire tributary drainage area is used. Table 23, Values of C for Interconnected Impervious Area Only to Each Storm Sewer Segment, shows the variables used to determine the runoff coefficient C for each storm sewer segment. C is now 0.78 rather than ranging from 0.30 to 0.35 when the entire tributary drainage area is considered. Table 24, Peak Flow Rates for Interconnected Impervious Area Only for Each Storm Sewer Segment, lists the peak flow rates from the interconnected impervious area only. Coincidentally, they are about the same as those shown in Table 21, the peak flow rates for the entire tributary drainage area.

TABLE 22

RAINFALL INTENSITY AVERAGING TIMES AND RAINFALL INTENSITIES FOR INTERCONNECTED IMPERVIOUS AREA ONLY TO EACH STORM SEWER SEGMENT

Segment	Subareas	Overland Flow Time min	Gutter Flow Time min	Gutter Flow Time min	t_c min	i_5 in./hr
1-2	A	2.3	0.8	0.9	5.0	5.7
2-3	A-B		0.5**		5.5	5.5
3-4	A-B				5.5	5.5
5-6*	C	2.3	0.6	1.6	5.0	5.7
6-7*	C-D		0.5**		5.5	5.5
7-4*	C-E		0.2**		5.7	5.4
4-8	A-F	5.5+0.8 or	5.7+0.2**		6.3	5.3
9-8*	G	2.3	2.8		5.1	5.7
8-10	A-H		2.8		9.1	4.8

* These segments are connector pipes to the main line and are designed based on t_c for those areas upstream of these segments

** An assumed time of flow in the storm sewer segment immediately upstream.

TABLE 23

VALUES OF C FOR INTERCONNECTED IMPERVIOUS AREA ONLY TO EACH STORM SEWER SEGMENT

Segment	Subareas	CN	RI yr	S %	i_5 in./hr	Imp	C
1-2	A	98	5	2.0	5.7	1.00	0.78
2-3	A-B	98	5	2.2	5.5	1.00	0.78
3-4	A-B	98	5	2.2	5.5	1.00	0.78
5-6	C	98	5	2.0	5.7	1.00	0.78
6-7	C-D	98	5	2.0	5.5	1.00	0.78
7-4	C-E	98	5	2.0	5.4	1.00	0.78
4-8	A-F	98	5	2.0	5.3	1.00	0.78
9-8	G	98	5	1.0	5.7	1.00	0.78
8-10	A-H	98	5	1.6	4.8	1.00	0.78

TABLE 24

PEAK FLOW RATES FOR INTERCONNECTED IMPERVIOUS AREA ONLY TO EACH STORM SEWER SEGMENT

Segment	Subareas	A acres	i_5 in./hr	C	Q_5 cfs
1-2	A	0.32	5.7	0.78	1.4
2-3	A-B	0.49	5.5	0.78	2.1
3-4	A-B	0.49	5.5	0.78	2.1
5-6*	C	0.35	5.7	0.78	1.6
6-7*	C-D	0.70	5.5	0.78	3.0
7-4*	C-E	0.84	5.4	0.78	3.5
4-8	A-F	1.73	5.3	0.78	7.2
9-8*	G	0.55	5.7	0.78	2.4
8-10	A-H	2.54	4.8	0.78	9.5

* These segments are connector pipes to the main line and are designed based on t_c for those areas upstream of these segments.

As shown above, the rational formula is used four times to estimate the peak flow rates for an urban storm sewer system: twice for each inlet and twice for each segment of the storm sewer; once in each case for the total tributary drainage area and once for the interconnected impervious area only. Then, for each segment of the storm sewer, this estimate could be modified because only a portion of the flow might be intercepted at an inlet with the remainder flowing down the gutter to be picked up at a downstream inlet.

Use of the Rational Formula as a Hydrograph for Sizing Detention Basins

Assume that the existing storm sewer in Austin Avenue, shown in Figure 8, is already at capacity and that the local ordinance requires the peak release rate for a 100-year storm to be no greater than the peak rate from the watershed in its undeveloped condition. For the new subdivision, this could be accomplished by converting some lots at the corner of Austin Avenue and Dougal Drive into a mini-park and shaping it so that it will contain the necessary volume of temporary storage. The storm sewer system and the streets near the park would also have to be modified so that flow is diverted into the park.

There are two methods that have been devised to use the rational formula for hydrograph development. These will be illustrated through the use of examples and will be compared with the short-cut method of sizing detention basins developed by the Soil Conservation Service in TR No. 55.

Rational Formula - Method 1

Method 1 is faithful to the underlying assumptions of the rational formula: the peak discharge rate is reached at a time equal to the time of concentration; the rainfall intensity is constant throughout the duration of the storm. The method proceeds as follows for some assumed recurrence interval. A peak discharge rate is estimated for a storm duration equal to the time of concentration. A triangular hydrograph is plotted with a peak equal to the estimated discharge and a time base equal to two times t_c. The area under the hydrograph is calculated. The area of a rectangle whose height is equal to the required release rate and whose base is equal to the storm duration is also calculated. The difference between these two areas is the required storage volume for that storm duration.

The same procedure is followed for storms of longer and longer duration. These hydrographs have the shape of a trapezoid with heights equal to the estimated discharge rates and bases equal to the storm duration plus t_c. The areas of these trapezoids as well as the release areas are calculated with the difference in area being the required storage vloume for that duration. The final required storage is the maximum value of the several calculated storages. For the subdivision shown in Figure 8, the allowable release rate (undeveloped condition) is calculated as follows.

1. A = 12.0 acres
2. t_c = 36 min   from equation (33) for N = .4, L = 800 ft, S = 1%
3. i_{100} = 4.1 in./hr   from an I-D-F curve
4. C = 0.21   from equation (6) for CN = 61, RI = 100, S = 1%, I = 4.1 in./hr, Imp = 0.00
5. Q = CiA = 0.21 x 4.1 x 12.0 = 10.3 cfs

The storage calculations are shown in Table 25, Calculations for Required Storage Volume Using the Rational Formula - Method 1, and plotted in Figure 10, Inflow and Outflow Hydrographs for the Rational Formula - Method 1. A sample set of calculations for a storm duration of 1.0 hr is shown below.

Col. (1) duration = 1.0 hr

Col. (2) i_{100} = 3.2 in./hr   from an I-D-F curve

Col. (3) C = 0.37   for CN = 74, RI = 100, S = 1%, I = 3.2 in./hr, Imp = 0.35

Col. (4) Q = CiA = 0.37 x 3.2 x 12.0 = 14.2 cfs

Col. (5) 10.3 cfs

Col. (6) Col. (4) - Col. (5) = 14.2 - 10.3 = 3.9 cfs

Col. (7) Required Storage = Col. (6) x Col. (1) x 3630   convert cfs-hr to acre-inch (?)

= 3.9 x 1.0 x 3630

= 14,200 cu ft

TABLE 25

CALCULATIONS FOR REQUIRED STORAGE VOLUME
USING THE RATIONAL FORMULA - METHOD 1

(1) Duration hr	(2) i_{100} in./hr	(3) C	(4) Inflow* cfs	(5) Outflow cfs	(6) Stored cfs	(7) Storage** cu ft
0.33	5.4	0.41	20.9	10.3	10.6	12.800
0.42	5.0	0.40	24.0	10.3	13.7	20,900
0.75	3.7	0.38	16.9	10.3	6.6	18,000
1.00	3.2	0.37	14.2	10.3	3.9	14.200
1.25	2.9	0.36	12.5	10.3	2.2	10,000

* A = 12.0 acres

** 1 cfs-hr = 1 ac-in. = 3,630 cu ft

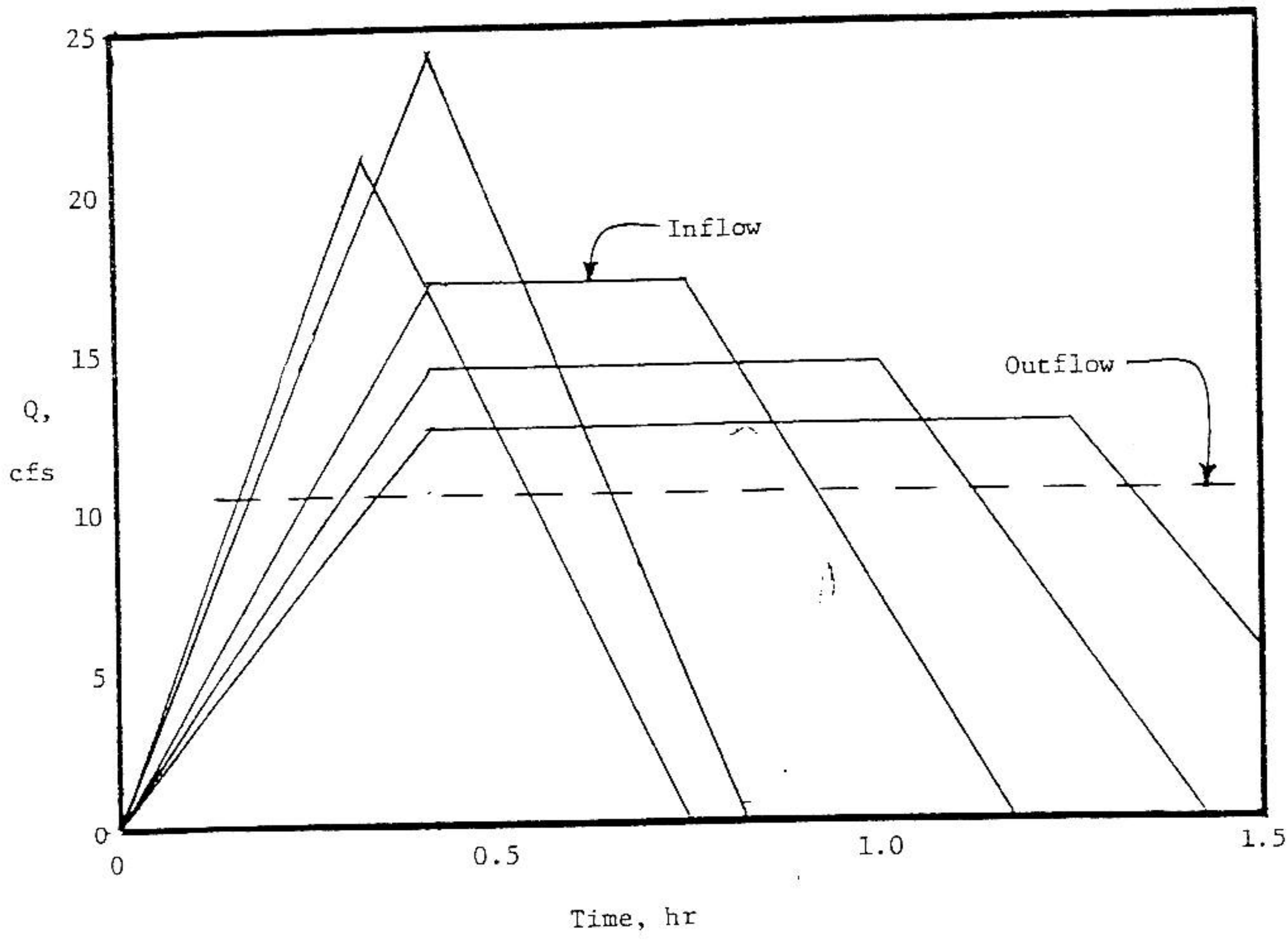

FIGURE 10 INFLOW AND OUTFLOW HYDROGRAPHS FOR THE RATIONAL FORMULA - METHOD 1

The required volume is the largest value in column (7) of Table 25, about 21,000 cu ft. One problem with Method 1 is that the developed hydrographs do not look like the hydrographs recorded at a gaging station. However, the volume of runoff is correct.

Rational Formula - Method 2

Method 2 takes a somewhat different approach and is difficult to justify. About its only redeeming feature is that the developed hydrograph looks like a hydrograph. The method proceeds as follows for some assumed recurrence interval. A peak discharge is estimated for a storm duration equal to the time of concentration. This peak rate is plotted as one point on a hydrograph at a time equal to t_c. The same procedure is followed for storms of shorter and longer durations. All these points are plotted and a smooth curve is drawn through the points as shown in Figure 11, Inflow and Outflow Hydrographs for the Rational Formula - Method 2.

The area under the hydrograph is calculated. The rectangular area of outflow is also calculated. The difference between these two areas is the required storage volume. The procedure shown in Table 26, Calculations for Required Storage Volume Using the Rational Formula - Method 2, calculates the difference in area in a somewhat different manner. The peak inflow rate for each "storm duration" is estimated as shown in Table 25. The total storage required is the summation of the storages in the last column, about 26,300 cu ft.

This second method is difficult to justify because the developed hydrograph is not the hydrograph of runoff from a single storm. Rather, it is something that looks like a hydrograph which was developed from a series of storms of ever-increasing duration. Or it might be thought of as the runoff hydrograph from a single storm whose rainfall distribution (and accompanying intensities) throughout the storm was such that the hydrograph shown in Figure

11 was produced. However, this explanation violates a basic assumption of the rational formula, that of a uniform rainfall intensity throughout the entire duration of a storm. For what it is worth, the required storage volumes estimated in Methods 1 and 2 are comparable, 21,000 cu ft vs. 26,000 cu ft.

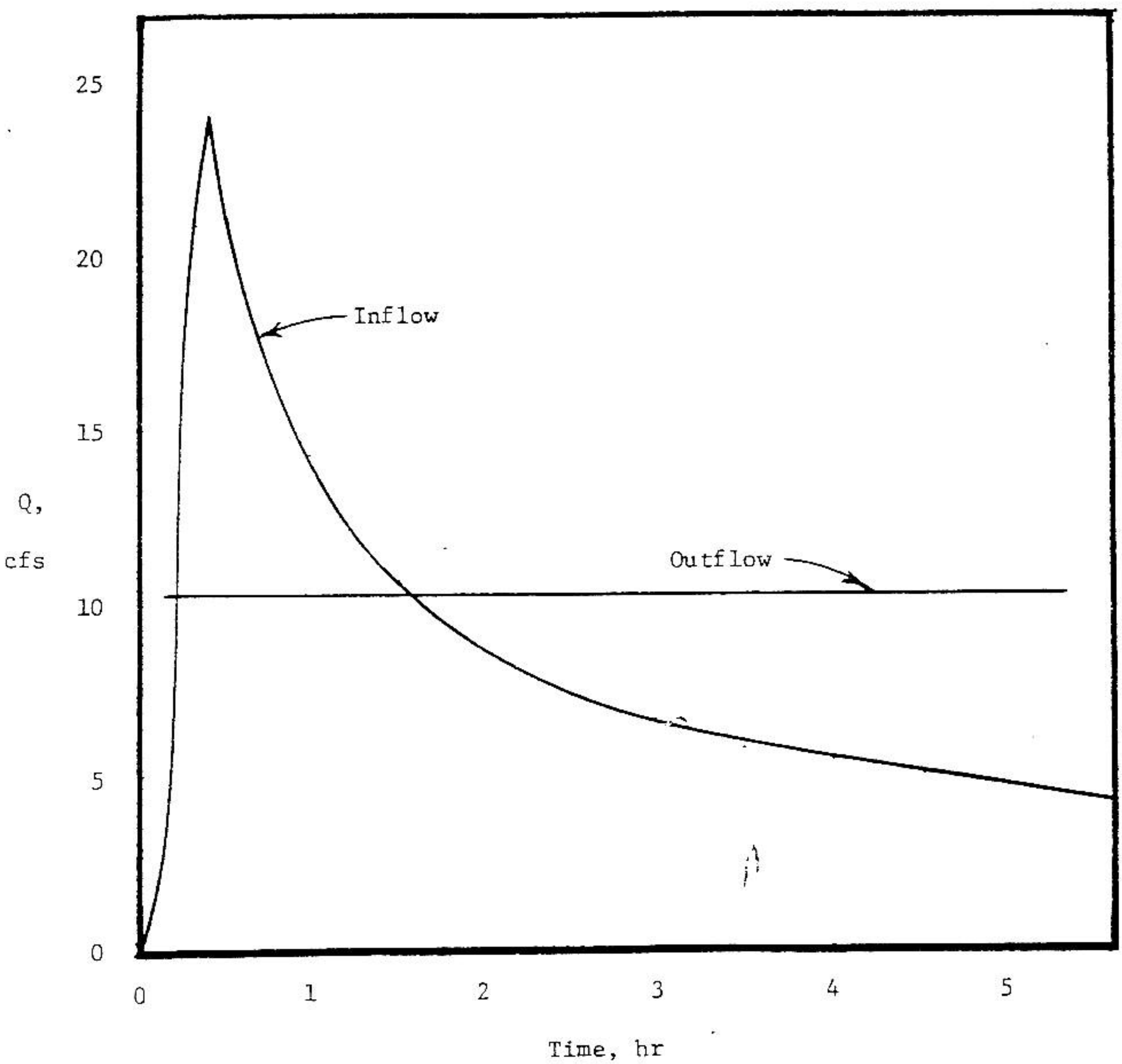

FIGURE 11 INFLOW AND OUTFLOW HYDROGRAPHS
FOR THE RATIONAL FORMULA - METHOD 2

TABLE 26

CALCULATIONS FOR REQUIRED STORAGE VOLUME
USING THE RATIONAL FORMULA - METHOD 2

Storm Duration hr	Peak Inflow cfs	Average Inflow cfs	Average Outflow cfs	Stored Inflow cfs	Delta Time hr	Delta Storage* cu ft
0.00	0.0					
		4.80	10.3	0.00	0.20	0
0.20	9.6					
		15.25	10.3	4.95	0.13	2,170
0.33	20.9					
		22.45	10.3	12.15	0.09	3,970
0.42	24.0					
		22.60	10.3	12.30	0.08	3,570
0.50	21.2					
		19.05	10.3	8.75	0.25	7,940
0.75	16.9					
		15.55	10.3	5.25	0.25	4,760
1.00	14.2					
		13.35	10.3	3.05	0.25	2,770
1.25	12.5					
		11.50	10.3	1.20	0.25	1,090
1.50	10.5					
		10.10	10.3	0.00	0.25	0
1.75	9.7					
		9.15	10.3	0.00	0.25	0
2.00	8.6					
		7.95	10.3	0.00	0.50	0
2.50	7.3					
		6.90	10.3	0.00	0.50	0
3.00	6.5					
		6.15	10.3	0.00	0.50	0
3.50	5.8					
		5.60	10.3	0.00	0.50	0
4.00	5.4					
				Required Total Storage	=	26,270

* 1 cfs-hr = 1 acre-in. = 3,630 cu ft

Delta Storage = Stored Inflow x Delta Time x 3,630

SCS Technical Release No. 55[18]

In 1975 the US Soil Conservation Service published Technical Release No. 55, Urban Hydrology for Small Watersheds. This manual contains several methods for estimating the increase in peak flow rate and runoff volume due to urbanization, as well as a quick method for estimating the effect of storage on peak discharges. This quick method is discussed in Chapter 7 of TR 55 and involves the use of one of two figures, reproduced here as Figure 12, Approximate Single-Stage Structure Routing for Weir Flow Structures up to 150 csm Release Rate and Pipe Flow Structures up to 300 csm Release Rate, and Figure 13, Approximate Single-Stage Structure Routing for Weir Flow Structures Over 150 csm Release Rate and Pipe Flow Structures Over 300 csm Release Rate.

These figures are based on the average storage and routing effects on a Type II, 24-hr storm for many structures. The accuracy of these two figures is discussed in the following excerpt from TR 55.

> The accuracy of the curves in figures 7-1 and 7-2 depends on the relationship between the storage available, the inflow volume, and the shape of the inflow hydrograph. In figure 7-1 (Figure 12) the peak inflow rate is not a factor in determining storage requirements. It can be seen that the ratio of volume of storage (V_s) to volume of runoff (V_r) is relatively high. Therefore, inflow peak is not a significant factor. Figure 7-1 is usually accurate within 5 percent for release rates under 100 csm (cubic feet per second per square mile) and within 10 percent for release rates over 100 csm.
>
> Figure 7-2 (Figure 13) relates the ratio of peaks to volumes. For this case the parameters affecting the shape of the hydrograph are important. In situations where runoff curve numbers are less than 65 in combination with short t_c values, V_s/V_r values read from the curve will be up to 25 percent too high. Runoff curve numbers over 85 with long t_c values cause V_s/V_r values to be up to 25 percent too low.

This quick method for determining the temporary storage volume needed is composed of the following five steps.

1. Determine the basic watershed parameters (DA, CN, T_c, etc.).
2. Determine the volume of runoff and peak rate of flow from the watershed.
3. Set the desired rate of outflow from the structure.
4. Determine the required volume of storage from the appropriate figure, Figure 12 or Figure 13.
5. Proportion the storage structure so that the design outflow rate and required storage occur at the same stage.

The peak discharge rate from the developed watershed can be estimated by two methods contained in TR 55. The computation sheets used here were developed by an SCS engineer in Des Moines, Iowa. The first method is shown in Figure 14, The TR 55 Appendix D Method of Estimating Peak Discharge - Developed Condition. The second method is illustrated in Figure 15, The TR 55 Lag-T_c Method of Estimating Peak Discharge - Developed Condition. The average peak inflow rate to the detention area is 63 cfs.

Step 3 is to set the desired rate of outflow. As stated above, the local ordinance requires that this rate be no greater than the peak rate from the watershed in its undeveloped condition. Based on the calculations shown in Figure 16, The TR 55 Appendix D Method of Estimating Peak Discharge - Undeveloped Condition, and Figure 17, The TR 55 Lag-T_c Method of Estimating Peak Discharge - Undeveloped Condition, this rate is 17 cfs.

Step 4 is to determine the required volume of temporary storage. Based on the calculations shown in Figure 18, TR 55 Quick Method of Estimating Volume of Temporary Storage Required, the volume needed is 1.6 acre-ft or 70,000 cu ft.

This compares with volumes of 21,000 and 26,000 cu ft determined using Methods 1 and 2 of the rational formula. There are two reasons for this large difference in required storage. First, the storm durations are much different

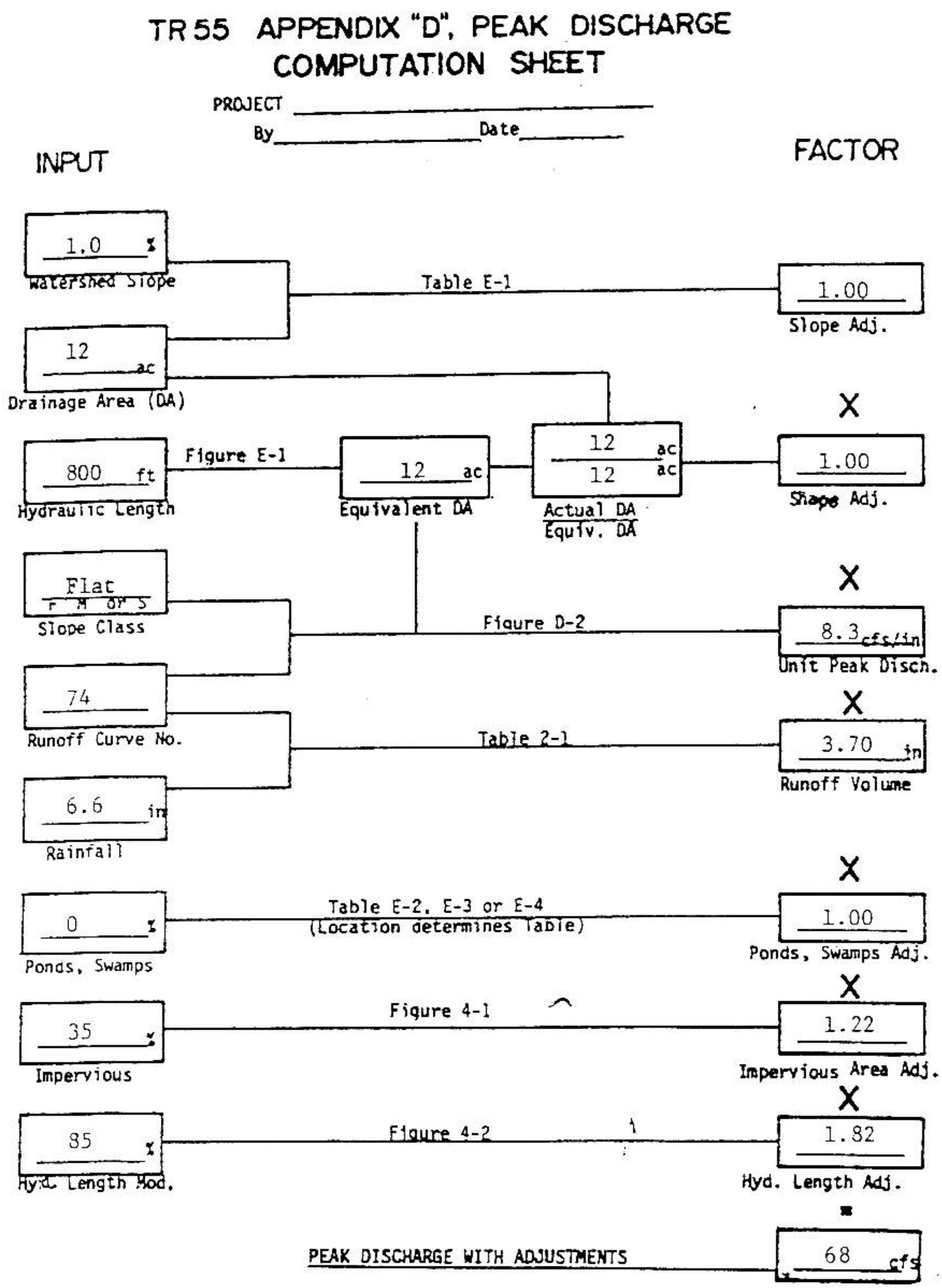

FIGURE 14 THE TR 55 APPENDIX D METHOD OF ESTIMATING PEAK DISCHARGE - DEVELOPED CONDITION

TR-55 LAG-T_C METHOD PEAK DISCHARGE COMPUTATION SHEET

PROJECT ______________________

By ______________________ Date ____________

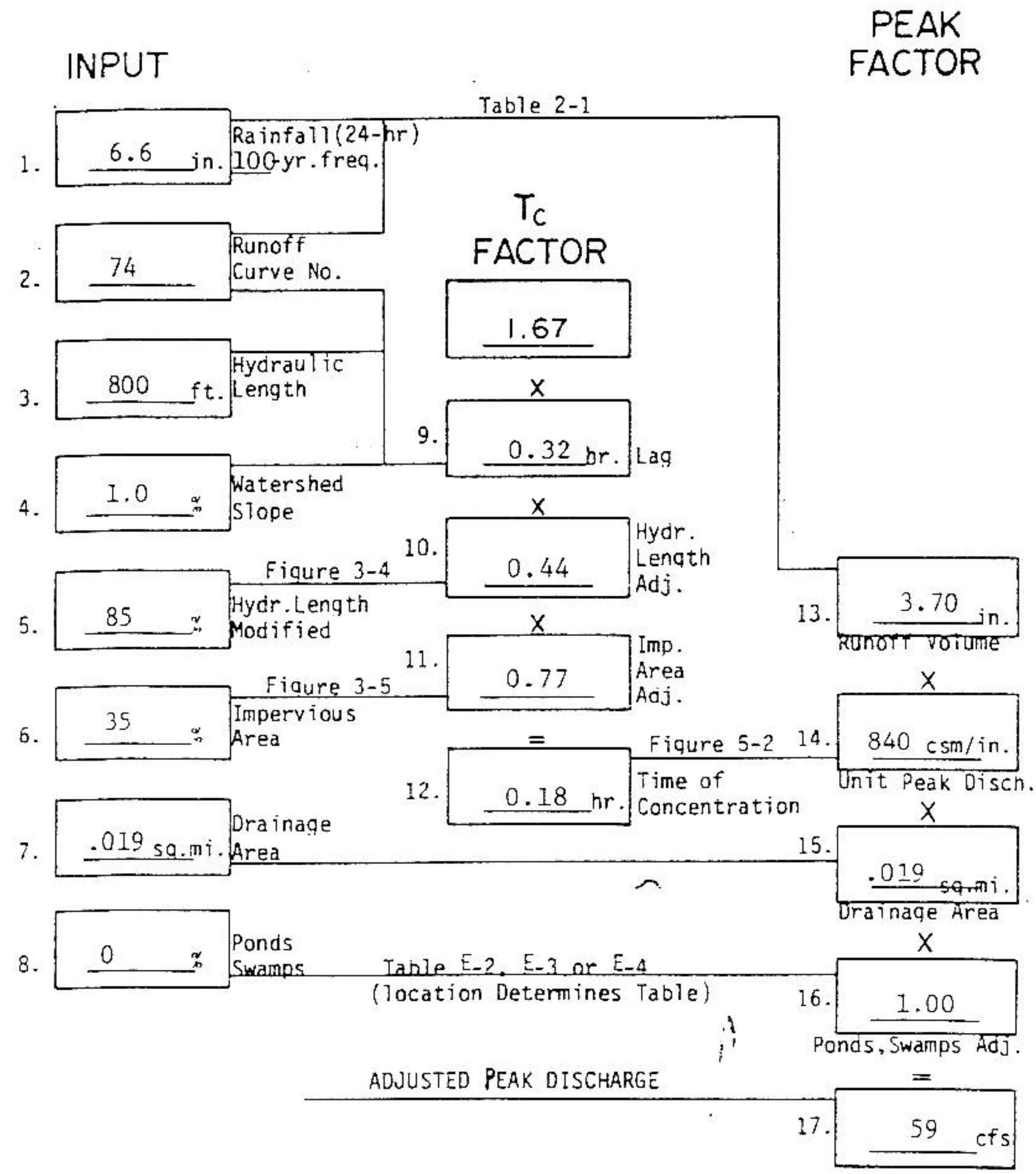

FIGURE 15 THE TR 55 LAG-T_c METHOD OF ESTIMATING PEAK DISCHARGE - DEVELOPED CONDITION

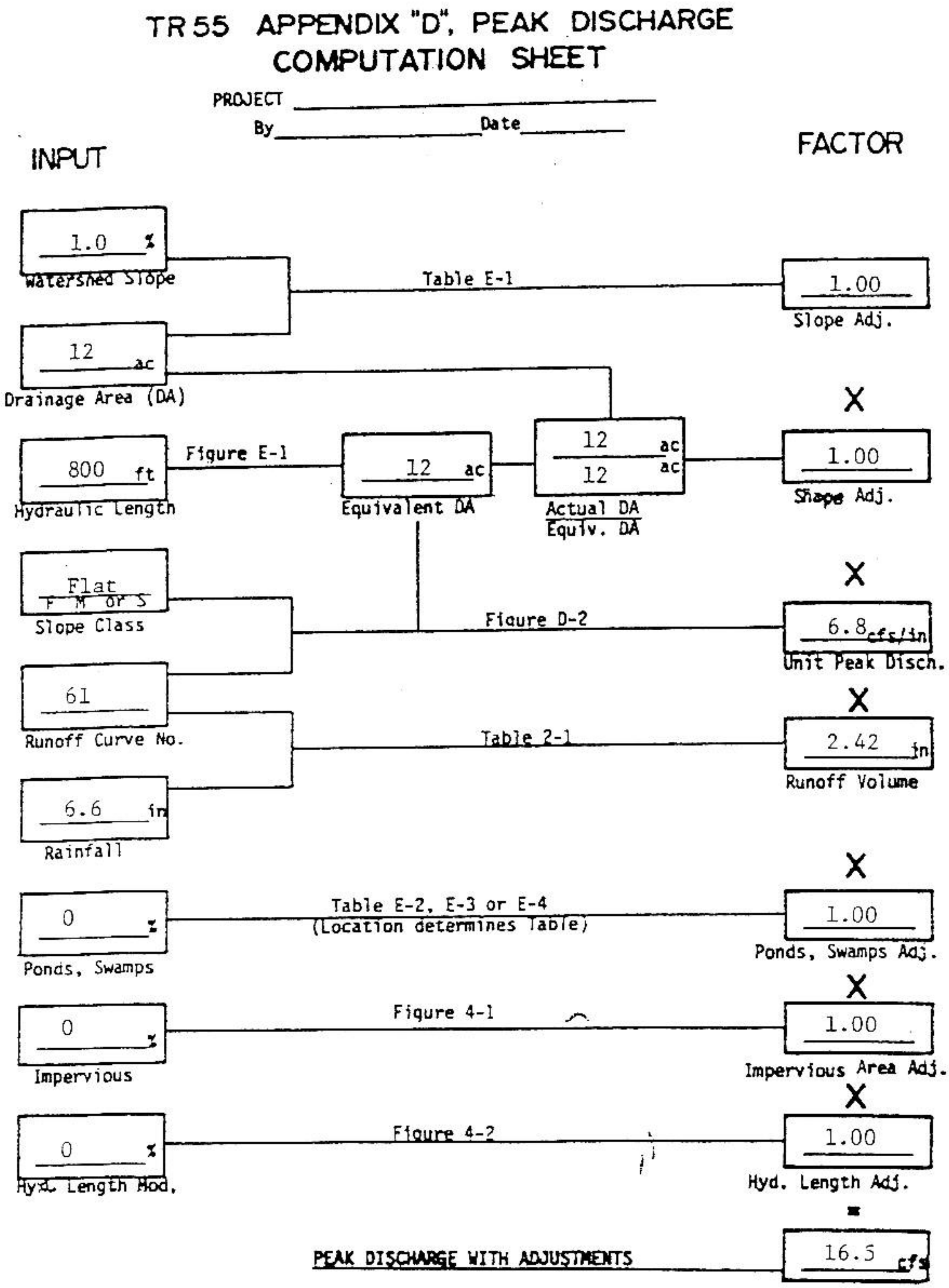

FIGURE 16 THE TR 55 APPENDIX D METHOD OF ESTIMATING PEAK DISCHARGE - UNDEVELOPED CONDITION

TR-55 LAG-T_c METHOD PEAK DISCHARGE COMPUTATION SHEET

PROJECT ____________________

By ____________________ Date __________

	INPUT			T_c FACTOR			PEAK FACTOR	
1.	6.6 in.	Rainfall (24-hr) 100 yr. freq.		1.67		13.	2.42 in.	Runoff Volume
2.	61	Runoff Curve No.		X			X	
3.	800 ft.	Hydraulic Length	9.	0.45 hr.	Lag	14.	390 csm/in.	Unit Peak Disch.
4.	1.0 %	Watershed Slope		X			X	
5.	0 %	Hydr. Length Modified	10.	1.00	Hydr. Length Adj.	15.	.019 sq.mi.	Drainage Area
6.	0 %	Impervious Area	11.	1.00	Imp. Area Adj.		X	
7.	.019 sq.mi.	Drainage Area	12.	= 0.75 hr.	Time of Concentration	16.	1.00	Ponds, Swamps Adj.
8.	0 %	Ponds Swamps				17.	= 17.7 cfs	

Table 2-1 (items 1–2 → 13)

Figure 3-4 (item 5 → 10)

Figure 3-5 (item 6 → 11)

Figure 5-2 (item 12 → 14)

Table E-2, E-3 or E-4 (location Determines Table) (item 8 → 16)

ADJUSTED PEAK DISCHARGE (17)

FIGURE 17 THE TR 55 LAG-T_c METHOD OF ESTIMATING PEAK DISCHARGE - UNDEVELOPED CONDITION

TR-55 STORM WATER STORAGE COMPUTATION SHEET

PROJECT ______________ By ______________ Date ______________

Given:
Prin. Spill.; Weir ___ or Pipe X
Reservoir Stage-Stg Curve(attached)
Struc. Release Rate (q_o) 17 cfs 900 csm
Peak Inflow Rate (q_i) 63 cfs

Dr.Area (DA) 12 Ac. .019 Sq.Mi.
Design Frequency 100 Yr.
Runoff Depth (Vr=Q) 3.70 In.
Sediment Storage ______ In.

2. Routing Curve: Use Release Rate (q_o) to select curve. Check one blank.

Figure 7-1	Type of Principal Spillway	Figure 7-2
___ Up to 150 csm	Weir Flow Structure	Over 150 csm ___
___ Up to 300 csm	Pipe Flow Structure	Over 300 csm X

3. Floodwater Storage Required (Vs), Inches Depth on watershed.

Figure 7-1. Use Runoff Depth (Vr) and Structure Release Rate (q_o) as input to find Storage (Vs) ______ in.

or

Figure 7-2. Use Structure Release Rate (q_o) and Peak Inflow Rate (q_i)

as the ratio, $\frac{q_o}{q_i} = \frac{17 \text{ cfs}}{63 \text{ cfs}} = 0.27$

to find the volume ratio, $\frac{\text{Volume of Storage (Vs)}}{\text{Volume of Runoff (Vr)}} = 0.435$

Vs = (0.435) (Vr) = (0.435) (3.70) = 1.60 in.

4. Floodwater Storage Required (Vs), acre-feet.

$Vs = \left(\frac{1.60 \text{ in.}}{12 \text{ in/ft}}\right)\left(\frac{12.0 \text{ ac}}{\text{DA}}\right) = 1.60$ acre-feet = 69,700 cu ft

5. Proportion the structure so that the desired release rate and the required storage occur at the same water surface elevation in the reservoir. Consider Sediment Storage requirement and prepare summary table below. The optional design should include provision for additional water use needs as can be met by the site.

STRUCTURE SUMMARY DATA

Reservoir Design	Water Surface Elevation ft.	Volume (in.)	Volume (ac.-ft.)	Surface Area (acres)
Minimum				
Permanent Pool	______	______	______	______
Flood Pool	______	______	______	______
Optional				
Permanent Pool	______	______	______	______
Flood Pool	______	______	______	______

6. Proportion an emergency spillway to safely convey flows during storms greater than the design frequency.

FIGURE 18 TR 55 QUICK METHOD OF ESTIMATING VOLUME OF TEMPORARY STORAGE REQUIRED with a corresponding difference in rainfall and runoff volumes. The durations range from less than an hour to 24 hours with runoff volumes ranging from 37,000 cu ft to 161,000 cu ft.

Second, the rational formula methods assume uniform rainfall intensities throughout the entire storm duration while the SCS method does not. The variation in rainfall intensity throughout the SCS 24-hr storm is shown in Table 27, Rainfall Intensities in a 100-yr, 24-hr SCS Type II Rainfall Distribution. The SCS Type II storm has average intensities which range from 0.07 in./hr to 7.32 in./hr. The average intensity for the 24-hr storm would be 6.6/24 = 0.28 in./hr. This points up the main difference and weakness of the rational formula method. With a uniform rainfall intensity for the entire storm duration, the i_T in the rational formula is smaller, resulting in a smaller value of Q_T. This smaller Q_T (an average Q_T?) may be less than the allowable outflow rate, thus yielding an answer which states that no temporary storage is required.

The assumption of average rainfall intensity throughout the entire storm duration in the rational formula is valid only for short duration storms, which translates into short rainfall intensity averaging times, which in turn translates into small drainage areas. How small is small? Is it 20 acres or 200 acres? We still have not decided this question. A better question might be: how short is a short duration storm or rainfall intensity averaging time? This time could then be converted into a range of watershed sizes based on land use, soil type, slope and imperviousness.

TABLE 27

RAINFALL INTENSITIES IN A 100-YR, 24-HR
SCS TYPE II RAINFALL DISTRIBUTION
(based on $P_{100,24} = 6.6$ in.)

Time hr	P_x/P_{24} *	P in.	Delta P in.	Delta T hr	I in./hr
0.0	0.000	0.00			
			0.14	2.0	0.07
2.0	0.022	0.14			
			0.18	2.0	0.09
4.0	0.048	0.32			
			0.21	2.0	0.10
6.0	0.080	0.53			
			0.26	2.0	0.13
8.0	0.120	0.79			
			0.18	1.0	0.18
9.0	0.147	0.97			
			0.11	0.5	0.22
9.5	0.163	1.08			
			0.11	0.5	0.22
10.0	0.181	1.19			
			0.16	0.5	0.32
10.5	0.204	1.35			
			0.20	0.5	0.40
11.0	0.235	1.55			
			0.32	0.5	0.64
11.5	0.283	1.87			
			0.68	0.25	2.72
11.75	0.387	2.55			
			1.83	0.25	7.32
12.0	0.663	4.38			
			0.47	0.5	0.94
12.5	0.735	4.85			
			0.25	0.5	0.50
13.0	0.772	5.10			
			0.17	0.5	0.34
13.5	0.799	5.27			
			0.14	0.5	0.28
14.0	0.820	5.41			
			0.40	2.0	0.20
16.0	0.880	5.81			
			0.47	4.0	0.12
20.0	0.952	6.28			
			0.32	4.0	0.08
24.0	1.000	6.60			

* Ratio of accumulated rainfall to total rainfall

Conclusion

Expenditures by local governments and developers for storm sewer facilities are expected to average about $3.5 billion annually for the next several years. New methods are emerging which provide improved design capabilities, but until these emerging methods come into more general use, the rational formula will continue to be used to design many of these new storm sewers. Therefore, those who use the rational formula should be fully aware of its limitations and inherent assumptions and of the many misconceptions which have developed over the past ninty years.

Four principal variables are used in the rational formula: a runoff coefficient, rainfall intensity averaging time, rainfall intensity and drainage area. Rather than being independent variables, there is some interdependency among them. Many misconceptions and divergent methodologies concerning these variables have arisen during this century. Hopefully, the preceeding discussion has dispelled some of these misconceptions and the recommendations made will lead to more uniform results when the rational formula is used in the future.

The rational formula is used four times in the design of a storm sewer system: twice for the design of the inlets and twice for the design of the storm sewer itself. However, this storm sewer system is just one portion of the minor drainage system in each community. This minor system is usually designed for the 2- or 5-year storm. The minor system is just that, a system which reduces or eliminates minor inconvenience. The major system is utilized whenever the capacity of the minor system is exceeded. Unfortunately, in many communities, because the major system was never designed or even recognized initially, these same 2- or 5-year storms often-times produce major inconvenience and damage.

The rational formula has also been used as a means for developing hydrographs. These "hydrographs" are then used to determine the volume of temporary storage needed for some given outflow rate. Two methods are presented for developing "hydrographs" using the rational formula. The results obtained from using these "hydrographs" are contrasted with those obtained from a quick method developed by the US Soil Conservation Service. A weakness of the SCS method is that it is only applicable for 24-hr duration storms. The weaknesses of the two methods using the rational formula are those inherent in the formula itself.

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