ABSTRACT

Title of Document: GUIDELINES FOR HYDROLOGIC DESIGN FOR NONHOMOGENEOUS MICROWATERSHEDS USING THE RATIONAL METHOD

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The Rational method is frequently used to calculate the peak discharge from a watershed for hydraulic design. However, the Rational method is based on several assumptions that can lead to underdesign and unexpected flood risk. The goal of this research was to improve the understanding of the effects of nonuniformity of land cover and watershed slope on hydrologic design for microwatersheds. The problematic assumptions of the Rational method were challenged through the development of trial microwatersheds with nonhomogeneous runoff coefficients and slopes. The results showed that under certain hydrologic conditions, the traditionally computed peak discharge rates could underestimate the actual maximum discharge that results from a subarea of the watershed. This peak was defined as a premature peak. General guidelines were developed to identify the hydrologic conditions for which a premature peak would be expected to occur. Following these guidelines should reduce flood risk in developing watersheds.

GUIDELINES FOR HYDROLOGIC DESIGN FOR NONHOMOGENEOUS MICROWATERSHEDS USING THE RATIONAL METHOD

By

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CHAPTER 1

INTRODUCTION

1.1 PROBLEM STATEMENT

Traditionally, the computation of a peak discharge rate assumes that the entire drainage area is contributing runoff. Related to this practice, the time of concentration is defined as the time required for direct runoff to travel from the most hydraulically distant part of the watershed to the outlet. However, under certain watershed conditions, it is possible that the maximum discharge could occur without the entire drainage area contributing. Such a discharge would be greater than that under the traditional peak discharge assumption. In such cases, infrastructure based on the traditional assumption could be underdesigned. The identification of these hydrologic conditions is necessary to obtain peak discharge estimates that will produce safe designs.

A runoff peak discharge is often used to design hydraulic infrastructure within a watershed. The estimation of a computed discharge needs to be accurate to best quantify the volume and rate of runoff that a water management facility must handle. Additionally, the peak discharge is utilized in environmental issues, such as predicting soil erosion and the transport of surface pollutants. Inaccurate discharge estimates can lead to either underdesigned infrastructure that increases flood risk or overdesigned facilities that require unnecessary expenditures.

Many different approaches are available for determining the peak runoff rate from a watershed, with the corresponding equations varying in complexity and constraints. Two general hydrologic model types are available: calibrated and uncalibrated. Calibrated models are based on the analysis of long-term stream gage data, which can be expensive and is only available at a few locations. Uncalibrated models are more commonly used because they are easy to apply and require input data that are readily available. They are generally more flexible to show effects, such as the effect of urbanization.

The Rational method is a frequently used, uncalibrated model that computes the peak discharge rate using a measure of the land cover/use, the intensity of rainfall for a specified length of time, and the watershed drainage area. The Rational method is a frequently used design tool because of its relative simplicity and easily obtainable inputs. The method makes several simplifying assumptions. The assumptions relate to the input variables. The assumptions most relevant to this research are:

(1) The runoff coefficient does not vary within the drainage area.

(2) The watershed is relatively homogeneous in terms of factors that affect timing of runoff, such as slope.

(3) The maximum discharge occurs when runoff is contributed from the entire drainage area.

(4) The total area of the watershed, but not its shape, is the principal characteristic used to define the volume of runoff.

Analyses of these assumptions could be used to identify conditions under which the Rational method as traditionally applied leads to underdesign.

1.1.1 Assumptions 1 and 2: Homogeneity Across a Watershed

Assumptions 1 and 2 define a watershed as being a relatively homogeneous area in terms of land cover, soil characteristics, and slope. However, these conditions are likely to vary, minutely or drastically, within a watershed. Traditionally, the Rational method is applied to nonhomogeneous watersheds by using a runoff coefficient that is an area-weighted average of runoff coefficients for the individual parcels of land. Hydraulic infrastructure that is designed when assuming watershed homogeneity could be inaccurate if the watershed is heterogeneous in land cover.

Challenging these two assumptions becomes particularly important when considering the increasing urbanization that is occurring across the globe. Urban development is often piecemeal, especially at the small watershed scale. One of the primary characteristics of urbanization is the increase in impervious areas such as roadways, rooftops, and parking lots; these are not dispersed spatially in a uniform pattern. Increases in impervious cover occur even at the microwatershed level, which renders the watershed heterogeneous even at a small scale. The increased imperviousness results in surface runoff that is greater in magnitude than that from pervious land cover; it also travels more quickly to the outlet. This adds considerable complexity to a description of the runoff processes. Urbanization can also change the flow path of runoff through the implementation of stormwater conveyance systems. These systems tend to provide a more direct route to the watershed outlet, whereas a natural flow path would have greater meandering. The increased urbanization contributes to the idea that the accurate designs require land cover to be treated as heterogeneous, as it is likely that some portion of the watershed will have been influenced by human development.

1.1.2 Assumption 3: Effect of Contributing Area on Peak Discharge

Assumption 3 defines the peak discharge as that which occurs when the entire drainage area contributes. However, it is possible that a premature peak discharge, which would be one that occurs before the entire drainage area contributes, could result when a watershed is nonhomogeneous. A premature peak discharge would likely be a result of a heterogeneous watershed. It would indicate that the land characteristics in a portion of a watershed are sufficiently different from that at other parts of the watershed to cause a maximum discharge prior to the time when the entire watershed contributes.

The hypothesis that peak discharge rates are the result of runoff from only a portion of the watershed is not new. Authors have proposed this under the concept of partial or variable source area hydrology. The source area is a dynamically expanding or shrinking subarea of the watershed that is highly variable during a storm event. It typically represents only a few percent of the total basin area (Hewlett, 1969). The variable source area concept was developed to account for the fact that neither stormflow nor baseflow is uniformly produced from the entire surface or subsurface area of a basin.

1.1.3 Assumption 4: Importance of Watershed Shape

Basin shape is typically not considered directly in hydrologic design. Assumption 4 indicates that watershed shape does not have an effect on the peak discharge rate. However, watersheds have an infinite variety of shapes, and watersheds of the same total area may differ considerably in terms of watershed length and width as well as the time that runoff requires to reach the outlet. Even under the traditional definition of peak discharge, it would be expected that watershed shape would affect the time of concentration, thus impacting the peak discharge. Some hypothetical watershed shapes are displayed in Figure 1.1. If the lines shown for each watershed reflect travel time, the equal area watersheds can be expected to produce quite different peak runoff rates.

The velocity concept of travel time assumes travel time can be computed as the ratio of the length to the velocity. If land cover is uniform throughout and two microwatersheds, one compact (see Figure 1.1(a)) and the other elongated (see Figure 1.1(g)), with constant runoff sheet flow velocities throughout each watershed, it is reasonable to assume that the runoff from different potions of the compact watershed would likely amass at the outlet more so than the elongated watershed. The extent of the amassment would depend on the watershed shape. Because of the importance of shape, numerous studies have resulted in a variety of shape indices.

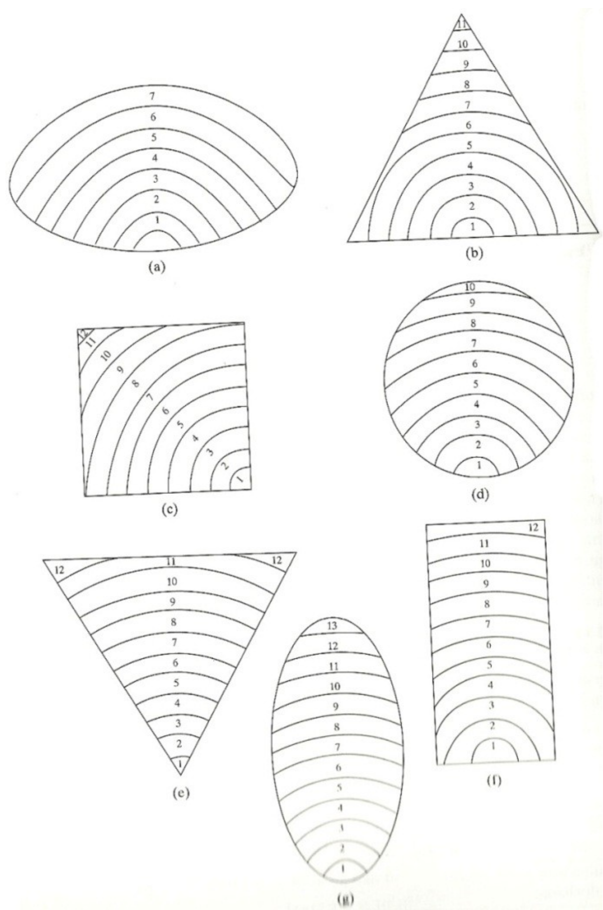


Figure 1.1 Hypothetical Watershed Shapes: (a) ellipse: side; (b) triangle: center; (3) square: corner; (d) circle; (e) triangle: vertex; (f) rectangle; and (g) ellipse: end (McCuen, 2005)

1.2 RESEARCH GOALS AND OBJECTIVES

The goal of this research is to improve the understanding of the effects of timing of microwatershed runoff on hydrologic design. The ability to accurately predict critical runoff rates for a microwatershed may be improved through the examination of how runoff travel times are assessed and calculated. This goal is achieved through several objectives.

The first objective is the development of a modeling approach that could demonstrate the effects of challenging the traditional assumptions that underlie the Rational method. These previously discussed assumptions of watershed spatial homogeneity, the traditional definition of the peak discharge, and the disregard of the watershed shape, are typically made to simplify use of the Rational method. Examination of the Rational method inputs, i.e., rainfall intensity, drainage area, and the spatial distribution of runoff coefficients throughout a watershed, is necessary to analyze this research objective because these parameters reflect the response of a watershed. It is unlikely that any watershed of even a minimal drainage area is homogeneous in watershed characteristics, including soils, land cover, slope, and any other variable that influences the timing of runoff. Even a watershed with a spatially uniform land cover can have considerable variation in slope, which could cause spatial variation of runoff velocities.

Conventional methods for evaluating these inputs need to be reconsidered for the analysis of microwatersheds in order to predict a more accurate peak discharge for hydrologic design. A format for modeling a watershed characterized by spatially varying characteristics seems to be the best approach to assessing the effects. The assessment of runoff travel times through heterogeneous watersheds could be used to show the potential occurrence of a premature peak. Thus, the modeling approach will need to allow for spatial variation of land cover and slope. Watershed heterogeneity would be inherent in the distribution of hydrologic characteristics, such as runoff coefficients and slopes, throughout the watershed.

The second objective is the development of guidelines for determining conditions that would produce a peak discharge rate that is higher than that produced when the entire watershed contributed runoff, i.e., under the conditions that underlie the assumptions of the Rational method. A premature peak would be expected to occur under certain hydrologic conditions of the watershed. The identification of these hydrologic conditions is necessary to obtain peak discharge estimates that will produce accurate designs. The assessment of the effect of variation in watershed characteristics, such as land cover and slope, is necessary to develop these guidelines. The guidelines could serve as a way to determine the watershed conditions that would cause a premature peak to occur. This knowledge is valuable in assessing flood risk.

In order to develop the guidelines, the modeling approach of the first research objective must be sensitive to the important factors. Guidelines cannot be substantiated unless the modeling analyses clearly show that the potential effect is realistic. Since the focus of the research is on the Rational method, the guidelines will be based on modeling with the Rational method as opposed to their occurrence on actual watersheds.

1.3 IMPLICATIONS OF RESEARCH

The completion of this research will provide improved understanding of the effect of timing of microwatershed runoff on computed discharge rates. If a premature peak occurs, then hydraulic designs that used a conventionally computed peak discharge may lead to underdesign. The implications to flood risk are evident. Hydraulic designs could be improved if more accurate estimates of peak discharge that reflect watershed heterogeneity and the consideration of a potential premature peak were used. Improvement in the understanding of stormwater runoff throughout a microwatershed could lead to better understanding of actual flood risk of exceedance.

The fulfillment of the first objective will provide a better assessment of the traditional assumptions of the Rational method. This is important because the Rational method is frequently used in hydrologic design. The analysis of the assumptions could show the need to consider spatial variability of watershed characteristics, such as land cover. The fulfillment of the second objective will provide guidelines for the design engineer to identify watershed conditions that would cause a premature peak to occur.

CHAPTER 2

LITERATURE REVIEW

2.1 INTRODUCTION

Research about predicting peak runoff from a watershed is a common theme in professional hydrology journals. Estimating peak runoff rates as accurately as possible is an important factor in designing drainage systems. Previous research that involved the Rational method, the Soil Conservation Service (SCS) Lag formula, and the determination of critical rainfall duration are discussed in this chapter.

2.2 THE RATIONAL METHOD

The Rational method relates the peak discharge (q_p, ft^3/sec), to the runoff coefficient (*C)*, rainfall intensity (*i,* in./hr), and drainage area (*A,* acres) with the following equation:

$$q_p = CiA \qquad (2\text{-}1)$$

The Rational method is widely applied to analyze the storm runoff of a watershed because of its simplicity and limited data requirements. The traditional Rational method associates design flow with hydrologic conditions under which the entire watershed contributes to runoff, and it equates critical storm duration with watershed concentration time.

2.1.1 Description of the Runoff Coefficient

The runoff coefficient reflects the relationship of runoff potential and land use, slope, and soil type. It reflects the proportion of rainfall that appears as runoff. Areas with more impervious surfaces typically experience greater volumes and rates of runoff. Table 2.1 is a summary of runoff coefficient values for common land uses (McCuen, 2005).

Table 2.1 Runoff Coefficients for the Rational Method (McCuen, 2005)

Description of Area/Surface	Recommended Value
Business	
Downtown	0.85
Neighborhood	0.60
Residential	
Single-family	0.40
Multiunits, detached	0.50
Multiunits, attached	0.70
Residential (suburban)	0.35
Apartment	0.60
Industrial	
Light	0.65
Heavy	0.75
Parks, cemeteries	0.20
Playgrounds	0.30
Railroad yard	0.30
Unimproved	0.20
Pavement	0.85
Asphalt and concrete	0.80
Brick	
Roofs	0.85
Lawns, sandy soil	
Flat, 2%	0.08
Average, 2% to 7%	0.13
Steep, 7%	0.18
Lawns, heavy soil	
Flat, 2%	0.15
Average, 2% to 7%	0.20
Steep, 7%	0.30

2.1.2 Rainfall Intensity

The rainfall intensity, *i*, is the average rainfall rate for a specific frequency and rainfall duration. Using these two inputs, i.e., the frequency and duration, the rainfall intensity is obtained from an intensity-duration-frequency (IDF) curve. An IDF curve is developed from a statistical analysis of long-term rainfall records. It is a plot of intensity vs. storm duration for selected return periods (i.e., exceedance frequencies). An IDF curve can be represented using the following equations:

$$i = \frac{a}{D+b} \quad for\ D \leq 2\ hr \tag{2-2}$$

$$i = cD^{d} \quad for\ D > 2\ hr \tag{2-3}$$

where the rainfall intensity, i, is in inches per hour, the duration, *D*, is in hours, and *a, b, c,* and *d* are coefficients that vary depending on the frequency and geographical region of interest. In the Rational method, the rainfall intensity is determined using the frequency along with a duration that is equal to the time of concentration. The method assumes that the intensity remains constant for the duration of the storm event. Thus, the total depth of rainfall equals the product of the intensity and duration. The time of concentration is used as the storm duration because the Rational method is often used for design problems of urban areas with small drainage areas and short times of concentrations. This assumption is based on the concept that flooding is a result of short-duration storms (McCuen, 2005).

2.1.3 Drainage Area

The Rational method is appropriate for estimating peak discharge rates for small drainage areas, i.e., microwatersheds. In calculating the peak discharge with the Rational method, two assumptions are made: (1) the rainfall intensity is constant over the duration of the storm and (2) the rainfall is distributed uniformly over the entire drainage area. However, as the drainage area increases, this assumption becomes less realistic as it is likely that the rainfall intensity will vary spatially and temporally. Microwatersheds consist of drainage areas that are small enough to apply the assumption of uniform rainfall distribution.

2.1.4 Modified Rational Formulas

Hua et al. (2003) presented a modified Rational formula intended for small elliptical basins, where a small basin was defined to have an upper limit of 25 km^2. They challenged the assumption of the traditional rational formula that the rainfall is uniformly distributed in the whole duration of a design storm. Hua et. al modified this assumption to be that the rainfall is uniformly distributed only in each time interval of the design storm hyetograph. This modification means that the Rational formula can be reasonable extended to include cases where the rainfall duration is less than the watershed time of concentration. In Figure 2.1, an ellipse-like basin is shown where W is the maximum width of the basin, L is the length of the basin, and T_c is the basin's time of concentration. The distance d is the distance between any two of the basin isochrones that are shown as the horizontal dotted lines. The duration of effective rainfall for design is denoted by D, which is the routing time of flow through the distance d.

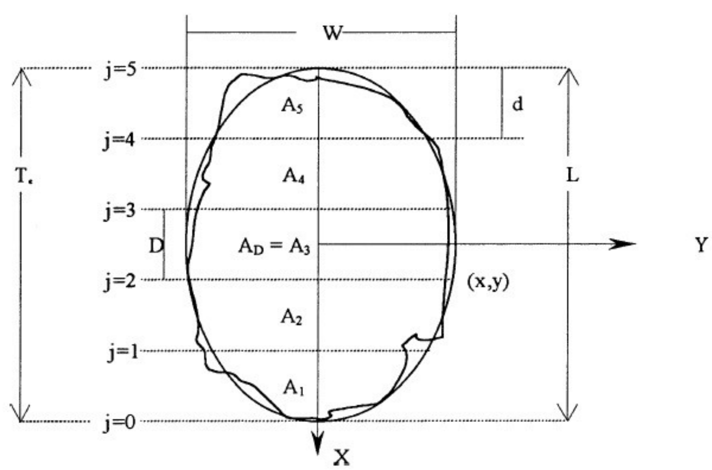


Figure 2.1 An Ellipse Like Basin Generalization (Hua, 2003)

In SI units, the traditional Rational formula is

$$Q = 0.278\, C\, i\, A \tag{2-4}$$

where Q is in m^3/s, i is in mm/hr, and A is in km^2. Hua et al. modified the traditional Rational formula for situations in which the duration of effective rainfall for design is less than the basin's time of concentration (i.e., $D < T_c$). Using this assumption, the peak flow can be estimated as

$$Q = 0.278\, R_{D,p}\, A_D \;/\; D \tag{2-5}$$

where $R_{D,p}$ is the design effective rainfall in the design time interval D (mm).

Guo (2001) expanded the Rational method into the Rational hydrograph method for small urban watersheds. In his study, the present runoff rate was defined as the accumulated precipitation over the past up to the time of concentration, where the time of concentration was considered as the system memory. The major assumptions of Guo's model included that the watershed storage effect was negligible and the present runoff rate was linearly related to the accumulated rainfall depth over the past up to the time of concentration. Using this methodology, Guo generated the complete runoff hydrograph under a continuous non-uniform hyetograph. When the continuous hyetograph is uniform in time the method reduces to the Rational method.

2.3 TIME OF CONCENTRATION

McCuen et al. (1984) analyzed and compared eleven equations for estimating the time of concentration. Data were collected from 48 urban watersheds, all of which were less than 4,000 acres. The mean time of concentration for all of the watersheds was computed from rainfall and hydrograph data. Estimates of the time of concentration were computed using eleven equations and then the goodness-of-fit statistics, primarily the accuracy and bias, were used as the comparison criteria. McCuen et al. (1984) concluded that a velocity based method is most likely to yield an accurate estimate of the time of concentration due to the segmented fashion in which it is applied. They also concluded that watersheds consisting of nonhomogeneous land and significant amounts of channel flow require a mixed method to estimate the time of concentration.

Wong (1997) derived a time of concentration formula by coupling the kinematic wave equations with the Darcy-Weisbach friction equation. The formula is applicable to a portion of overland plane subject to uniform rainfall excess with a single flow regime. The results show that application of the formula for varying flow regime yields a longer estimate of the time of concentration than if based on a single flow regime.

Schmid (1997) made an analysis that permitted “the strongly nonlinear interactions between rainfall intensity, critical duration, ponding time, infiltration and maximum overland peak flow to be reproduced.” He derived general relationships to determine the occurrence of maximum peak flows from infiltrating plane by coupling kinematic theory with regard to overland flow with a physically based representation of the infiltration process. The assumption of uniform distribution, spatially and temporally, was assumed for rainfall through the use of an IDF relationship. Schmid (1997) concluded that the traditional method of setting time of concentration equal to storm duration should be replaced by the potential critical storm duration for cases where it is smaller than the time of concentration.

2.4 SOIL CONSERVATION SERVICE (SCS) LAG FORMULA

The SCS defines watershed lag as the time from the center of mass of the excess rainfall to the peak discharge. The SCS indicates that the time of concentration equals 1.67 times the lag (McCuen, 2005). Using this assumption, the following equation presents the time of concentration in minutes:

$$t_c = 0.00526L^{0.8}\left(\frac{100}{CN} - 9\right)^{0.7} S^{-0.5} \tag{2-6}$$

where L is the watershed length (ft), CN is the runoff curve number, and S is the watershed slope (ft/ft).

2.4.1 Runoff Curve Number

The runoff curve number, *CN*, is an empirical parameter used in the SCS Lag method to determine the approximate amount of rainfall runoff. The *CN* for an area is a function of three main factors: the hydrologic soil group, cover complex, and the hydrologic condition. The *CN* serves as an index that represents the combination of these three factors. Table 2.2 shows the runoff curve numbers for several different land uses, hydrologic soil groups, and cover complex conditions.

Table 2.2 Runoff Curve Numbers for Average Watershed Condition (McCuen, 2005)

Land Use	Curve Numbers for Hydrologic Soil Groups			
	A	B	C	D
Fully developed urban areas (vegetation established)				
Lawns, open spaces, parks, golf courses, etc.				
Good condition: grass cover on 75% or more of the area	39	61	74	80
Fair condition: grass cover on 50% to 75% of the area	49	69	79	84
Poor condition: grass cover on 50% or less of the area	68	79	86	89
Paved parking lots, roofs, driveways, etc.	98	98	98	98
Streets and roads				
Paved with curbs and storm sewers	98	98	98	98
Gravel	76	85	89	91
Dirt	72	82	87	89
Paved with open ditches	83	89	92	93
Commercial and business areas	89	92	94	95
Industrial districts	81	88	91	93
Row houses, townhouses, and residential homes (1/8 acre or less)	77	85	90	92
Residential: average lot size				
1/4 acre	61	75	83	87
1/3 acre	57	72	81	86
1/2 acre	54	70	80	85
1 acre	51	68	79	84
2 acre	46	65	77	82
Developing urban areas (no vegetation established)				
Newly graded area	77	86	91	94

The hydrologic soil groups are classified by the letters A, B, C, and D, depending on the characteristics of the area. The *CN* for a particular land use typically has four values, one for each soil group, where, Group A would have the lowest *CN* and Group D the highest. A brief description of the characteristics of each soil group is shown in Table 2.3.

Table 2.3 Characteristics of Soils Assigned to Soil Groups (McCuen, 2005)

Group A:	Deep sand; deep loess; aggregated silts
Group B:	Shallow loess; sandy loam
Group C:	Clay loams; shallow sandy loams; soils usually high in clay
Group D:	Soils that swell significantly when wet; heavy plastic clays; certain saline soils

The SCS cover complex classification is a function of three factors: land use, treatment or practice, and hydrologic condition. Three classifications of the hydrologic condition of the land, which depend on the density of vegetation or ground cover, are defined in Table 2.4. An area that has poor hydrologic conditions would relate to a higher *CN* than an area with good hydrologic conditions.

Table 2.4 Classification of Hydrologic Condition (McCuen, 2005)

Poor:	Heavily grazed or regularly burned areas. Less than 50% of the ground surface is protected by plant cover or brush and tree canopy.
Fair:	Moderate cover with 50 to 75% of the ground surface protected by vegetation.
Good:	Heavy or dense cover with more than 75% of the ground surface protected by vegetation.

2.4.2 Relation between Curve Number and Runoff Coefficient

McCuen and Bondelid (1981) established a relationship between the curve number of the SCS TR-55 graphical method and the runoff coefficient of the Rational method. The SCS TR-55 graphical method is given by:

$$Q = q_u A_1 V \tag{2-7}$$

where Q is the peak discharge (cfs), q_u is the unit peak discharge (cfs/mi^2/in. of runoff), A_1 is the drainage area (mi^2), and V is the runoff volume (in.). The runoff volume is computed as,

$$V = \frac{(P-0.2S)^2}{P+0.8S} \tag{2-8}$$

where P is the 24 hour precipitation volume (in.) and S is calculated as,

$$S = \frac{1000}{CN} - 10 \tag{2-9}$$

McCuen and Bondelid represented the graph of unit peak discharge versus the time of concentration with the exponential relationship:

$$q_u = b_0 {t_c}^{b_1} \tag{2-10}$$

where b_0 and b_1 are fitting coefficients. The rainfall intensity was also represented by an exponential relationship where d_0 and d_1 are fitting coefficients:

$$i = d_0 {t_c}^{d_1} \tag{2-11}$$

McCuen and Bondelid used the exponential relationships of Equations 2-10 and 2-11 to provide the following equality:

$$C\left(d_0 {t_c}^{\boldsymbol{d_1}}\right)A = \left(b_0 {t_c}^{b_1}\right) \frac{A}{640} \left(\frac{(P-0.2S)^2}{(P+0.8S)}\right) \tag{2-12}$$

Solving the Equation 2-12 for C results in:

$$C = \frac{(b_0 {t_c}^{b_1})}{640\,(\boldsymbol{d_0} {t_c}^{\boldsymbol{d_1}})} \left(\frac{(P-0.2S)^2}{(P+0.8S)}\right) \tag{2-13}$$

Since S is a function of the CN, Equation 2-13 directly relates C and CN. Figure 2.2 shows a graphical representation of the relationship between C and CN for several return periods (T).

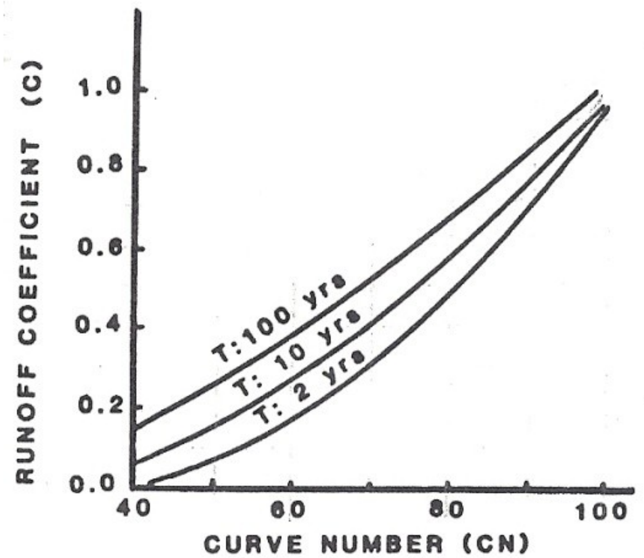


Figure 2.2 Relationship between Runoff Coefficient and Curve Number for Baltimore, MD (McCuen and Bondelid, 1981)

Johnson (1980) coupled the Rational formula with SCS curve numbers to create a graphical method for estimating peak runoff rate. Johnson expressed the runoff coefficient as:

$$C = 1 - \frac{S}{P}\left(1.2 - \frac{S}{P+0.8S}\right) \tag{2-14}$$

Thus, a unique C value can be determined for any design event with an established return period, time of concentration, intensity, and CN. Figure 2.3 shows the resulting relationship between the C, CN, and time of concentration.

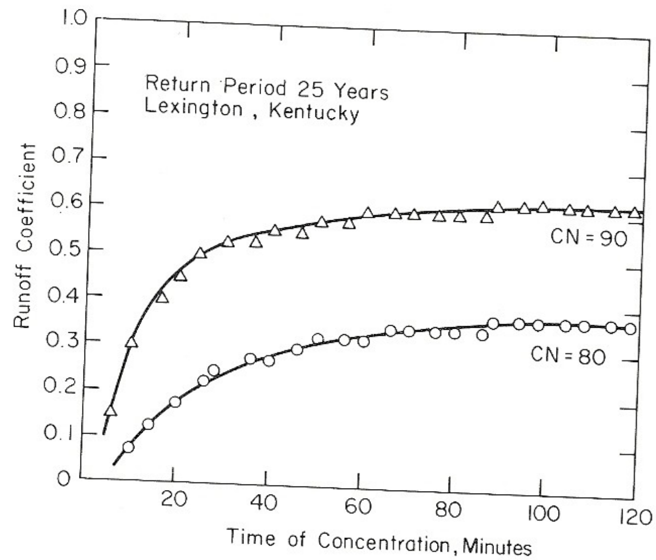


Figure 2.3 Variation of Runoff Coefficients (C) for CNs of 80 and 90 for 25-Year Return Period (Johnson, 1980)

2.5 THE EFFECTS OF URBANIZATION

Carluer and Marsily (2004) examined the influence of man-made networks, such as roads and drainage systems, on the hydrology of a watershed. In particular, they analyzed the impact of such systems in rural landscapes on the accelerated transport of contaminates from fields. Numerical simulations with a two-dimensional saturated/unsaturated flow code were used to generally represent the influence of ditches. A model was constructed to represent the influence of man-made networks on a study site catchment in Brittany, France. However, Carluer and Marsily indicated that their model could not be applied in a totally satisfactory way due to the lack of adequate data for calibration. Carluer and Marsily (2004) concluded that man-made networks can have a very significant effect on the functioning of a watershed, which enforces the need to utilize an improved distributed model for watershed management.

Bledsoe and Watson (2001) examined potential changes in a flow regime associated with varying levels of watershed imperviousness. The predicted changes in flow parameters were then employed with risk-based models of channel form and instability. They determined that low levels of imperviousness (10 to 20%) have the potential to severely destabilize streams. Bledsoe and Watson stated that watershed-specific conditions also influenced the impact of imperviousness and that calibration should be performed regionally since different stream types exhibit different levels of resilience.

2.6 CONTRIBUTING DRAINAGE AREA

The partial or variable source area concept was proposed by forest hydrologists to describe how the flow of water in a stream is under the influence of dynamically expanding or shrinking source area (Hewlett and Hibbert, 1967). The source area typically represents only a small percentage of the total basin area, but is highly variable during a storm event. Figure 2.4 (Hewlett, 1969) shows the source area pattern of an example permeable basin with a dendritic drainage network. The hydrographs show how streamflow increases as the variable source extends into swamps, shallow soils and ephemeral channels.

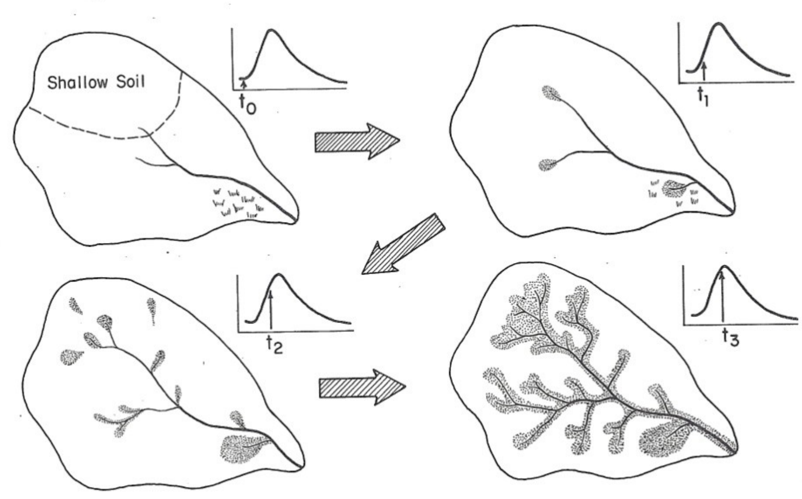
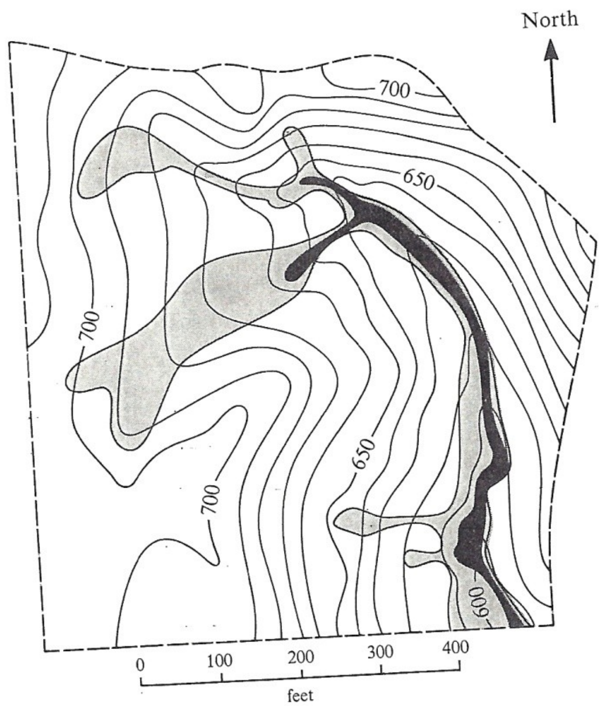


Figure 2.4 The Variable Source Area Concept (Hewlett, 1969).

Dunne and Leopold (1978) analyzed a Vermont watershed to assess how saturation overland flow varied within the basin. Their results showed that only a portion of the drainage basin contributed saturation overland flow, namely, the lower parts of swales and hillsides that became saturated during a storm event. As a storm event progressed, the saturated area expanded upslope, which caused more of the subarea to contribute saturation overland flow. Figure 2.5 shows the expansion of saturated areas during a single rainstorm of 46 mm.



The soild black shows the saturated area at the beginning of the rain; the lightly shaded area is saturated by the end of the storm and is the area over which the water table had risen to the ground surface.

Figure 2.5 Saturated Areas Showing Expansion During a Storm Event (Dunne and Leopold, 1978)

2.7 WATERSHED SHAPE

Several equations have been developed for quantification of basin shape. Horton (1932) defined the dimensionless form factor, R_f, as the ratio of the basin area, A_w, to the basin length, L, squared:

$$R_f = \frac{A}{L^2} \tag{2-15}$$

Miller (1953) introduced the circularity ratio, R_C, as:

$$R_c = \frac{A_W}{A_P} = \frac{4\pi A_W}{{L_P}^2} \tag{2-16}$$

where, A_W is the basin area of a given order w, A_P is the area of a circle with a circumference equal to the basin perimeter, L_P. As the basin shape approaches a circle, the value of the circularity ratio approaches 1.0.

Schumm (1956) defined the elongation ratio, R_E, as the ratio of the diameter of a circle, D_C, with the same area as that of the basin, to the maximum basin length, L:

$$R_E = \frac{D_C}{L} \tag{2-17}$$

As the shape of the drainage basin approaches a circle, the value of the elongation ratio approaches 1.0.

Chorley et al. (1957) introduced the lemniscate ratio, which compared basin shape to the mathematical teardrop shape, the lemniscate. The lemniscate function was defined as

$$L_F = L \cos a\theta \tag{2-18}$$

where, L_f is the radius in polar coordinates, L is the basin length, and θ is the angle in polar coordinates. The coefficient a determines the rotundity of the basin, where, when a equals 1, the basin shape is circular.

CHAPTER 3

GUIDELINE DEVELOPMENT

3.1 CHARACTERIZING THE RATIONAL METHOD INPUT PARAMETERS

A series of analyses were made for the purpose of understanding the general conditions under which the maximum discharge from a subarea of a watershed could exceed the discharge for the case where the entire watershed is contributing to the runoff. Such a peak is referred to as being premature. If the layout and characteristics of a watershed result in premature peaks, flood control facilities may be under designed. Trials were created to better understand how the inputs of travel time and the runoff coefficient affect the occurrence of premature peaks.

The initial trials with the linear model consisted of twenty equal-area cells (see **Figure 3.1**). Each cell was represented by a C value and a corresponding travel time (T_t) within the cell. The contributing drainage area was calculated for each cell as a summation over all cells that had travel times less than or equal to a series of set values. The C value was averaged over the corresponding cells to yield C_{avg} for all cells with a cumulative travel time less than each set value. A peak discharge rate was computed for each set of cells with travel times less than or equal one of the values. As more and more of the watershed is assumed to contribute runoff to the outlet, the drainage area increases, the time of concentration increases, and the rainfall intensity decreases. Ultimately, the entire watershed contributes, which yields a peak discharge that would reflect the peak discharge normally associated with the Rational method.

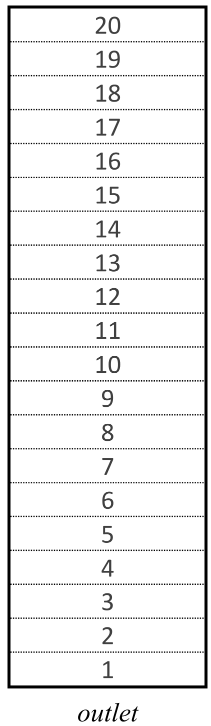


Figure 3.1 Schematic of 20-cell Linear Watershed

The intensity for a given set of cells was calculated as a function of the total T_t value for the contributing portion of the watershed. For these initial exploration trials, the approximate coefficients for the Baltimore, Maryland, IDF curve were used:

$$i = \frac{2}{T_t + 0.3} \quad for\ T_t \leq 2\ hr \tag{3-1}$$

Actual coefficients for traditionally applied return periods in Baltimore, Maryland, are shown in Table 3.1, and the coefficients are inserted into the following equations:

$$i = \frac{a}{D + 0.333} \quad for\ D \leq 2\ hr \tag{3-2a}$$

$$i = cD^{-0.75} \quad for\ D > 2\ hr \tag{3-2b}$$

These equations were used for most analyses once general effects were identified.

Table 3.1 Coefficients for the Baltimore, Maryland, Intensity-Duration-Frequency Model

T (years)	*a*	*c*
2	1.88	1.367
5	2.54	1.873
10	3.20	2.280
25	3.60	2.665
50	4.05	2.981
100	4.61	3.364

Discharge rates were computed for every group of cells starting with the first cell at the watershed outlet, then the first two cells, etc. This provided an array of discharge rates calculated by beginning at the bottom of the watershed, the outlet, and then systematically increasing the area by each one of the twenty cells, which resulted in twenty discharge rates. As each cell was included, the time of concentration, rainfall intensity, and area would change, which was reflected in the discharge rate computed for that portion of the watershed. The location that corresponded to the largest of the twenty discharges was identified from the twenty computed discharges. A peak discharge that occurred without the entire drainage area contributing was deemed a premature peak. The effects of varying the distribution of only travel time, only the runoff coefficient, and then both components were examined within the simplified watershed in order to characterize how these parameters influenced whether the peak occurred prematurely.

The distribution of travel times and runoff coefficients were varied between trials to assess how the variations impacted the occurrence of a premature peak. The arrangements of the trials discussed in Chapter 3 are shown in Table 3.2. These trials were used to develop the initial guidelines for the occurrence of a premature peak.

Table 3.2 Arrangements of the 20-cell Linear Trials Discussed in Chapter 3

Trial	C_L	C_U	ΔC	Area C_L (%)	Premature Peak?	Comment
A	0.2	0.2	0.0	-	No	Linear Tt distribution; homogeneous C
B	0.2	0.2	0.0	-	Yes	Nonlinear Tt distribution; homogeneous C
C	0.2	0.2	0.0	-	Yes	Nonlinear Tt distribution; homogeneous C
D	0.5	0.2	0.3	50	Yes	Effect of ΔC; compared to Trial E
E	0.3	0.2	0.1	50	No	Effect of ΔC; compared to Trial D
F	0.5	0.2	0.3	50	Yes	Effect of magnitude of C values; compared to Trial G
G	0.9	0.6	0.3	50	No	Effect of magnitude of C values; compared to Trial F
H	0.9	0.6	0.3	70	Yes	Effect of area of lower portion; compared to Trial I
I	0.9	0.6	0.3	55	No	Effect of area of lower portion; compared to Trial H

3.1.1 Distribution of Travel Time

To assess the general relationship between the distribution of travel times and the occurrence of premature peaks, the trials were modified to hold the *C* coefficient constant over the entire watershed. This would reflect the condition of homogeneous land cover/use. To provide standardized results the range of travel times was varied from 0.00 at the outlet to 1.00 hr at the top of the watershed. Travel times were modeled such that the distribution reflected a uniformly increasing function until a condition where the change in travel time between cells occurred more abruptly; this would reflect a change in some characteristic that influenced travel times, such as slope.

Table 3.3(a) shows an example of the distribution of travel times. They are uniformly distributed from 0.00 at the outlet to 1.00 hr at the upper end of the watershed. Trial A shows that when travel time is uniformly linearly increasing over the watershed and the land cover is homogeneous, i.e., C is constant, that the peak is expected to occur when the entire watershed contributes, which occurs at T_t = 1.00 hr (see Table 3.3b)). Figure 3.2 and the last column of Table 3.3(b) include the discharge rates as a function of the travel time. This layout was not expected to produce a premature peak because the increasing drainage area offset the decreasing intensity that results from the increasing travel time. Thus, the lack of a premature peak is reasonable.

Table 3.3 Trial A: Uniform Distribution of Travel Time

(a)

C	T_t
0.2	1.00
0.2	0.95
0.2	0.90
0.2	0.85
0.2	0.80
0.2	0.75
0.2	0.70
0.2	0.65
0.2	0.60
0.2	0.55
0.2	0.50
0.2	0.45
0.2	0.40
0.2	0.35
0.2	0.30
0.2	0.25
0.2	0.20
0.2	0.15
0.2	0.10
0.2	0.05

outlet

(b)

T_t	ΣA	C_{avg}	i	q
1.00	20	0.2	1.5385	6.1538
0.95	19	0.2	1.6000	6.0800
0.90	18	0.2	1.6667	6.0000
0.85	17	0.2	1.7391	5.9130
0.80	16	0.2	1.8182	5.8182
0.75	15	0.2	1.9048	5.7143
0.70	14	0.2	2.0000	5.6000
0.65	13	0.2	2.1053	5.4737
0.60	12	0.2	2.2222	5.3333
0.55	11	0.2	2.3529	5.1765
0.50	10	0.2	2.5000	5.0000
0.45	9	0.2	2.6667	4.8000
0.40	8	0.2	2.8571	4.5714
0.35	7	0.2	3.0769	4.3077
0.30	6	0.2	3.3333	4.0000
0.25	5	0.2	3.6364	3.6364
0.20	4	0.2	4.0000	3.2000
0.15	3	0.2	4.4444	2.6667
0.10	2	0.2	5.0000	2.0000
0.05	1	0.2	5.7143	1.1429

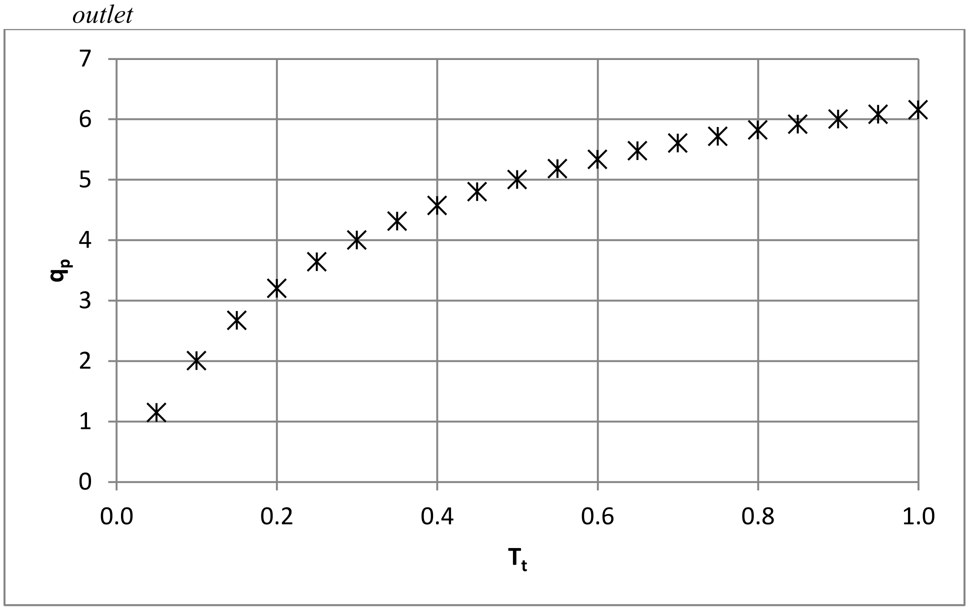


Figure 3.2 Trial A: Discharge for Uniform Distribution of Travel Time

A second analysis, Trial B, was made to show the effect of a nonlinear change in the travel time. Table 3.4(a) shows the distribution of travel times over a watershed with an abrupt change at a time of 0.80 hours. This means that the cumulative area that contributes runoff at the outlet is greater than at the same time for Trial A. A discharge of 6.18 cfs occurred at T_t = 0.80 hr (Table 3.4(b)) versus 5.82 cfs for Trial A, i.e., the uniformly varying case. Trial B, which is shown Table 3.4, differs from Trial A in that a single T_t value, presented in bolded font, was modified to have a gap in the linearity of the travel times, which could physically represent a change in the properties of the land within the watershed.

The comparative results, i.e., Trial A versus Trial B, show that even the small variation between the two trials yields a premature peak discharge that occurs at T_t = 0.80 hr. The difference in the T_t values appears in the summation of the cell areas, $\sum A$. Since the C_{avg} and intensity corresponding to T_t of 0.80 hr does not change between the two trials, then the premature peak in Trial B occurs because the $\sum A$ by T_t = 0.80 hr is significant enough to result in a slightly greater discharge than that of the entire watershed. In this case, the rapid increase in drainage area with the same rainfall intensity produces a peak that is greater than when the entire watershed is contributing.

Table 3.4 Trial B: Nonlinear Change in *Tt* Occurring at *Tt* = 0.80

(a)

C	T_t
0.2	1.00
0.2	0.95
0.2	0.90
0.2	**0.80**
0.2	0.80
0.2	0.75
0.2	0.70
0.2	0.65
0.2	0.60
0.2	0.55
0.2	0.50
0.2	0.45
0.2	0.40
0.2	0.35
0.2	0.30
0.2	0.25
0.2	0.20
0.2	0.15
0.2	0.10
0.2	0.05

outlet

(b)

T_t	ΣA	C_{avg}	i	q
1.00	20	0.2	1.5385	6.1538
0.95	19	0.2	1.6000	6.0800
0.90	18	0.2	1.6667	6.0000
0.80	17	0.2	1.8182	6.1818
0.75	15	0.2	1.9048	5.7143
0.70	14	0.2	2.0000	5.6000
0.65	13	0.2	2.1053	5.4737
0.60	12	0.2	2.2222	5.3333
0.55	11	0.2	2.3529	5.1765
0.50	10	0.2	2.5000	5.0000
0.45	9	0.2	2.6667	4.8000
0.40	8	0.2	2.8571	4.5714
0.35	7	0.2	3.0769	4.3077
0.30	6	0.2	3.3333	4.0000
0.25	5	0.2	3.6364	3.6364
0.20	4	0.2	4.0000	3.2000
0.15	3	0.2	4.4444	2.6667
0.10	2	0.2	5.0000	2.0000
0.05	1	0.2	5.7143	1.1429

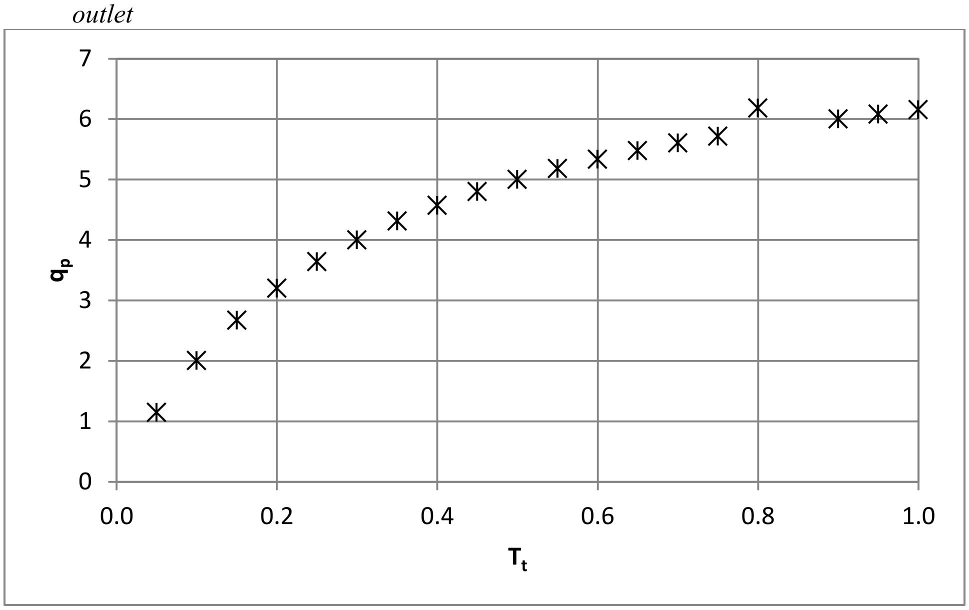


Figure 3.3 Trial B: Nonlinear Change in Tt Occurring at Tt = 0.80

Additional analyses were made to show the effect of changes in the travel time on discharge rates and their time of occurrence. The change in travel time was systematically varied to create a gap that was larger than that for Trial B. This could physically represent a change in terrain characteristics (e.g., slope, land cover, roughness) that influence the timing of runoff. The setup of travel times and results for Trial C are shown in Table 3.5. In Trial C, the change in travel time occurred at 0.70 hr with the $\sum A = 17$. The results of Trial C show that the peak occurs prematurely at $T_t = 0.70$ hr, which coincides with the gap in travel time. The premature peak at $T_t = 0.70$ hr may be attributed to the contributing $\sum A$, which is sufficient to yield a greater discharge than at $T_t = 1.00$ hr. This increase in drainage area offsets the decrease in intensity. This indicates that the runoff characteristics of the upper reaches of the watershed, i.e., those after the travel time gap, were not sufficient to cause the peak discharge to continue increasing; thus a premature peak occurs.

In both Trials B and C, the runoff coefficients were held constant throughout the watershed to more clearly isolate the effect of the change in travel time. When comparing Trial C to Trial B, the difference in travel time means that the runoff in Trial C, $T_t = 0.70$ hr, is discharging to the outlet at a faster rate than in Trial B, $T_t = 0.80$ hr, but for the same quantity of area, $\sum A = 17$. Additionally, the discharge of 6.80 cfs is greater in Trial C than the discharge of 6.18 cfs in Trial B despite having the same contributing $\sum A$ for the premature peak. This occurs because the intensity for a shorter T_t is higher. The intensity at the premature peak, where $\sum A = 17$, is 2.00 in./hr for Trial C and 1.82 in./hr for Trial B.

Table 3.5 Trial C: Nonlinear Change in *Tt* Occurring at *Tt* = 0.70

(a)

C	T_t
0.2	1.00
0.2	0.95
0.2	0.90
0.2	**0.70**
0.2	0.70
0.2	0.65
0.2	0.65
0.2	0.55
0.2	0.55
0.2	0.50
0.2	0.50
0.2	0.45
0.2	0.40
0.2	0.35
0.2	0.30
0.2	0.25
0.2	0.20
0.2	0.15
0.2	0.10
0.2	0.05

outlet

(b)

T_t	ΣA	C_{avg}	i	q
1.00	20	0.2	1.5385	6.1538
0.95	19	0.2	1.6000	6.0800
0.90	18	0.2	1.6667	6.0000
0.70	17	0.2	2.0000	6.8000
0.65	15	0.2	2.1053	6.3158
0.55	13	0.2	2.3529	6.1176
0.50	11	0.2	2.5000	5.5000
0.45	9	0.2	2.6667	4.8000
0.40	8	0.2	2.8571	4.5714
0.35	7	0.2	3.0769	4.3077
0.30	6	0.2	3.3333	4.0000
0.25	5	0.2	3.6364	3.6364
0.20	4	0.2	4.0000	3.2000
0.15	3	0.2	4.4444	2.6667
0.10	2	0.2	5.0000	2.0000
0.05	1	0.2	5.7143	1.1429

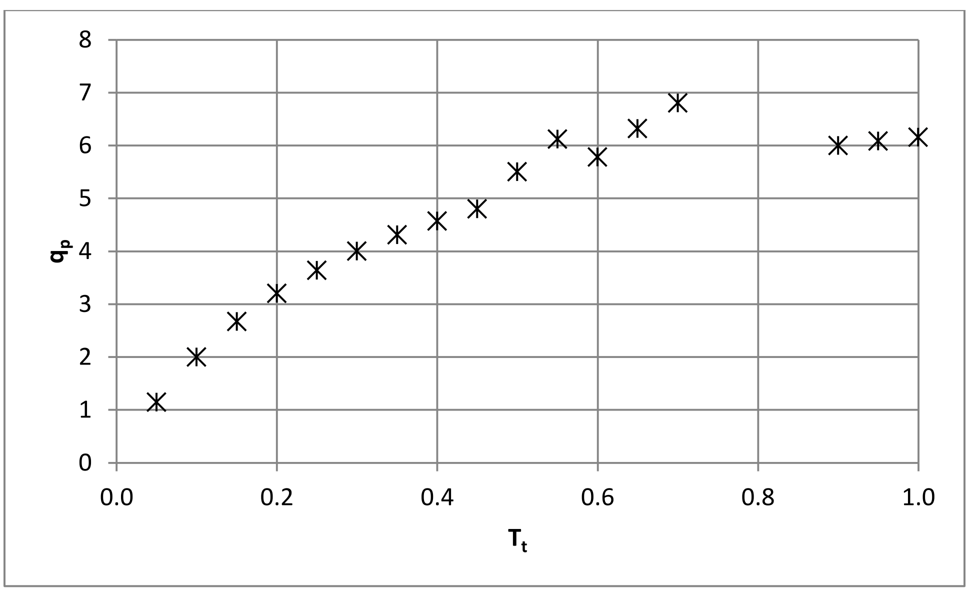


Figure 3.4 Trial C: Nonlinear Change in *Tt* Occurring at *Tt* = 0.70

3.1.2 Fitting a Function to Discharge vs. Travel Time

The models used in the previous section represented the watersheds using a set of discrete points. In reality, watershed runoff is continuous. In an attempt to better represent the relationship between peak discharge and the contributing area, polynomials were fitted to the discrete data. The functions are used only for descriptive purposes.

The rate of change of a function is described by the first derivative, i.e., the slope. For analyses of this issue, the slope of a curve describes the rate of change of the discharge with the travel time. The first derivative of a quadratic function yields a constant rate of change, which means that the cubic function should be used in order to provide a more conceptually accurate portrayal of the data. Due to the monotonic properties of the cubic function and the range of the trial data, the point at which the slope equals zero corresponds with the maxima of the function, which is the peak discharge of the watershed. The first derivative of a cubic function can be calculated to determine the slope of a function and the location of the maximum peak discharge. The equation for a cubic function is:

$$f(t) = at^3 + bt^2 + ct + d \tag{3-3}$$

where a, b, c, and d are coefficients. The derivative of Equation 3-3 can be calculated:

$$f'(t) = 3at^2 + 2bt + c \tag{3-4}$$

Setting the derivative equal to zero provides the time at which the maximum peak discharge occurs:

$$t_p = \frac{-b \pm \sqrt{b^2 - 4ac}}{2a} \tag{3-5}$$

Under the assumption of the cubic, the time of the peak discharge can be determined.

Land cover, travel times, and the contributing areas within a watershed occur continuously, rather than the discrete intervals of the previous case studies. Therefore, the discharge versus travel time data were fitted with polynomials so that the peak discharge and the time of the peak could be determined by finding the derivative of the polynomial functions. Polynomial functions were fitted to the distributions of discharge versus travel time to determine if the relationship could be accurately represented using a known function. Quadratic and cubic functions were fitted and then compared to determine which would provide the better fit to the data.

In most cases, the quadratic and cubic functions provided similar goodness-of-fit statistics with high coefficients of determination (R^2). Figure 3.5 displays trial data that were fitted with both functions. In this trial, the resulting quadratic and cubic relationships were, respectively,

$$Q = -17.678{T_t}^2 + 23.869T_t - 0.477 \tag{3-6a}$$

$$Q = -11.932{T_t}^3 + 0.2842{T_t}^2 + 16.97T_t + 0.1191 \tag{3-6b}$$

The quadratic function had an R^2 value of 0.95, and the cubic had an R^2 value of 0.96, which indicates that both functions provide a good fit.

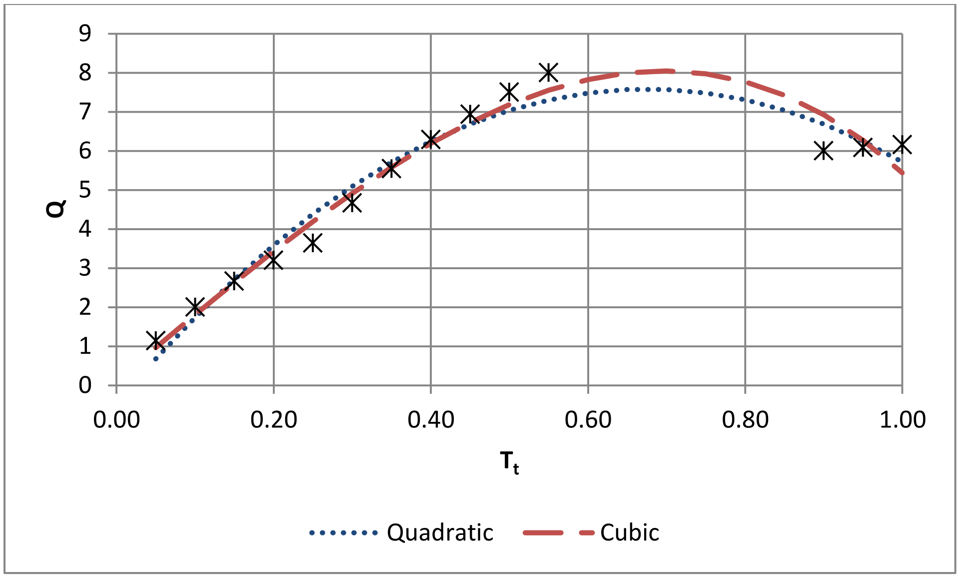


Figure 3.5 Fitting Quadratic and Cubic Functions

3.2 GENERAL GUIDELINES FOR THE OCCURRENCE OF A PREMATURE PEAK

General guidelines about the occurrence of when premature peaks occur were developed from making many analyses such as Trials A, B, and C. These analyses involved the distribution of the travel time, which manifests itself in the Rational method as the $\sum A$, the intensity, and the spatial distribution of the *C* values. Premature peak discharge rates are likely to occur when:

- The land use pattern for the watershed is heterogeneous with high *C* land covers near the outlet and lower *C* values in the upper reaches of the watershed.
- Watershed characteristics near the outlet are such that relatively fast travel times are produced, with the upper part of the watershed subject to relatively slower travel times.

- The rainfall intensity decreases as the cumulative travel time increases; therefore, a premature peak discharge will occur if the cumulative area stops increasing sufficiently to overcome the decrease in intensity.
- If a premature peak does occur, it is likely to occur at the point where an abrupt change in watershed condition occurs.

3.2.1 Premature Peak Resulting from the Difference between *C* Values

Analyses were made to test the hypothesis that a premature peak can occur when the land use shows a significant decrease in the *C* value in the upper reaches of a watershed. The difference between *C* values, ΔC, can be defined as the difference between the *C* value at the lower end of the watershed and the *C* value at the upper end of the watershed. A positive ΔC reflects the scenario where the *C* value at the lower end is greater than the upper end of the watershed. A positive ΔC would suggest that the area closest to the outlet is more urbanized. ΔC is one of the factors in determining whether a premature peak occurs because it is a characteristic that defines the spatial distribution of *C* values. A large ΔC physically represents a heterogeneous watershed, i.e., one that has significantly different properties, such as land cover.

In Table 3.6(a), Trial D is shown where the lower 50% of the watershed has a *C* value of 0.5 and the upper 50% has a *C* value of 0.2, which would yield a ΔC of 0.3. The results, which are given in Table 3.6(b), show that a premature peak occurs at the time corresponding to the change in C values, $T_t = 0.45$. Trial E, presented in Table 3.6, shows a watershed with a smaller ΔC than Trial D. Table 3.6(a) shows Trial E where the lower 50% of the watershed has a C value of 0.3 and the upper 50% has a *C* value of 0.2, which would yield a ΔC value of 0.1. For the conditions in Trial E a premature peak does not occur because the effect of the difference in *C* is not sufficient to produce a premature peak. Figure 3.6 shows the results of Trials D and E, providing a graphical comparison of the effect of ΔC on the occurrence on a premature peak between these two trials. The point at which the peak discharge occurs is signified by the filled mark for each of the data series.

Trials D and E indicate that ΔC has an effect on the occurrence of a premature peak. A general conclusion can be made that the larger the difference in *C* values, i.e., the larger the ΔC, the more likely a premature peak will occur. For example, a watershed that was characterized by urbanized areas near the outlet and forested areas in the upper portion would be more likely to have a premature peak than a watershed consisting of medium density residential areas near the outlet and low density residential areas in the upper portion. An assessment of the magnitude of the difference in land properties needed to produce a premature peak is needed.

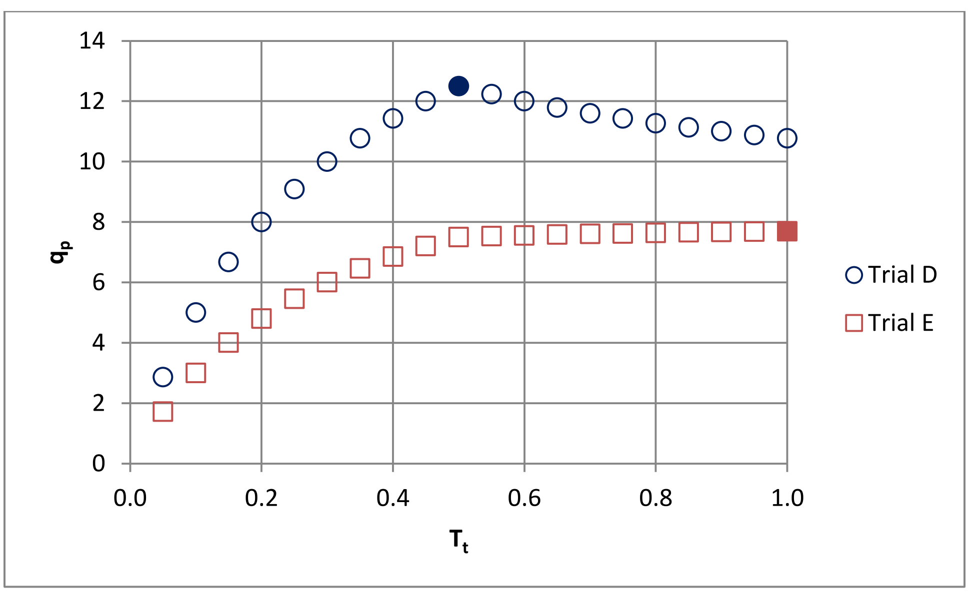


Figure 3.6 Comparison of Trials D and; Effect of ΔC on the Occurrence of a Premature Peak

Table 3.6 Trial D: Effect of ΔC, C Values of 0.5 and 0.2, $\Delta C = 0.3$

(a)

C	T_t
0.2	1.00
0.2	0.95
0.2	0.90
0.2	0.85
0.2	0.80
0.2	0.75
0.2	0.70
0.2	0.65
0.2	0.60
0.2	0.55
0.5	0.50
0.5	0.45
0.5	0.40
0.5	0.35
0.5	0.30
0.5	0.25
0.5	0.20
0.5	0.15
0.5	0.10
0.5	0.05

outlet

(b)

T_t	ΣA	C_{avg}	i	q
1.00	20	0.3500	1.5385	10.7692
0.95	19	0.3579	1.6000	10.8800
0.90	18	0.3667	1.6667	11.0000
0.85	17	0.3765	1.7391	11.1304
0.80	16	0.3875	1.8182	11.2727
0.75	15	0.4000	1.9048	11.4286
0.70	14	0.4143	2.0000	11.6000
0.65	13	0.4308	2.1053	11.7895
0.60	12	0.4500	2.2222	12.0000
0.55	11	0.4727	2.3529	12.2353
0.50	10	0.5000	2.5000	12.5000
0.45	9	0.5000	2.6667	12.0000
0.40	8	0.5000	2.8571	11.4286
0.35	7	0.5000	3.0769	10.7692
0.30	6	0.5000	3.3333	10.0000
0.25	5	0.5000	3.6364	9.0909
0.20	4	0.5000	4.0000	8.0000
0.15	3	0.5000	4.4444	6.6667
0.10	2	0.5000	5.0000	5.0000
0.05	1	0.5000	5.7143	2.8571

Table 3.7 Trial E: Effect of ΔC, C Values of 0.3 and 0.2, $\Delta C = 0.1$

(a)

C	T_t
0.2	1.00
0.2	0.95
0.2	0.90
0.2	0.85
0.2	0.80
0.2	0.75
0.2	0.70
0.2	0.65
0.2	0.60
0.2	0.55
0.3	0.50
0.3	0.45
0.3	0.40
0.3	0.35
0.3	0.30
0.3	0.25
0.3	0.20
0.3	0.15
0.3	0.10
0.3	0.05

outlet

(b)

T_t	ΣA	C_{avg}	i	q
1.00	20	0.2500	1.5385	7.6923
0.95	19	0.2526	1.6000	7.6800
0.90	18	0.2556	1.6667	7.6667
0.85	17	0.2588	1.7391	7.6522
0.80	16	0.2625	1.8182	7.6364
0.75	15	0.2667	1.9048	7.6190
0.70	14	0.2714	2.0000	7.6000
0.65	13	0.2769	2.1053	7.5789
0.60	12	0.2833	2.2222	7.5556
0.55	11	0.2909	2.3529	7.5294
0.50	10	0.3000	2.5000	7.5000
0.45	9	0.3000	2.6667	7.2000
0.40	8	0.3000	2.8571	6.8571
0.35	7	0.3000	3.0769	6.4615
0.30	6	0.3000	3.3333	6.0000
0.25	5	0.3000	3.6364	5.4545
0.20	4	0.3000	4.0000	4.8000
0.15	3	0.3000	4.4444	4.0000
0.10	2	0.3000	5.0000	3.0000
0.05	1	0.3000	5.7143	1.7143

3.2.2 Premature Peak Due to the Magnitude of the *C* Values

Analyses suggest that the occurrence of a premature peak are also dependent on the magnitude of *C*. Watersheds with the same ΔC may differ considerably in the occurrence of a premature peak. To test this hypothesis analyses were made using different values of *C* while holding ΔC constant. The arrangement of Trial F is shown in Table 3.7(a), where, *C* values of 0.5 were used for the lower 50% of the watershed and *C* values of 0.2 were used in the upper 50% of the watershed. The configuration of Trial G is shown in Table 3.9(a), where, *C* values of 0.9 were used for the lower 50% of the watershed and *C* values of 0.6 were used for the upper 50%. The results shown in Table 3.7(b) and Table 3.8(b) indicate that despite the ΔC values being the same, i.e., 0.3, the peak discharge rates do not occur at the same time. In the analysis of Trial F, a premature peak occurs at the location of the change in *C* values, at T_t = 0.45 hr. However, in the analysis of Trial G, a premature peak did not occur. Trials F and G support the hypothesis that the magnitudes of the *C* values impact the occurrence of a premature peak. Figure 3.7 shows the results of Trials F and G, providing a graphical comparison of the effect of the magnitude of the *C* values on the occurrence on a premature peak between these two trials. The point at which the peak discharge occurs is signified by the filled mark for each of the data series.

Table 3.8 Trial F: Effect of Magnitude of *C* Values, *C* Values of 0.5 and 0.2, Δ*C* = 0.3

(a)

C	T_t
0.2	1.00
0.2	0.95
0.2	0.90
0.2	0.85
0.2	0.80
0.2	0.75
0.2	0.70
0.2	0.65
0.2	0.60
0.2	0.55
0.5	0.50
0.5	0.45
0.5	0.40
0.5	0.35
0.5	0.30
0.5	0.25
0.5	0.20
0.5	0.15
0.5	0.10
0.5	0.05

outlet

(b)

T_t	ΣA	C_{avg}	i	q
1.00	20	0.3500	1.5385	10.7692
0.95	19	0.3579	1.6000	10.8800
0.90	18	0.3667	1.6667	11.0000
0.85	17	0.3765	1.7391	11.1304
0.80	16	0.3875	1.8182	11.2727
0.75	15	0.4000	1.9048	11.4286
0.70	14	0.4143	2.0000	11.6000
0.65	13	0.4308	2.1053	11.7895
0.60	12	0.4500	2.2222	12.0000
0.55	11	0.4727	2.3529	12.2353
0.50	10	0.5000	2.5000	12.5000
0.45	9	0.5000	2.6667	12.0000
0.40	8	0.5000	2.8571	11.4286
0.35	7	0.5000	3.0769	10.7692
0.30	6	0.5000	3.3333	10.0000
0.25	5	0.5000	3.6364	9.0909
0.20	4	0.5000	4.0000	8.0000
0.15	3	0.5000	4.4444	6.6667
0.10	2	0.5000	5.0000	5.0000
0.05	1	0.5000	5.7143	2.8571

Table 3.9 Trial G: Effect of Magnitude of *C* Values, *C* Values of 0.9 and 0.6, Δ*C* = 0.3

(a)

C	T_t
0.6	1.00
0.6	0.95
0.6	0.90
0.6	0.85
0.6	0.80
0.6	0.75
0.6	0.70
0.6	0.65
0.6	0.60
0.6	0.55
0.9	0.50
0.9	0.45
0.9	0.40
0.9	0.35
0.9	0.30
0.9	0.25
0.9	0.20
0.9	0.15
0.9	0.10
0.9	0.05

outlet

(b)

T_t	ΣA	C_{avg}	i	q
1.00	20	0.7500	1.5385	23.0769
0.95	19	0.7579	1.6000	23.0400
0.90	18	0.7667	1.6667	23.0000
0.85	17	0.7765	1.7391	22.9565
0.80	16	0.7875	1.8182	22.9091
0.75	15	0.8000	1.9048	22.8571
0.70	14	0.8143	2.0000	22.8000
0.65	13	0.8308	2.1053	22.7368
0.60	12	0.8500	2.2222	22.6667
0.55	11	0.8727	2.3529	22.5882
0.50	10	0.9000	2.5000	22.5000
0.45	9	0.9000	2.6667	21.6000
0.40	8	0.9000	2.8571	20.5714
0.35	7	0.9000	3.0769	19.3846
0.30	6	0.9000	3.3333	18.0000
0.25	5	0.9000	3.6364	16.3636
0.20	4	0.9000	4.0000	14.4000
0.15	3	0.9000	4.4444	12.0000
0.10	2	0.9000	5.0000	9.0000
0.05	1	0.9000	5.7143	5.1429

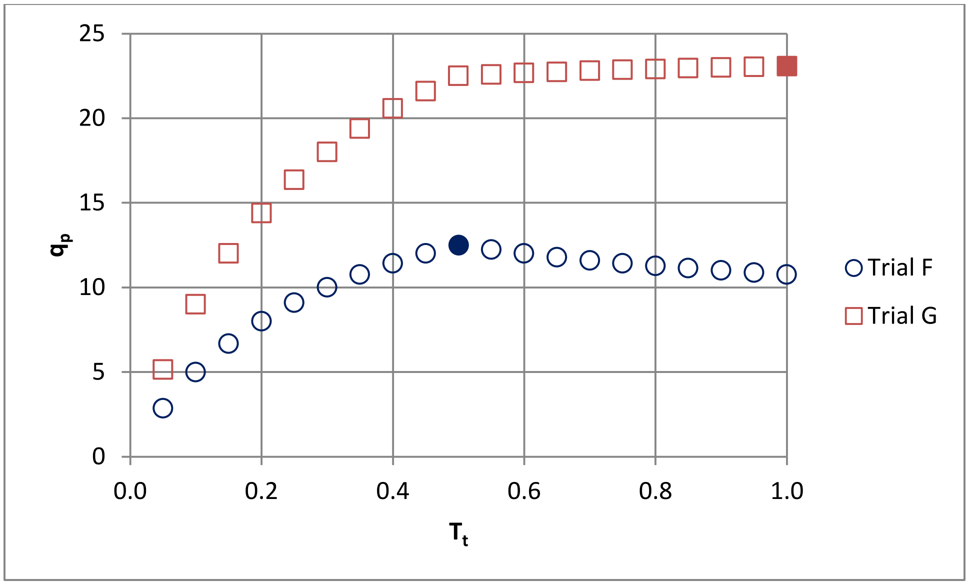


Figure 3.7 Comparison of Trials F and G, Effect of Magnitude of *C* on Occurrence of a Premature Peak

3.2.3 Premature Peak Based on the Contributing Area of the *C* Values

Analyses indicated that the spatial distribution of the *C* values influenced the occurrence of a premature peak. The degree of heterogeneity, as indicated by the variation in values of *C* determines if a premature peak will occur. For example, a watershed that is mostly urban with only a small section of forested area at the upper end would likely experience different discharges than a watershed that with a greater portion of forested area.

This concept is shown in comparing Trials H and I, shown in Table 3.9 and Table 3.10, respectively. The configuration of Trial H is shown in Table 3.9(a), where the lower 70% of the watershed has a *C* value of 0.9 and the upper 30% has a *C* value of 0.6. In Trial H, a premature peak of 25.2 cfs occurred at the travel time $T_t = 0.70$. The location of the premature peak corresponds to the place in the watershed where the *C* value changes from the downslope *C* value of 0.9 to the *C* value of 0.6 in the upper part of the watershed.

Trial I, shown in Table 3.11(a), displays a trial with the same ΔC and magnitude of C values as Trial H, but with different weighting, where the C value of 0.9 comprises 55% and the C value of 0.6 is 45% of the watershed. The peak discharge for the entire watershed was 23.5 cfs, which resulted when all of the cells were contributing. A premature peak does not result from the change in weighting because the weight of the C value in the lower part of the watershed is not sufficient to dominate.

Figure 3.8 shows the results of Trials H and I, providing a graphical comparison of the effect of the weighting of the C values on the occurrence on a premature peak between these two trials. The point at which the peak discharge occurs is signified by the filled mark for each of the data series. Trials H and I have the same discharge rates until T_t = 0.60, which corresponds to the point where the lower C value of 0.9 changes to the upper C value of 0.6 in Trial I.

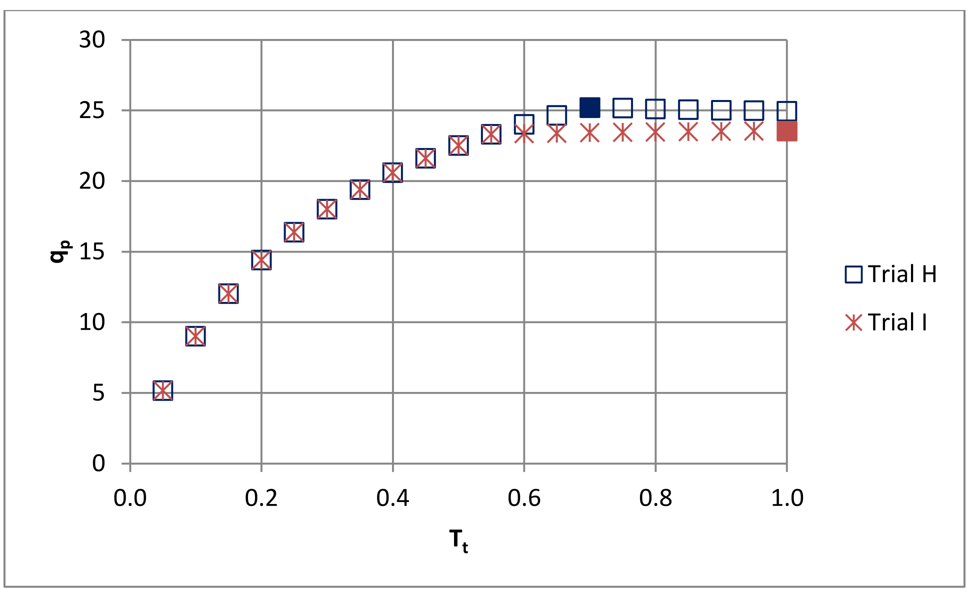


Figure 3.8 Comparison of Trials H and I, Effect of Weighting on Occurrence of Premature Peak

Table 3.10 Trial H: Effect of Weighting, *C* Values of 0.9 (70%) and 0.6 (30%)

(a)

C	T_t
0.6	1.00
0.6	0.95
0.6	0.90
0.6	0.85
0.6	0.80
0.6	0.75
0.9	0.70
0.9	0.65
0.9	0.60
0.9	0.55
0.9	0.50
0.9	0.45
0.9	0.40
0.9	0.35
0.9	0.30
0.9	0.25
0.9	0.20
0.9	0.15
0.9	0.10
0.9	0.05

outlet

(b)

T_t	ΣA	C_{avg}	i	q
1.00	20	0.8100	1.5385	24.9231
0.95	19	0.8211	1.6000	24.9600
0.90	18	0.8333	1.6667	25.0000
0.85	17	0.8471	1.7391	25.0435
0.80	16	0.8625	1.8182	25.0909
0.75	15	0.8800	1.9048	25.1429
0.70	14	0.9000	2.0000	25.2000
0.65	13	0.9000	2.1053	24.6316
0.60	12	0.9000	2.2222	24.0000
0.55	11	0.9000	2.3529	23.2941
0.50	10	0.9000	2.5000	22.5000
0.45	9	0.9000	2.6667	21.6000
0.40	8	0.9000	2.8571	20.5714
0.35	7	0.9000	3.0769	19.3846
0.30	6	0.9000	3.3333	18.0000
0.25	5	0.9000	3.6364	16.3636
0.20	4	0.9000	4.0000	14.4000
0.15	3	0.9000	4.4444	12.0000
0.10	2	0.9000	5.0000	9.0000
0.05	1	0.9000	5.7143	5.1429

Table 3.11 Trial I: Effect of Weighting, C Values of 0.9 (55%) and 0.6 (45%)

(a)

C	T_t
0.6	1.00
0.6	0.95
0.6	0.90
0.6	0.85
0.6	0.80
0.6	0.75
0.6	0.70
0.6	0.65
0.6	0.60
0.9	0.55
0.9	0.50
0.9	0.45
0.9	0.40
0.9	0.35
0.9	0.30
0.9	0.25
0.9	0.20
0.9	0.15
0.9	0.10
0.9	0.05

outlet

(b)

T_t	ΣA	C_{avg}	i	q
1.00	20	0.7650	1.5385	23.5385
0.95	19	0.7737	1.6000	23.5200
0.90	18	0.7833	1.6667	23.5000
0.85	17	0.7941	1.7391	23.4783
0.80	16	0.8063	1.8182	23.4545
0.75	15	0.8200	1.9048	23.4286
0.70	14	0.8357	2.0000	23.4000
0.65	13	0.8538	2.1053	23.3684
0.60	12	0.8750	2.2222	23.3333
0.55	11	0.9000	2.3529	23.2941
0.50	10	0.9000	2.5000	22.5000
0.45	9	0.9000	2.6667	21.6000
0.40	8	0.9000	2.8571	20.5714
0.35	7	0.9000	3.0769	19.3846
0.30	6	0.9000	3.3333	18.0000
0.25	5	0.9000	3.6364	16.3636
0.20	4	0.9000	4.0000	14.4000
0.15	3	0.9000	4.4444	12.0000
0.10	2	0.9000	5.0000	9.0000
0.05	1	0.9000	5.7143	5.1429

CHAPTER 4

VERIFYING GUIDELINES

4.1 VERIFYING GUIDELINES FOR PREMATURE PEAKS

In Section 3.2, the general guidelines for the occurrence of a premature peak were presented based on trials of simplified watersheds. However, realistically, different watersheds exist in numerous shapes and sizes. Incorporating these watershed characteristics into the analytical model is important to better understand the intricacies of a microwatershed. A computer program, described further in Section 3.3.1, was developed that accounts for the spatial configuration of a watershed. The program utilizes inputs of slope, curve number, and the flow identification numbers that describe the principal flow paths of each watershed cell; these are necessary to calculate the peak discharge. These parameters assist in providing more information about the watershed, which ultimately should lead to a more realistic computation of peak discharge rates and the detection of the condition under which a premature peak discharge occurs.

The general guidelines from the previous linear trials were then applied to the computerized watersheds to determine whether or not the guidelines were still accurate for more realistic systems. The guidelines tested involved the occurrence of a premature peak based on:

(1) The difference between the *C* values in the lower and upper portions of the watershed, ΔC (as described in Section 3.2.1). This was tested by gradually changing ΔC and observing the effect on the occurrence of a premature peak.

(2) The magnitude of the *C* values at the upper and lower reaches of the watershed (as described in Section 3.2.2). This was tested by holding ΔC constant between trials, but systematically changing the *C* values in the watershed.

(3) The contributing area of the *C* values (as described in Section 3.2.3). This was tested by holding the C value of the upper reaches and the C value of the lower reaches constant across trials while gradually increasing the proportion of area that is included with the C value of the lower reaches.

Along with testing these former general guidelines, the effect of the additional watershed parameters (slope, time of concentration, and watershed shape and size) may be analyzed to gain a more comprehensive understanding of the occurrence of a premature peak in microwatersheds.

4.2 DESCRIPTION OF PROGRAM INPUTS AND OUTPUTS

The exploratory trials discussed earlier in Sections 3.1 and 3.2 were used to assess general trends in the relationship between the occurrence of a premature peak and the distribution of time and *C* values. However, a more realistic model was needed to show the effect of the spatial configuration of the watershed on the occurrence of a premature peak. A computer program was created that models the watershed as a <*m x n*> matrix, depending on the size and shape of the watershed. An example of fitting the watershed shape to a <4 *x* 3> matrix is shown in Figure 4.1, where the oval indicates the actual boundary and the bolded lines represent the boundaries of the computerized watershed. The lack of agreement between the true watershed shape and the computerized approximation should not negate the validity of the results, as the cells can be made small enough to closely match the watershed boundary.

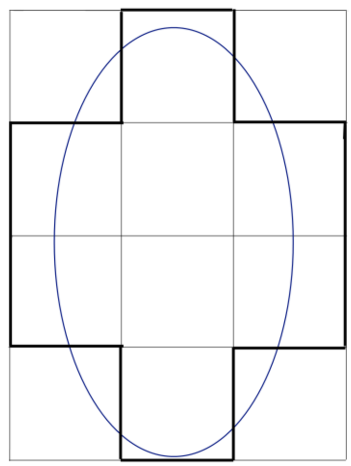


Figure 4.1 Example of Using the Shape of the Watershed to Define the Program Matrix

The computerized model divides the watershed into a series of equal-area cells. The land characteristics of each cell within the watershed are described through the runoff coefficient, the slope, and flow identification numbers, which are all inputs into the program as elements of the <*m x n*> matrix. The *C* values reflect the type of land use within each cell, and the *S* values reflect the steepness of the terrain. The *C*, *S*, and flow identification numbers are then used to calculate the travel time for each cell.

The program calculates the travel time within each cell using the SCS Lag method discussed in Section 2.4. Since the SCS Lag method requires curve numbers, a relationship between the inputted runoff coefficients and the curve numbers had to be established. The methodology for building this relationship was discussed in Section 4.3.2. The travel time through a cell was then calculated using Equation 2-4, where, the length of the cell is an input parameter and the same for each cell throughout the watershed.

The flow path of runoff through the watershed is an important factor in determining the total travel time from any one cell to the watershed outlet. The flow path was represented within the program by the flow identification (ID) number. An ID number (1, 2, …, 8) was input for each cell based on the direction in which runoff would drain from the cell, as shown in Figure 4.2, where, a value of 1 runoff flows northward and a 3 indicates flow to the east. Figure 4.3(a) depicts an example of the flow path of water through a simplified watershed, and Figure 4.3(b) demonstrates how the ID numbers were correspondingly assigned.

8	1	2
7	**CELL**	3
6	5	4

Figure 4.2 Assignment of Flow Identification Numbers (ID)

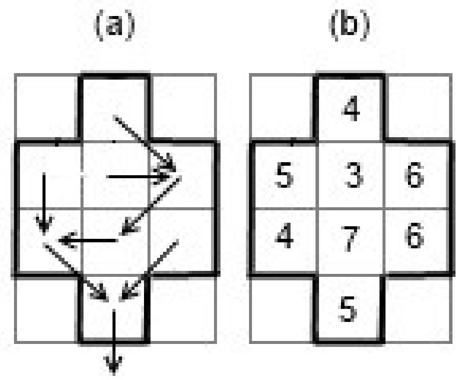


Figure 4.3 Example of the Assignment of Flow Identification Numbers (ID)

The cumulative travel times were calculated by summing the travel times from the cell in question along each cell of the flow path to the watershed outlet. The cumulative travel times show spatially and temporally the parts of the watershed that take the longest time to reach the outlet. Once the travel times for all of the cells were computed, a time-area diagram was generated. The program divided the longest cumulative travel time into time intervals and determined the number of cells that had travel times shorter than a fraction of the total travel time. The number of cells that fell within each time-to-outlet increment was cumulatively added. The mean *C*, which is denoted as $\bar{C}$, was computed for each time-to-outlet increment as the average *C* value for all of the cells that are contained in that increment. The rainfall intensity was calculated as a function of the time-to-outlet increment and the return period. The area that was associated with a time-to-outlet interval was the summation of the area of the cells with times-to-outlet values less than the specific interval. The discharge for each increment was then calculated using the Rational method,

$$q_t = \overline{C}_t \, i_t \, A_t \tag{4-1}$$

in which $\overline{C_t}$ is the average C value for all cells with cumulative travel times less than t, i_t is the intensity using time-to-outlet t as the duration, and A_t is the total area of all cells with times-to-outlet less than cumulative travel time t. Figure 4.4 shows the schematic of a 24-cell watershed.

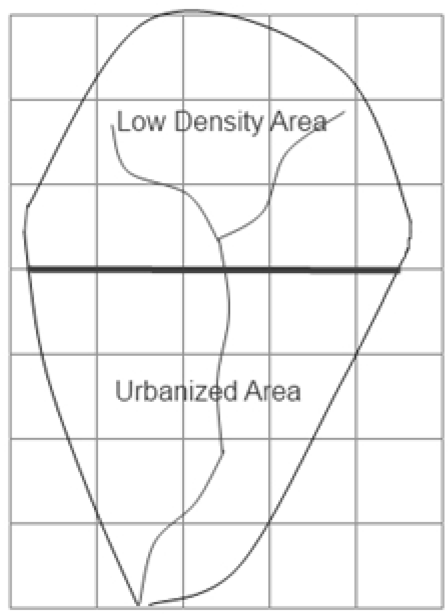


Figure 4.4 Schematic of a 24-cell Watershed

The tabular output table from the computer program displays the number of cells that are contained in each corresponding time-to-outlet interval. In addition, the $\overline{C_t}$, intensity, and total cumulative area for that point in the watershed are also displayed. The last row of the table corresponds to the inclusion of all cells, which would be equal to the inclusion of the entire watershed. A premature peak would be indicated by a maximum discharge that occurs prior to the last row. An example of the results is shown in Table 4.1. In the example displayed, a premature peak occurs when only 11 of the 12 cells are included.

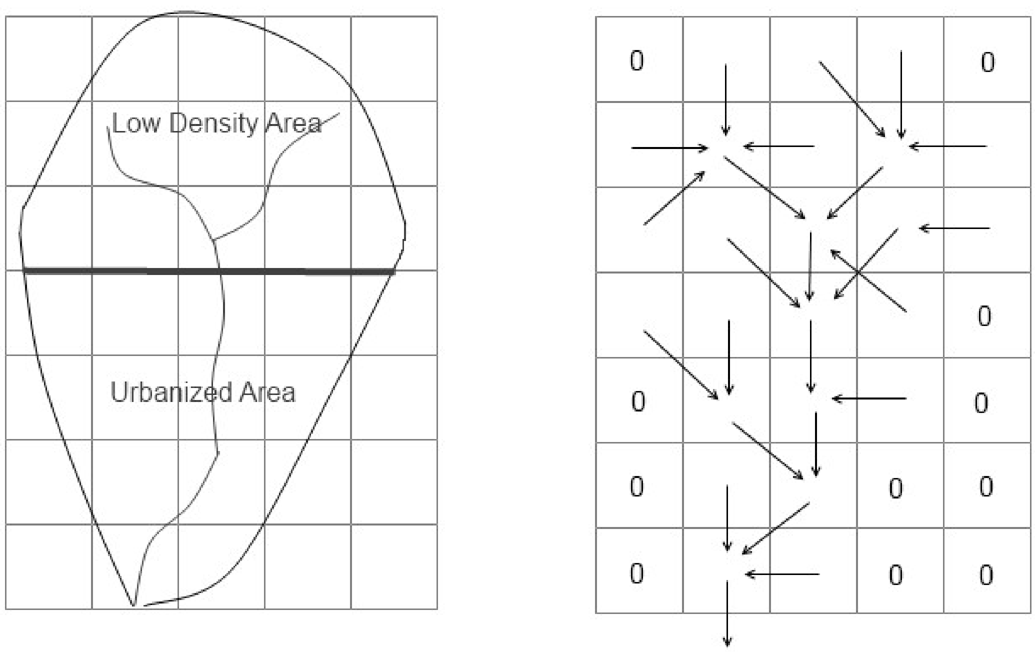
Table 4.1 Example of the Output from the Program

Number of cells	Time to outlet (hr)	Mean C	Rainfall intensity (in./hr)	Total area (acres)	Discharge (ft^3/s)
1	0.0074	0.7000	9.4009	0.9183	6.0428
3	0.0148	0.6000	9.2011	2.7548	15.2085
4	0.0222	0.6000	9.0096	3.6731	19.8560
5	0.0296	0.5400	8.8260	4.5914	21.8826
7	0.0591	0.4143	8.1605	6.4279	21.7313
9	0.0813	0.3556	7.7237	8.2645	22.6960
10	0.1035	0.3300	7.3313	9.1827	22.2161
11	0.1109	0.3182	7.2092	10.1010	23.1702
12	0.1478	0.3083	6.6551	11.0193	22.6115

4.2.1 Physical Meaning of the Flow Identification Numbers

The flow identification (ID) numbers define the flow path of the runoff through the watershed. Different characteristics of the terrain will determine how the runoff will move towards the outlet of the watershed. For a natural watershed, topography is the dominant factor, with the flow direction generally following the steepest slope. When properties are developed, flow directions can be modified, such as flow directed to gutters and storm sewers. Channels can be straightened and slopes changed through cut-and-fill activities. In the computer model, those characteristics are the runoff coefficients, and the slopes. For an urbanized area, it would be expected that the runoff path would be more linear as it flows through the designed stormwater management systems that are implemented in high density areas. Undeveloped areas would be expected to have paths that are more meandering and follow natural channels to the outlet.

The watershed shown in Figure 4.5(a) is one that has low density area in the upper reaches and urbanized area in the lower reaches. This arrangement of land cover is an example of a watershed that is nonhomogeneous. Figure 4.5(b) is the corresponding runoff route through the watershed, where each of the cells would be assigned an ID number to be inputted into the computer program. The zeros represent area that lies outside of the watershed boundary, and they are marked so that the computer can identify the general shape of the watershed. The travel time through each individual cell is calculated. The cumulative travel time is computed using the flow ID numbers to determine the route of the discharge through the watershed.



(a) (b)

Figure 4.5 Example of a Flow Path for a Low Density and Urbanized Area

4.2.2 Estimation of Curve Numbers Using Runoff Coefficients

The computer program utilizes the SCS Lag Method to compute the time of concentration for each cell. Required inputs for the program include runoff coefficients and slopes as a percent for each cell. Therefore, a relationship between the inputted runoff coefficients and the curve numbers necessary for the SCS Lag Method needed to be established.

The estimation of the curve numbers based off of the runoff coefficients was based on previous work performed by McCuen and Bondelid (1981), which was discussed in Section 2.4.2. Their work involved the estimation of runoff coefficients from curve numbers, and the results were displayed graphically (see Figure 4.6).

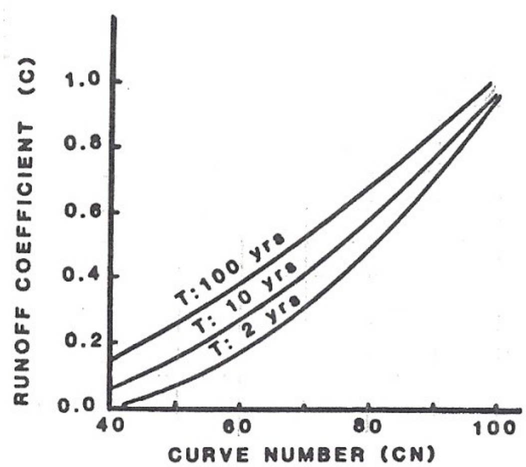


Figure 4.6 Relationship between Runoff Coefficient and Curve Number for Baltimore, MD (McCuen and Bondelid, 1981)

Using the graphical relationship of Figure 4.6 curve numbers were estimated as a function of the runoff coefficients for intervals of 0.1 over the range 0.0 to 1.0. This was done for the return periods of 2, 10 and 100 years. The results were then regressed to form a relationship that would yield a curve number (*CN*) given a runoff coefficient (*C*):

$$CN = b_1 + b_2 C^{b_3} e^{b_4 C} \tag{4-2}$$

where b_1, b_2, b_3, and b_4 are coefficients that depend on the return year. Those coefficients are shown in Table 4.2 for the return periods of 2, 5, 10, 25, 50, and 100 years.

Table 4.2 Coefficients for Estimating Curve Numbers Based on Runoff Coefficients

T	b1	b2	b3	b4
2	33.6	89.9	0.67	-0.32
5	30.0	94.0	0.72	-0.32
10	26.8	98.2	0.77	-0.32
25	23.0	100.0	0.79	-0.31
50	20.0	103.0	0.80	-0.28
100	15.9	106.5	0.81	-0.27

4.3 VERIFYING THE GENERAL GUIDELINES: WATERSHED SETUP

An example watershed was created to provide the basic characteristics necessary to use the program. The watershed consisted of 7 rows and 5 columns that were assigned flow ID numbers to represent how the runoff might flow through the system. The flow ID numbers represent the flow direction, as shown in Figure 4.3, where the cells with a zero value lay outside of the watershed boundary. While the total matrix consists of 35 cells, 24 cells are within the watershed boundary. These are referred to as active cells.

0	5	4	5	0
3	4	7	6	7
2	4	5	6	7
4	5	5	8	0
0	4	5	7	0
0	5	6	0	0
0	0	7	0	0

Figure 4.7 Flow ID Arrangement Used for Example Watershed

The program inputs required that *C* and slope values be provided for each cell within the watershed boundary. The *C* and slope inputs were varied to test the general guidelines of how the identified variables influenced the occurrence of a premature peak. The program requires the length of each cell, the return period, and the desired time increment. For now, the length and width of each cell was set as 200 ft and a return period as 2 years was used for calculating the rainfall intensity.

The configurations of the 24-cell trials that are discussed in Chapter 4 are shown in Table 4.3. These trials were used to demonstrate how the guidelines for the occurrence of a premature peak were verified. The initial guidelines were based on the 20-cell trials.

Table 4.3 Arrangements of the 24-cell Trials Discussed in Chapter 4

Trial	CL	ΔC	SL	ΔS	Area CL (%)	Premature Peak?	Comment
J	0.4	0.2	2	0	45.8	No	Effect of ΔC; compared to Trial K and L
K	0.5	0.3	2	0	45.8	Yes	Effect of ΔC; compared to Trial J and L
L	0.6	0.4	2	0	45.8	Yes	Effect of ΔC; compared to Trials L and K Effect of magnitude of C values; compared to Trial M
M	0.8	0.4	2	0	45.8	No	Effect of magnitude of C values; compared to Trial L
N	0.9	0.5	2	0	70.8	No	Effect of area of lower portion; compared to Trial O
O	0.9	0.5	2	0	75	Yes	Effect of area of lower portion; compared to Trial N

4.3.1 General Guideline: Difference between *C* Values

Analyses from the linear watershed trials indicated that a premature peak was more likely to occur under the conditions when ΔC between the upper and lower reaches of the watershed was large. To assess the validity of this general observation in watersheds with more realistic layouts, only the C values were altered between trials. The ID numbers, cell size, and return period were kept constant across the trials to show the effect of changing ΔC. For this analysis, the slope was also kept constant across the watershed, at a value of 2%.

The first trial, Trial J, was organized such that the cells in the upper reaches of the watershed had C values of 0.2 and the cells in the lower reaches had C values of 0.4, which would give $\Delta C = 0.2$. The lower reaches consisted of 11 of the 24 cells within the watershed boundary, or 45.8%, and the upper reaches consisted of 54.2% of the watershed. The arrangement of the *C* values for Trial J is shown in Figure 4.8.

0	.2	.2	.2	0
.2	.2	.2	.2	.2
.2	.2	.2	.2	.2
.4	.4	.4	.4	0
0	.4	.4	.4	0
0	.4	.4	0	0
0	.4	.4	0	0

Figure 4.8 Trial J: Arrangement of C Values

The comparison trial, Trial K, was different from Trial J in that the cells in the lower reaches had a *C* value of 0.5 but those in the upper part remained at a *C* value of 0.2, which resulted in $\Delta C = 0.3$. The arrangement of the *C* values for Trial K is shown in Figure 4.9. The larger difference in *C* values across the watershed would represent an area that is more heterogeneous.

0	.2	.2	.2	0
.2	.2	.2	.2	.2
.2	.2	.2	.2	.2
.5	.5	.5	.5	0
0	.5	.5	.5	0
0	.5	.5	0	0
0	.5	.5	0	0

Figure 4.9 Trial K: Arrangement of C Values

The results of Trial J (see Table 4.4) show that the peak does not occur prematurely under the conditions presented with $\Delta C = 0.2$. The peak discharge of 9.0 cfs occurs when the entire watershed is contributing. The time to the outlet is 1.0 hr and the corresponding rainfall intensity is 1.4 in./hr. The results of Trial K (see Table 4.5) show that changing the ΔC to 0.3, while all other inputs remained the same as in Trial J, cause a premature peak to occur with only 10 of the 24 contributing cells. The peak discharge of 11.6 cfs occurs over only 9.2 acres of the total 22.0 acres. The discharge ratio for Trial K was 1.1, which serves as an indicator of how the premature peak discharge relates to the entire watershed discharge. Figure 4.10 shows a graphical comparison of the discharge versus time to outlet for Trials J and K. The filled node for each series on the graph represents the location of the peak discharge. Cubic polynomials were fitted to both trials to provide a continuous relationship for travel time and total discharge, as previously discussed in Section 3.1.2. The resulting cubic relationships for Trials J and K were, respectively,

$$Q = 6.1792T_t^3 - 24.887T_t^2 + 28.566T_t - 1.8886 \tag{4-3}$$

$$Q = 19.877T_t^3 - 58.265T_t^2 + 51.087T_t - 3.3431 \tag{4-4}$$

The R^2 values for the cubic polynomials fitted to Trials J and K were 0.93 and 0.92. These R^2 values indicate that the cubic function fits the total discharge versus travel time data well for both trials.

Table 4.4 Trial J: Effect of Δ*C*, *C* Values of 0.4 and 0.2, Δ*C* = 0.2

Number of cells	Time to outlet (hr)	Mean C	Rainfall intensity (in./hr)	Total area (acres)	Discharge (ft^3/s)
1	0.1208	0.4000	4.1426	0.9183	1.5216
4	0.2416	0.4000	3.2716	3.6731	4.8068
6	0.3625	0.4000	2.7033	5.5096	5.9576
10	0.4833	0.4000	2.3031	9.1827	8.4596
13	0.6444	0.3538	1.9235	11.9376	8.1251
14	0.7652	0.3571	1.7119	12.8558	7.8600
17	0.8055	0.3294	1.6514	15.6107	8.4918
24	1.0068	0.2917	1.4032	22.0386	9.0194

Table 4.5 Trial K: Effect of ΔC, C Values of 0.5 and 0.2, $\Delta C = 0.3$

Number of cells	Time to outlet (hr)	Mean C	Rainfall intensity (in./hr)	Total area (acres)	Discharge (ft^3/s)
1	0.1121	0.5000	4.2237	0.9183	1.9393
4	0.2242	0.5000	3.3740	3.6731	6.1965
6	0.2989	0.5000	2.9750	5.5096	8.1955
10	0.4110	0.5000	2.5267	9.1827	11.6011
13	0.5605	0.4308	2.1040	11.9376	10.8197
14	0.6726	0.4357	1.8695	12.8558	10.4719
17	0.7474	0.3941	1.7402	15.6107	10.7062
24	0.9342	0.3375	1.4836	22.0386	11.0350

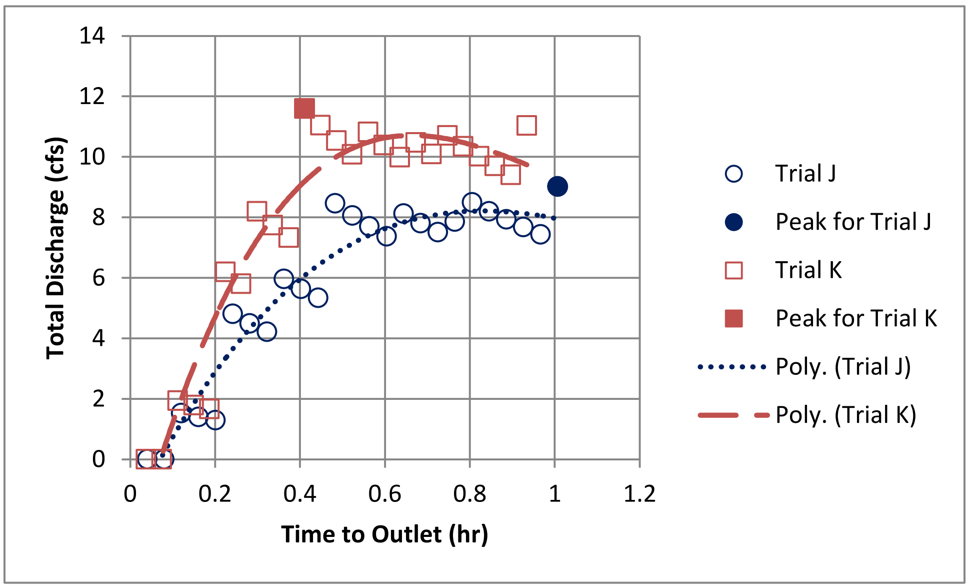


Figure 4.10 Comparison of Trials J and K Fitted with Cubic Polynomials

A larger ΔC would represent a watershed where the lower and upper reaches had significantly different land cover characteristics. In the linear trials, as ΔC became larger a premature peak was more likely to occur and also had a greater difference from the traditional peak discharge of the entire watershed. Trial L was setup similar to Trials J and K, where the upper portion of the watershed had a C value of 0.2; however, Trial L had a C value in the lower portion of 0.6, which resulted in a $\Delta C = 0.4$ (see Figure 4.11).

0	.2	.2	.2	0
.2	.2	.2	.2	.2
.2	.2	.2	.2	.2
.6	.6	.6	.6	0
0	.6	.6	.6	0
0	.6	.6	0	0
0	.6	.6	0	0

Figure 4.11 Trial L: Arrangement of *C* Values

The results of Trial L (see Table 4.6) showed that the premature peak of 15.2 cfs occurred when 10 of the 24 active cells were contributing. Comparing the premature peak discharge to the entire watershed discharge of 13.1 cfs yields a discharge ratio of 1.2. The discharge ratio of 1.2 for Trial L is greater than the discharge ratio of 1.1 for Trial K. This occurs because the ΔC of Trial L was intentionally made larger than in Trial K, causing a larger difference between the magnitudes of the discharge at the two points in the watershed. The larger ΔC indicates a more heterogeneous watershed that is more conducive to the occurrence of a premature peak. Figure 4.12 displays a graphical comparison of the total discharge versus the time to outlet for Trials J, K, and L. Fitting a cubic polynomial to Trial L yields the following relationship with an R^2 value of 0.92,

$$Q = 57.047T_t^{\ 3} - 123.85T_t^{\ 2} + 83.768T_t - 4.7076 \quad (4\text{-}5)$$

The relatively high coefficient of determination indicates that the cubic function provides a good fit and an acceptable continuous relationship for travel time and discharge.

Table 4.6 Trial L: Effect of Δ*C*, *C* Values of 0.6 and 0.2, Δ*C* = 0.4

Number of cells	Time to outlet (hr)	Mean C	Rainfall intensity (in./hr)	Total area (acres)	Discharge (ft^3/s)
1	0.1051	0.6000	4.2916	0.9183	2.3645
4	0.1751	0.6000	3.6999	3.6731	8.1541
6	0.2802	0.6000	3.0660	5.5096	10.1354
10	0.3502	0.6000	2.7516	9.1827	15.1605
13	0.5253	0.5077	2.1903	11.9376	13.2743
14	0.5954	0.5143	2.0250	12.8558	13.3884
17	0.7005	0.4588	1.8191	15.6107	13.0296
24	0.8756	0.3833	1.5555	22.0386	13.1414

Trials J, K, and L support the general guideline that Δ*C* has an effect on the occurrence of a premature peak. Both of these experiments show that the larger the difference in the C values, the more likely a premature peak will occur. These trials show that the general guideline applies to the more spatially realistic watershed shapes as well as the linear ones previously used. It is also evident when comparing Figure 4.10 and Figure 4.12 that the curvature of the cubic function increases as the Δ*C* increases. This suggests that the greater difference in premature peak occurs for the greater nonhomogeneity of land cover.

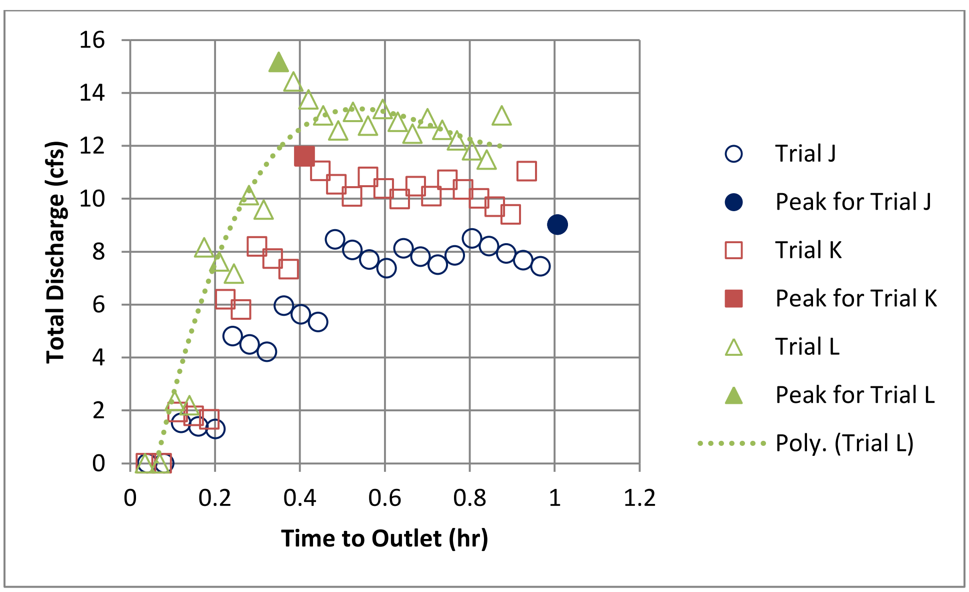


Figure 4.12 Comparing Trials J, K, and L

4.3.2 General Guideline: Magnitude of the *C* Values

The linear watershed trials of Chapter 3 indicated that despite two different watersheds having the same ΔC, the occurrence of a premature peak was influenced by the magnitude of the *C* values. In the linear trials, this guideline was supported by showing that, if the *C* values were altered from trial-to-trial, while keeping ΔC the same that *C* values of lower magnitude were more likely to experience a premature peak. Applying this trend to the more spatially realistic trials will indicate whether it remains a reasonable guideline to make about the occurrence of a premature peak discharge.

Sample trials were developed to test if the guideline held true for realistic watershed shapes. In the previously discussed Trial L (see Figure 4.13a), a premature peak resulted from a $\Delta C = 0.4$ with a corresponding discharge ratio of 1.2. Trial L was arranged such that the lower 45.8% of the watershed near the outlet had a *C* value of 0.6 and the upper 54.2% of the watershed had a C value of 0.2. From the linear trials, it would be expected that as the magnitude of the *C* values in both the lower and upper reaches of the watershed increased while ΔC remained constant that a premature peak would be less likely to occur.

The *C* values of Trial M (see Figure 4.13b) were arranged such that ΔC would equal 0.4, as in Trial L, but the *C* value in the lower portion was equal to 0.8 and the *C* value in the upper portion was equal to 0.4 The configuration of *C* values in Trial M would represent a watershed that is overall more highly developed than that of Trial L, but with the same amount of difference between the *C* values of the lower and upper portions of the watershed.

0	.2	.2	.2	0
.2	.2	.2	.2	.2
.2	.2	.2	.2	.2
.6	.6	.6	.6	0
0	.6	.6	.6	0
0	.6	.6	0	0
0	.6	.6	0	0

(a) Trial L

0	.4	.4	.4	0
.4	.4	.4	.4	.4
.4	.4	.4	.4	.4
.8	.8	.8	.8	0
0	.8	.8	.8	0
0	.8	.8	0	0
0	.8	.8	0	0

(b) Trial M

Figure 4.13 Trial M: Arrangement of *C* Values as Compared to Trial L

The configuration of *C* values for Trial M (see Table 4.6) resulted in a peak discharge of 25.4 cfs that occurred when the entire watershed contributed. The magnitude of the discharge is greater throughout each time increment of Trial M, as compared to Trial L, because the *C* values are those of a more urbanized watershed. Figure 4.14 displays the total discharge versus time to outlet for both trials. Despite the ΔC being equal to 0.4 for both trials, the occurrence of a premature peak in Trial L and the absence of a premature peak in Trial M indicate that the magnitude of the *C* values are also a factor in the occurrence of a premature peak. The comparison of Trials L and M indicate that the general guideline from the linear trials may also be applied to more spatially realistic watershed models.

Table 4.7 Trial M: Effect of Magnitude of *C*, *C* Values of 0.8 and 0.4, $\Delta C = 0.4$

Number of cells	Time to outlet (hr)	Mean C	Rainfall intensity (in./hr)	Total area (acres)	Discharge (ft^3/s)
1	0.0742	0.8000	4.6168	0.9183	3.3916
4	0.1484	0.8000	3.9051	3.6731	11.4751
6	0.1979	0.8000	3.5412	5.5096	15.6086
10	0.2721	0.8000	3.1069	9.1827	22.8239
13	0.3710	0.7077	2.6703	11.9376	22.5587
14	0.4453	0.7143	2.4156	12.8558	22.1823
17	0.4947	0.6588	2.2713	15.6107	23.3592
24	0.6184	0.5833	1.9760	22.0386	25.4032

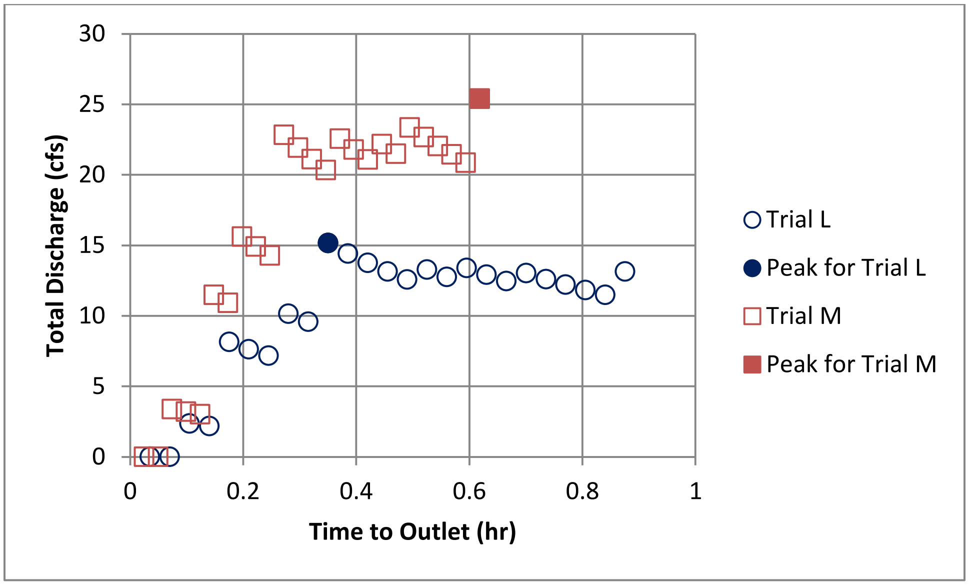


Figure 4.14 Comparing Discharge vs. Time to Outlet for Trials L and M

4.3.3 General Guideline: Contributing Area of the Lower *C* Value

Analyses of the linear trials in Chapter 3 indicated that the size of the contributing area of the *C* values influenced the occurrence of a premature peak. The analyses indicated that, if the lower portion of the watershed consisted of sufficient area, then a premature peak could occur. This guideline was analyzed in the more spatially realistic watershed by systematically increasing the contributing area of the lower *C* value until a premature peak occurred.

The example trials for this guideline were arranged with the lower *C* value equal to 0.9 and the upper *C* value equal to 0.4, which yields a $\Delta C = 0.5$. The contributing area of the lower *C* value was increased systematically until a premature peak occurred. The arrangement of Trial N (see Figure 4.15) consists of a lower portion that is 17 of the 24 active cells (i.e., 70.8%) and an upper portion that is 7 of the 24 active cells. Trial N did not result in a premature peak, as shown in Table 4.8. Trial N experienced a peak discharge of 36.7 cfs with all 24 of the active cells contributing, which would yield a discharge ratio of 1.0. In addition, trials with less contributing area of the lower *C* value than Trial N also did not result in a premature peak.

0	.4	.4	.4	0
.9	.4	.4	.4	.4
.9	.9	.9	.9	.9
.9	.9	.9	.9	0
0	.9	.9	.9	0
0	.9	.9	0	0
0	.9	.9	0	0

Figure 4.15 Trial N: Arrangement of *C* Values, Lower Portion = 70.8% of the Watershed

Table 4.8 Trial N: Effect of Contributing Area of Lower *C*, Lower Portion = 70.8%

Number of cells	Time to outlet (hr)	Mean C	Rainfall intensity (in./hr)	Total area (acres)	Discharge (ft^3/s)
1	0.0623	0.9000	4.7559	0.9183	3.9305
4	0.1246	0.9000	4.1085	3.6731	13.5817
6	0.1661	0.9000	3.7666	5.5096	18.6774
10	0.2284	0.9000	3.3487	9.1827	27.6748
13	0.2700	0.9000	3.1180	11.9376	33.4991
15	0.3322	0.9000	2.826	13.7741	35.0332
17	0.3945	0.8412	2.5840	15.6107	33.9318
19	0.4568	0.8474	2.3802	17.4472	35.1898
24	0.5191	0.7542	2.2062	22.0386	36.6690

The contributing area of the lower *C* value was then systematically increased from Trial N by changing one of the *C* values of 0.4 to a *C* value of 0.9. The arrangement of Trial O is displayed in Figure 4.16. The contributing area of the lower C value in Trial O was increased from the 70.8% of Trial N to 75.0%. The results of Trial O are shown in Table 4.9. The result of switching the *C* value of a single cell is a premature peak of 39.4

cfs that occurs with 19 of the 24 active cells contributing. A graphical comparison of the total discharge versus travel time for Trials N and O is shown in Figure 4.17.

0	.4	.4	.4	0
.9	.9	.4	.4	.4
.9	.9	.9	.9	.9
.9	.9	.9	.9	0
0	.9	.9	.9	0
0	.9	.9	0	0
0	.9	.9	0	0

Figure 4.16 Trial O: Arrangement of *C* Values, Lower Portion = 75.0% of the Watershed

The Δ*C* and magnitude of the *C* values were the same in both Trial N and Trial O. The difference between the trials was the contributing area of the lower *C* value, where Trial N had a lower portion that consisted of 70.8% of the watershed and Trial O had a lower portion that consisted of 75%. The contributing area of the lower *C* value in Trial N was not sufficient enough to cause a premature peak, whereas it was in Trial O. This indicates that the general guideline established in the linear trials also applies to the more spatially realistic watersheds. Fitting cubic polynomials to Trials N and O provided the following continuous relationships discharge and travel time:

$$Q = -240.72T_t^3 - 16.956T_t^2 + 148.76T_t - 6.0844 \tag{4-6}$$

$$Q = -528.28xT_t^3 + 225.07T_t^2 + 101.19T_t - 4.144 \tag{4-7}$$

The R^2 values for the cubic polynomials fitted to Trials N and O were 0.96 and 0.97. These R^2 values indicate that the cubic function fits the total discharge versus travel time data well for both trials.

Table 4.9 Trial O: Effect of Contributing Area of Lower *C*, Lower Portion = 75.8%

Number of cells	Time to outlet (hr)	Mean C	Rainfall intensity (in./hr)	Total area (acres)	Discharge (ft^3/s)
1	0.0623	0.9000	4.7559	0.9183	3.9305
4	0.1246	0.9000	4.1085	3.6731	13.5817
6	0.1661	0.9000	3.7666	5.5096	18.6774
10	0.2284	0.9000	3.3487	9.1827	27.6748
13	0.2700	0.9000	3.1180	11.9376	33.4991
16	0.3322	0.9000	2.8260	14.6924	37.3688
19	0.3945	0.8737	2.5840	17.4472	39.3893
21	0.4568	0.8286	2.3802	19.2837	38.0312
24	0.5191	0.7750	2.2062	22.0386	37.6820

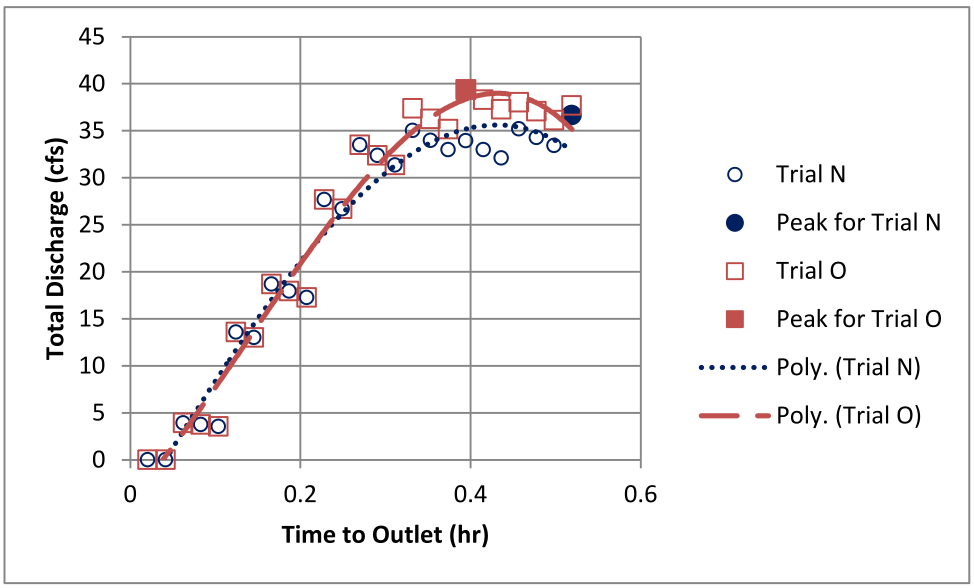


Figure 4.17 Comparing Discharge vs. Time to Outlet for Trials N and O

4.4 DISCHARGE RATIO MODEL

The discharge ratio was used in the spatially realistic trials to gage the relationship between the premature peak, if present, and the traditionally defined peak discharge that occurs for the entire watershed. A discharge ratio of 1.0 for a trial watershed indicated that a premature peak did not occur. An increase in the discharge ratio occurs when the difference between the premature peak and the entire contributing area discharge increases. A large discharge ratio means that infrastructure, such as stormwater conveyance pipes, could be sized differently depending on which peak was used for the design.

4.4.1 Effect of Cell Size

The previous trials consisted of 24 active cells that each had a cell length of 200 ft, which resulted in the example watersheds having an area of 22 acres. Increasing the number of active cells to 75 while keeping the total area equal to 22 acres would allow for simulations with a better spatial representation of hydrologic characteristics. The finer cell size would more accurately represent the watershed, and therefore, the results obtained from the 75-cell trials would be expected to be more reflective of actual runoff.

The boundaries of the 75-cell watershed lay within a *12 x 8* matrix, and the cell length was 113 ft, which was necessary to keep the desired total area of 22 acres. The flow ID (previously discussed in Section 4.2) of the 75-cell watershed is shown in Figure 4.18. The percent area of the lower and upper portions of the 75-cell trials were set to best match the 24-cell trials, which consisted of an upper portion of 54% and a lower portion of 46%. The lighter shaded area of Figure 4.18 consists of 40 cells that represent the upper 53% portion of the watershed. The darker shaded area consists of 35 cells that represent the lower 47% portion of the watershed. The lower portion was arranged such that it contained all cells that were within five flow paths of the outlet.

0	0	4	4	5	6	0	0
0	0	5	4	5	6	6	0
0	4	5	4	5	6	6	0
3	3	5	4	5	6	5	6
3	3	4	5	5	6	5	6
4	4	4	4	5	4	5	6
4	5	6	4	5	6	6	7
4	5	4	3	5	6	6	7
3	4	3	4	5	6	7	0
0	3	4	3	5	7	6	0
0	0	3	4	5	5	0	0
0	0	0	3	**0**	7	0	0

Figure 4.18 Flow ID for 75-cell Example Watershed

The 75-cell watershed was used to create 58 trials where the runoff coefficients and slopes were altered to analyze when a premature peak would occur. The data from these trials were used in conjunction with the data from the 98 24-cell trials to calibrate the discharge ratio model.

4.4.2 Model Calibration Using Numerical Optimization

A model was formulated to predict the discharge ratio. The calibration of the model was based on data from 156 of the realistic watershed trials (55 75-cell trials and 98 24-cell trials). The calibration data are presented in Appendix A. The variables used to develop the model were:

- S_L: average slope of the lower portion of the watershed, where the lower potion is the area closest to the outlet.
- ΔS: change in slope from the lower (S_L) to upper (S_U) portions of the watershed,

$$\Delta S = S_L - S_U \tag{4-8}$$

- C_L: average C value for lower portion of the watershed, and

- ΔC: change in C value from the lower (C_L) to upper (C_U) portions of the watershed,

$$\Delta C = C_L - C_U \tag{4-9}$$

An exponential model was selected to maintain rationality since a discharge ratio less than 1.0 does not make physical sense. An exponential model yields a predicted value of 1.0 when the exponent is zero. Therefore, if ΔS and ΔC are both zero, which occurs when the upper and lower portions have the same slope and same C, then a value of 1.0 for the discharge ratio would be expected. The following model was calibrated to the data base to predict the dimensionless discharge ratio (R_D):

$$R_D = \exp\left[0.4148\left(\frac{\Delta S}{S_L}\right)^{2.361} + 1.848\left(\frac{\Delta C}{C_L}\right)^{5.044}\right] \tag{4-10}$$

The model provided a standard error of estimate (S_e) of 0.07, a standard error ratio (S_e/S_y) of 0.45, and a R^2 of 0.80. These goodness-of-fit statistics indicate that the calibrated model can provide good predictions. The model has two constraints that are necessary to maintain rationality of the discharge ratio:

$$C_L \geq C_U \tag{4-11a}$$

$$S_L \geq S_U \tag{4-11b}$$

The constraint in Equation 4-11a requires that the average C value of the lower portion of the watershed is greater than that of the upper portion. Physically, this would mean that the portion of the watershed closest to the outlet is the more developed section. This constraint is reasonable in that it would be expected that a premature peak would be more likely to occur under these conditions. The constraint in Equation 4-11b requires that the average slope of the lower portion of the watershed is greater than that of the upper portion. Since a steeper slope conveys runoff more quickly, it would be expected that a premature peak would be more likely to occur under the conditions presented by this constraint.

4.4.3 Relative Sensitivity of Predictor Variables

Sensitivity analysis is used to measure the effect or importance of one element with respect to another element. Sensitivity can be expressed in three forms: absolute, relative, and deviation. The relative sensitivity was used to assess the relative importance of the predictor variables of the discharge ratio model because, as a dimensionless statistic, it allows for the sensitivity of all included variables to be directly compared. The relative sensitivities for ΔS, S_L, ΔC. and C_L were computed as:

$$\frac{\partial R_D/R_D}{\partial \Delta S/\Delta S} = \frac{0.9793\ \Delta S^{2.361}}{{S_L}^{2.361}} \tag{4-12a}$$

$$\frac{\partial R_D/R_D}{\partial S_L/S_L} = \frac{-0.9793\ \Delta S^{2.361}}{{S_L}^{2.361}} \tag{4-12b}$$

$$\frac{\partial R_D/R_D}{\partial \Delta C/\Delta C} = \frac{9.3213\ \Delta C^{5.044}}{{C_L}^{5.044}} \tag{4-12c}$$

$$\frac{\partial R_D/R_D}{\partial C_L/C_L} = \frac{-9.3213\ \Delta C^{5.044}}{{C_L}^{5.044}} \tag{4-12d}$$

These analyses indicate that the relative sensitivity of ΔS is equal to that of S_L, and the relative sensitivity of ΔC is equal to that of C_L. Therefore, the computation of the relative sensitivities will indicate whether slope or runoff coefficient is the more important element in an individual watershed.

The hydrologic characteristics of the watershed, and therefore, the configuration of the S and C values of the trials, dictate the inputs for determining the relative sensitivities of each of the predictor variables. The first example watershed had input parameters of ΔS = 1, S_L = 3%, ΔC = 0.3, and C_L = 0.6. The configuration of this watershed is shown in Figure 4.19. The relative sensitivities ($R_\$$) for the predictor variables in this setup were:

$$|R_{\$\Delta S}| = |R_{\$SL}| = 0.07 \tag{4-13a}$$

$$|R_{\$\Delta C}| = |R_{\$CL}| = 0.28 \tag{4-13b}$$

In this scenario, the relative sensitivity of the variables related to the *C* values was greater than those related to the slopes. These sensitivity results seem reasonable as a computed peak discharge is directly related to *C* while the discharge is influenced by slope through the calculation of the travel time and, therefore, the rainfall intensity.

0	.3	.3	.3	0
.3	.3	.3	.3	.3
.3	.3	.3	.3	.3
.6	.6	.6	.6	0
0	.6	.6	.6	0
0	.6	.6	0	0
0	.6	.6	0	0

(a) Runoff Coefficients

0	2	2	2	0
2	2	2	2	2
2	2	2	2	2
3	3	3	3	0
0	3	3	3	0
0	3	3	0	0
0	3	3	0	0

(b) Slopes (as %)

Figure 4.19 Relative Sensitivity Analysis: First Arrangement of *C* and *S* Values

The relative sensitivity varies from watershed to watershed. The second example watershed had input parameters of ΔS = 2%, S_L = 3%, ΔC = 0.3, and C_L = 0.6. The second example differed from the first example in that the value for ΔS was increased by 1%. The configuration of this watershed is shown in Figure 4.19. The relative sensitivities ($R_\$$) for the predictor variables in this setup were:

$$|R_{\$\Delta S}| = |R_{\$SL}| = 0.38 \tag{4-14a}$$

$$|R_{\$\Delta C}| = |R_{\$CL}| = 0.28 \tag{4-14b}$$

The increased difference between the lower and upper slope values leads to the predictor variables related to slope becoming more sensitive than those related to the C values. In this case, the higher slope causes the peak discharge to be more sensitive to travel times and, therefore, rainfall intensities. This shows how the relative sensitivities of the predictor variables vary depending on the hydrologic conditions of each watershed.

0	.3	.3	.3	0
.3	.3	.3	.3	.3
.3	.3	.3	.3	.3
.6	.6	.6	.6	0
0	.6	.6	.6	0
0	.6	.6	0	0
0	.6	.6	0	0

(a) Runoff Coefficients

0	1	1	1	0
1	1	1	1	1
1	1	1	1	1
3	3	3	3	0
0	3	3	3	0
0	3	3	0	0
0	3	3	0	0

(b) Slopes (as %)

Figure 4.20 Relative Sensitivity Analysis: Second Arrangement of C and S Values

4.5 CONSIDERATION OF ALTERNATE WATERSHED CONFIGURATIONS

The previously discussed trials were based on the assumption that the land characteristics of the lower portion of the watershed were such that the runoff rates would be greater than in the upper portion of the watershed. This assumption was inherent in the configuration of the runoff coefficients of the linear trials and in the arrangement of the matrices for runoff coefficients and slopes of the spatially realistic trials. The runoff coefficients for both methods were such that the C values in the lower portions of the watershed (C_L) were higher than the C values in the upper portion (C_U). A higher runoff coefficient reflects a more urbanized area whose increased impervious area lends itself to a greater quantities and faster rates of runoff. The slopes of the spatially realistic trials were arranged so that the slope in the lower portion of the watershed (S_L) was steeper than the slope in the upper portion (S_U). As identified in the general guidelines presented earlier, these arrangements are ones for which a premature peak would be expected to occur given certain watershed conditions (e.g., if ΔC is sufficiently large).

Several other general arrangements may be considered to show the effect of watershed characteristics on the occurrence of a premature peak. The following cases presented are some of the additional configurations that could occur within a watershed. The cases were set-up in a similar manner to the previously discussed spatially realistic trials. The watershed boundary is the same, resulting in 24 active cells. The flow ID numbers are the same as displayed in Figure 4.7. A return period of 2 years and a cell length of 200 ft were used as inputs.

4.5.1 Test Case #1: Watershed Homogeneity ($C_L = C_U$, $S_L = S_U$)

Test Case #1 considers a watershed that is homogeneous in terms of the runoff coefficients and slopes. In this scenario, both C_L and C_U were equal to 0.4. Therefore, the ΔC would be equal to zero. The slope of the entire watershed was 2 percent; therefore, ΔS is also equal to zero. Graphical representations of the arrangements of the runoff coefficients and slopes of the watershed are shown in Figure 4.21(a) and (b), respectively.

0	.4	.4	.4	0
.4	.4	.4	.4	.4
.4	.4	.4	.4	.4
.4	.4	.4	.4	0
0	.4	.4	.4	0
0	.4	.4	0	0
0	.4	.4	0	0

(a) Runoff Coefficients

0	2	2	2	0
2	2	2	2	2
2	2	2	2	2
2	2	2	2	0
0	2	2	2	0
0	2	2	0	0
0	2	2	0	0

(b) Slopes (as %)

Figure 4.21 Test Case #1: Arrangement of Runoff Coefficients and Slopes

For the configuration of Test Case #1, a premature peak would not be expected because the land characteristics are the same throughout the watershed. The watershed model did not produce a premature peak. The maximum discharge rate of 14.1 cfs occurred when the entire watershed area was contributing, with a maximum travel time to the outlet of 0.8 hr. The observed discharge ratio was equal to 1.0. The increase in total discharge occurred linearly as more of the cells were included in the contributing area, as shown by the trend in the points in Figure 4.22. The filled point within the data series represents the occurrence of the peak discharge. A cubic function was fitted to the points, but due to the homogeneity of the data the best-fit line was of a linear form:

$$Q = 15.831T_t - 0.6494 \quad (4\text{-}15)$$

This model yields a R^2 of 0.94, which indicates that it is a good fit. The linear trend occurs because the watershed is homogeneous.

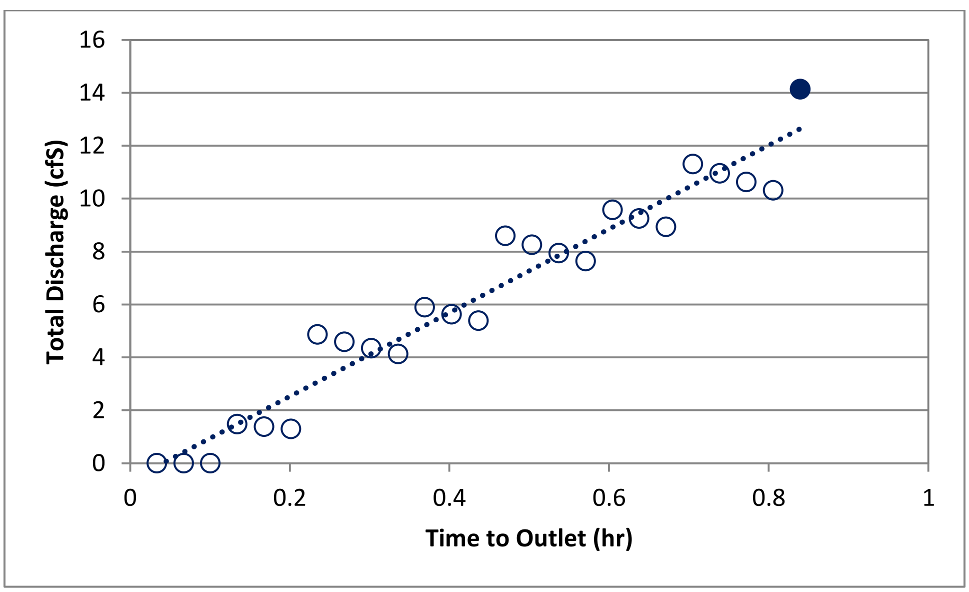


Figure 4.22 Test Case #1: Total Discharge vs. Time to Outlet with a Linear Trendline The Filled Point Corresponds to the Peak Discharge.

Equation 4-10 yielded a predicted discharge ratio of 1.0 because the watershed is homogeneous. This occurs because both ΔC and ΔS are equal to zero. Thus, the model reduces to the exponential of zero, which equal 1.0. The homogeneity of the watershed yields a scenario where a premature peak will probably not occur, which is reflected in the computation of the predicted discharge ratio.

4.5.2 Test Case #2: Urbanization in the Upper Portion ($C_L < C_U, S_L = S_U$)

Test Case #2 considers a watershed where the upper 45.8% consists of a more urbanized area than the lower portion, which resulted in $C_L < C_U$, with C_L equal to 0.4 and C_U equal to 0.7. As ΔC was previously defined as C_L minus C_U, the ΔC for Test Case #2 was -0.3. The slope of the watershed was homogeneous throughout. Graphical representations of the arrangements of the runoff coefficients and slopes of the watershed are shown in Figure 4.23. A premature peak would not be expected for Test Case #2 because the magnitude and rate of runoff would be largest when the upper portion of the watershed was contributing.

0	.7	.7	.7	0
.7	.7	.7	.7	.7
.7	.7	.7	.7	.7
.4	.4	.4	.4	0
0	.4	.4	.4	0
0	.4	.4	0	0
0	.4	.4	0	0

(a) Runoff Coefficients

0	2	2	2	0
2	2	2	2	2
2	2	2	2	2
2	2	2	2	0
0	2	2	2	0
0	2	2	0	0
0	2	2	0	0

(b) Slopes (as %)

Figure 4.23 Test Case #2: Arrangement of Runoff Coefficients and Slopes

The maximum total discharge of 22.5 cfs occurred when the entire watershed area was contributing. The flow travel time to the outlet for the entire watershed was equal to 0.70 hr. The observed discharge ratio for Test Case #2 was 1.0. The relationship of the total discharge and the time to outlet was fit with a cubic function. The following cubic function yielded a R^2 of 0.96, which indicates that it is a good fit:

$$Q = 84.733T_t^{\,3} - 53.123T_t^{\,2} + 25.542T_t - 1.4646 \qquad (4\text{-}16)$$

As previously discussed in Section 3.1.2, taking the first derivative of the cubic function allows a mathematical analysis of the rate of change. Figure 4.25 shows the first derivative of Equation 4-16, which shows the rate of change of the discharge as a function of travel time. The location where the change in the runoff coefficient occurs is evident in the rate of change of total discharge over the watershed because it is where the rate of change begins to increase more rapidly. The time to outlet of 0.5 hr corresponds to the approximate place in the watershed where C_L changes to C_U.

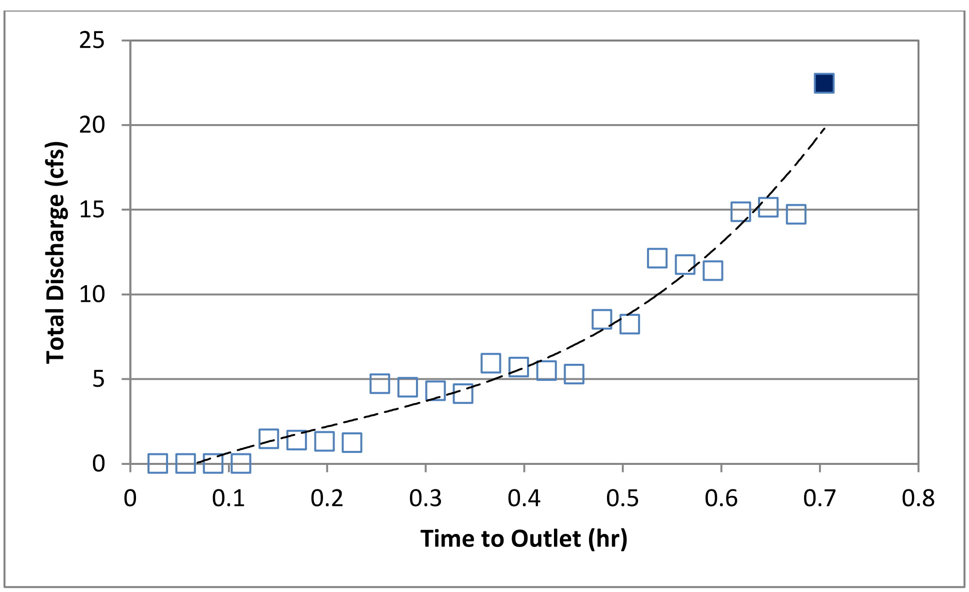


Figure 4.24 Test Case #2: Total Discharge vs. Time to Outlet with a Cubic Trendline. The Filled Point Corresponds to the Peak Discharge.

One of the constraints for the predicted discharge ratio model (Equation 4-10) was that C_L had to be greater than C_U. The watershed configuration of Test Case #2 is such that the model should not be used to compute the predicted discharge ratio. Using the discharge ratio model in this scenario would result in an imaginary number, which would indicate that the results are irrational. The relative sensitivities for *S* and *C* were computed as 0.00 and 2.18, respectively, using Equations 4-12(a)-(d). The runoff coefficient would be expected to have the greater relative sensitivity since it is nonhomogeneous across the watershed, whereas, the slope does not change.

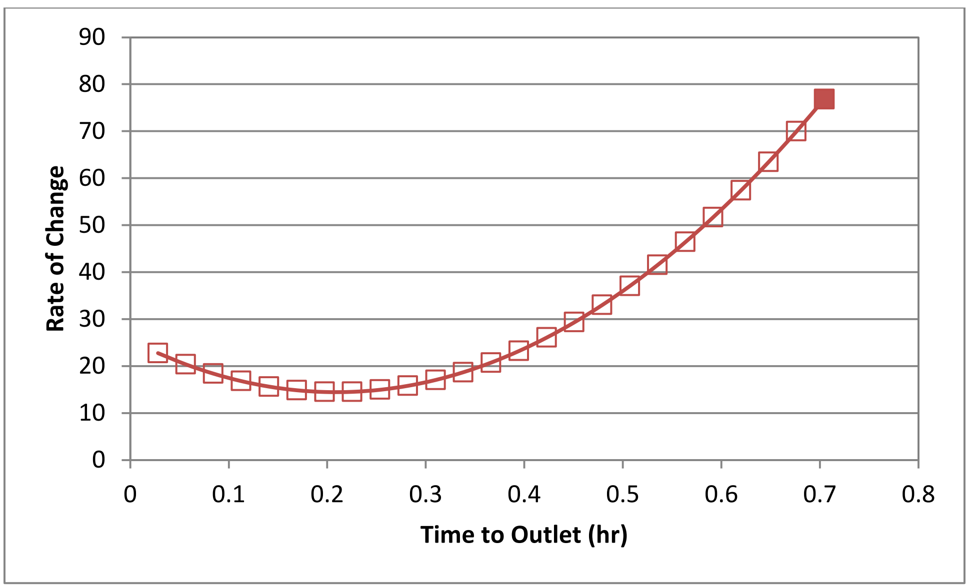


Figure 4.25 Test Case #2: First Derivative of the Cubic Function, Rate of Change vs. Time to Outlet The Filled Point Corresponds to the Peak Discharge.

4.5.3 Case #3: Urbanization in the Upper Portion, High Slope in the Lower Portion ($C_L < C_U$, $S_L > S_U$)

Test Case #3 considers a watershed with the same arrangement of runoff coefficients as Case #2, where the upper 45.8% C_U was equal to a value of 0.7 and the lower 54.2% C_L

was equal to a value of 0.4. This produces a ΔC equal to -0.3. However, the slope of the watershed was not kept homogeneous as it was in Test Case #2. In Test Case #3, S_L was equal to 4% and S_U was equal to 2%, which yields a ΔS of 2%. Graphical representations of the arrangements of the runoff coefficients and slopes of the watershed are shown in Figure 4.26.

0	.7	.7	.7	0
.7	.7	.7	.7	.7
.7	.7	.7	.7	.7
.4	.4	.4	.4	0
0	.4	.4	.4	0
0	.4	.4	0	0
0	.4	.4	0	0

(a) Runoff Coefficients

0	2	2	2	0
2	2	2	2	2
2	2	2	2	2
4	4	4	4	0
0	4	4	4	0
0	4	4	0	0
0	4	4	0	0

(b) Slopes (as %)

Figure 4.26 Test Case #3: Arrangement of Runoff Coefficients and Slopes

The maximum total discharge of 26.0 cfs for Test Case #3 occurs when the entire watershed is contributing. The time to outlet for the entire watershed was 0.56 hr. The observed discharge ratio was equal to 1.0 since the peak discharge occurred when all of the area contributed. Fitting the total discharge and the time to outlet with a cubic function (see Figure 4.27) yielded the following equation with a R^2 was equal to 0.95, which indicates a good fit.

$$Q = 94.125{T_T}^3 - 33.637{T_T}^2 + 31.491T_T - 1.3895 \tag{4-17}$$

The results show that a ΔS of 2% was not sufficiently large to produce a premature peak when ΔC was equal to -0.3. However, Test Case #3 is one of many watershed arrangements that fall within the conditions of $C_L < C_U$ and $S_L > S_U$, and it is possible that given the right combination of these factors that a premature peak could occur.

The first derivative was calculated as a function of the time to outlet to better analyze the rate of change of total discharge over the entire watershed. The results are graphed in Figure 4.28. The rate of change of total discharge for Case #3 shows a function that is slightly decreasing from the time to outlet of 0.00 hr to 0.16 hr, which can be attributed to the fact that the runoff coefficient for this portion is constant ($C_U = 0.4$), but as additional cells are included in the contributing area, the intensity decreases. When the Rational method is used to compute the total discharge, the decrease in intensity is more dominant than the increase in area, and therefore, the resulting rate of change function is decreasing. The function begins to rapidly increase at the point where the division between the lower and upper portions of the watershed was made. This rapid increase occurs because the slope of the lower portion is steeper.

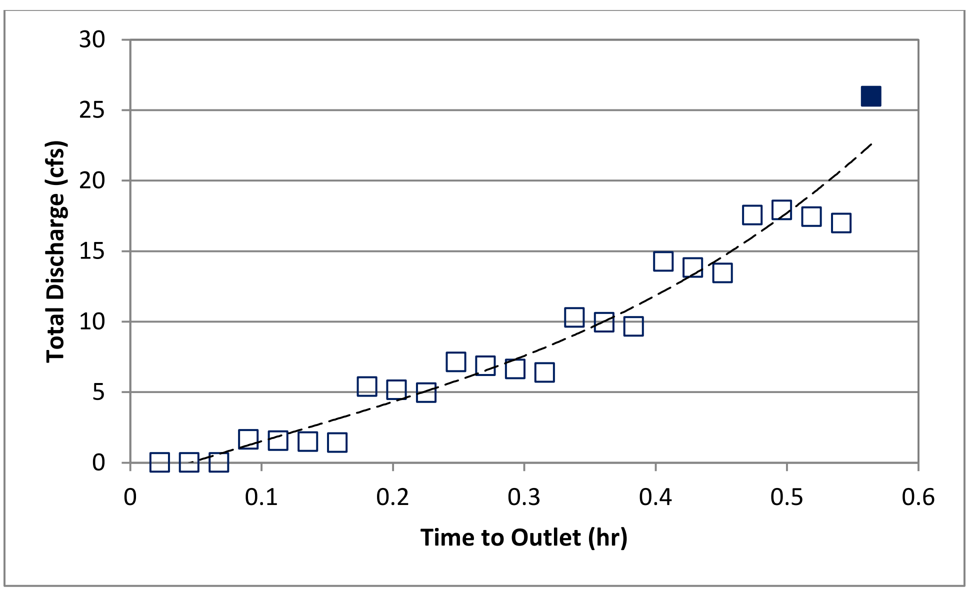


Figure 4.27 Test Case #3: Total Discharge vs. Time to Outlet with a Cubic Trendline The Filled Point Corresponds to the Peak Discharge.

One of the constraints for the predicted discharge ratio model was that C_L had to be greater than C_U. The watershed configuration of Test Case #3 is such that the model should not be used to compute the predicted discharge ratio because ΔC is a negative value. Trying to use the discharge ratio model for this configuration may lead to irrational results. The relative sensitivities for S and C were computed as 0.19 and 2.18, respectively, using Equations 4-12(a)-(d). These results indicate that the runoff coefficient is the more important element in this watershed.

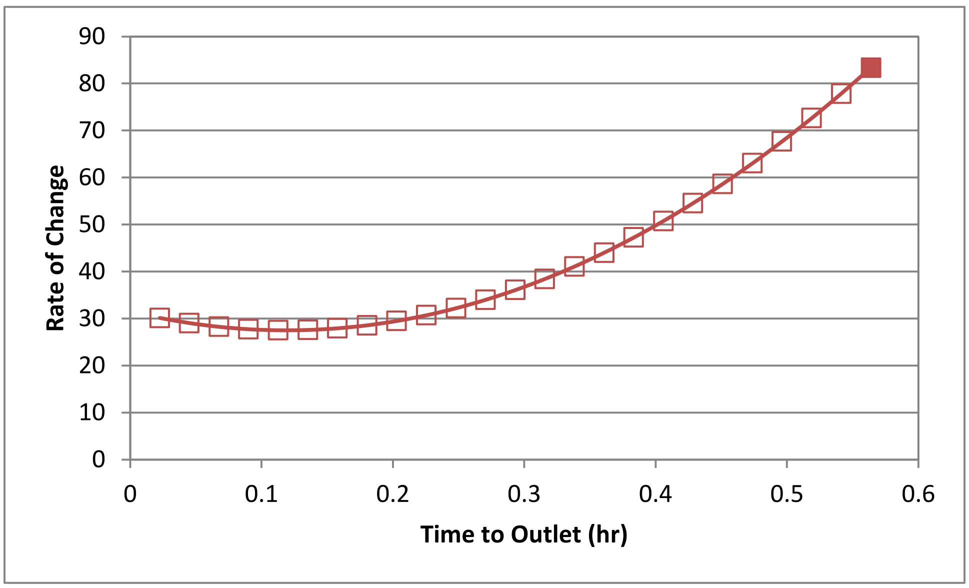


Figure 4.28 Test Case #3: First Derivative of the Cubic Function, Rate of Change vs. Time to Outlet The Filled Point Corresponds to the Peak Discharge.

4.6 EFFECT OF WATERSHED SHAPE ON THE TIMING OF RUNOFF

One of the traditional assumptions used to simplify the Rational method is that the total area of the watershed, but not its shape or spatial distribution, is the characteristic used to define the physical characteristics of the watershed. However, the basin shape would be expected to influence the timing of runoff to the outlet. Three general basin shapes were used for this analysis: normal, long, and wide. These example watersheds each consisted of 75 active cells, but with varying watershed lengths and widths. The flow ID arrangement for each example watershed shape is shown in Figure 4.29.

Normal (12 *x* 8)							
0	0	4	4	5	6	0	0
0	0	5	4	5	6	6	0
0	4	5	4	5	6	6	0
3	3	5	4	5	6	5	6
3	3	4	5	5	6	5	6
4	4	4	4	5	4	5	6
4	5	6	4	5	6	6	7
4	5	4	3	5	6	6	7
3	4	3	4	5	6	7	0
0	3	4	3	5	7	6	0
0	0	3	4	5	5	0	0
0	0	0	3	**0**	7	0	0

Wide (6 *x* 15)														
0	0	4	4	4	4	5	5	5	6	6	6	6	0	0
4	4	5	5	5	5	4	3	6	6	5	5	6	7	0
3	3	4	4	5	5	4	5	6	4	5	6	7	6	7
3	3	3	3	3	4	5	5	7	6	6	6	6	7	7
0	0	3	3	2	4	3	5	6	6	7	7	7	0	0
0	0	0	3	3	3	3	**0**	7	7	7	7	0	0	0

Long (15 *x* 6)					
0	0	4	5	0	0
0	4	5	5	6	0
5	4	5	5	6	6
5	5	5	6	6	7
4	5	5	6	6	5
3	5	5	6	5	5
3	5	5	5	5	6
3	3	5	7	5	7
4	4	5	5	5	6
3	4	5	5	6	6
3	4	5	6	6	6
3	4	5	5	7	7
0	4	5	6	7	0
0	4	5	7	0	0
0	0	**0**	7	0	0

Figure 4.29 Basin Shape Analysis: Normal, Wide, and Long Watersheds

The normal basin shape was outlined within a 12*x*8 matrix of cells, which yielded a length-to-width ratio of 1.5. The wide basin shape and long basin shape were outlined within a 6*x*15 and a 15*x*6 matrix of cells, respectively, which yielded length-to-width ratios 0.40 and 2.50. The nonzero values indicate the direction of flow for cells within the watershed, while the zero value that lies within the watershed border denotes the location of the outlet. The difference in the timing of runoff between these three basin shapes was analyzed by comparing the discharge versus travel times results for a scenarios where a premature peak was expected. The results of these examples were analyzed to assess the general effect of basin shape on the timing of runoff.

4.6.1 Effect of Basin Shape: Premature Peak Expected

The developed general guidelines indicate that a premature peak is likely to occur when the land cover is significantly nonhomogeneous. Following this guideline, a scenario was arranged such that the C value in the upper portion of the watershed was equal to 0.2 and the C value in the lower portion of the watershed was equal to 0.8, which yielded a ΔC = 0.6. The slope of the upper portion of the watershed was equal to 2% and the slope in the lower portion of the watershed was equal to 4%, which yielded a ΔS = 2%. This setup would be expected to result in a premature peak because the previously discussed Trial L, a 24-cell watershed, experienced a premature peak, and it consisted of C values in the lower (46%) and upper (54%) portions of the watershed of 0.6 and 0.2, respectively, which yielded a ΔC = 0.4. The slope in Trial L was 2% throughout the entire watershed. The lower portion of the 75-cell watersheds consisted of 35 cells (47%) and the upper portion consisted of 40 cells (53%). The division of the lower and upper portions of the

75-cell watersheds were arranged to match the conditions of Trial L and are shown in different shades of grey in Figure 4.29. These inputs were used for all three basin shapes.

The normal basin shape had a premature peak discharge of 30.78 cfs that occurred when 35 of the 75 cells contributed. The travel time for those 35 cells was 0.17 hr, which corresponded to a rainfall intensity equal to 3.75 in./hr. The total discharge when all 75 cells contributed was 17.39 cfs with a travel time of 0.81 hr and a rainfall intensity equal to 1.65 in./hr. This resulted in an observed discharge ratio of 1.77 and a predicted discharge ratio (Equation 4-10) of 1.67. The total discharge versus time-to-outlet for the normal basin shape is shown in Figure 4.30.

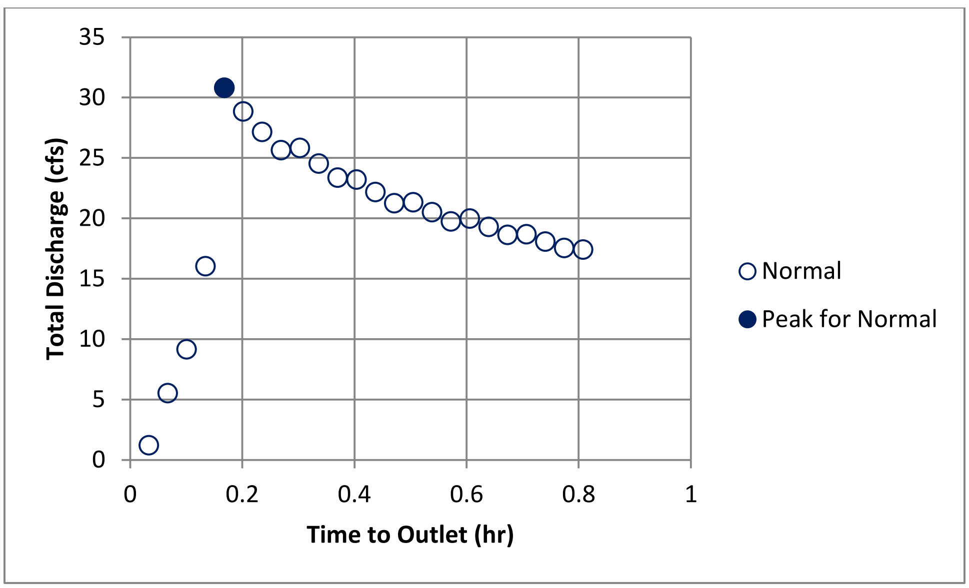


Figure 4.30 Normal Basin Shape: Total Discharge vs. Time to Outlet (C_L = 0.8, ΔC = 0.6)

The wide basin shape had a premature peak discharge of 32.51 cfs that occurred when 35 of the 75 cells contributed. The travel time for those 35 cells was 0.14 hr, which corresponded to a rainfall intensity equal to 3.98 in./hr. The total discharge when all 75 cells contributed was 21.50 cfs with a travel time of 0.59 hr and a rainfall intensity equal to 2.04 in./hr. The observed discharge ratio for the wide basin shape was 1.51 and the predicted discharge ratio (Equation 4-10) was 1.67. The total discharge versus time-to-outlet for the wide basin shape is shown in Figure 4.31.

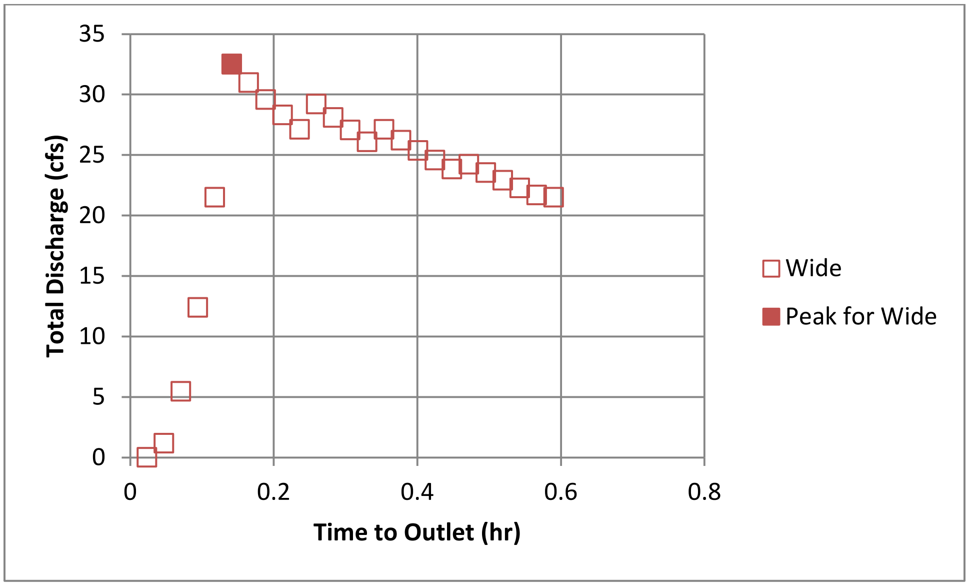


Figure 4.31 Wide Basin Shape: Total Discharge vs. Time to Outlet ($C_L = 0.8$, $\Delta C = 0.6$)

The long basin shape had a premature peak discharge of 26.81 cfs that occurred when 35 of the 75 cells contributed. The travel time for those 35 cells was 0.24 hr, which corresponded to a rainfall intensity equal to 3.26 in./hr. The total discharge when all 75 cells contributed was 14.76 cfs with a travel time of 1.01 hr and a rainfall intensity equal to 1.40 in./hr. The observed discharge ratio for the long basin shape was 1.82 and the predicted discharge ratio (Equation 4-10) was 1.67. The total discharge versus time-to-outlet for the long basin shape is shown in Figure 4.32.

The normal, wide, and long basin shapes were quantified using a couple of the dimensionless parameters previously discussed in Section 2.7. The form factor, R_F, is defined as the ratio of the basin area to the square of the basin length. In these 75-cell trials, the total area was equal to 22 acres (958320 ft^2). The length of an individual cell was 113 ft, which resulted in basin lengths of 1256 ft, 678 ft, and 1695 ft for the normal, wide, and long basin shape, respectively. Therefore, the resulting form factors for the normal, wide, and long basin shapes were 0.61, 2.08, and 0.33. The elongation ratio, R_E, is defined as the ratio of diameter of a circle with the same area as that of the basin to the maximum basin length. The diameter of a circle with an area of 958320 ft^2 is equal to 1105 ft. Using the basin lengths from the calculation of the form factors, yields elongation ratios for the normal, wide, and long basin shapes of 0.88, 1.63, and 0.65, respectively. The value of the elongation ratio is 1.0 if the drainage basin is a perfectly circular shape. The computed elongation ratios indicate that the normal basin shape is most similar to a circular watershed, whereas the wide and long basin shapes are more elliptical.

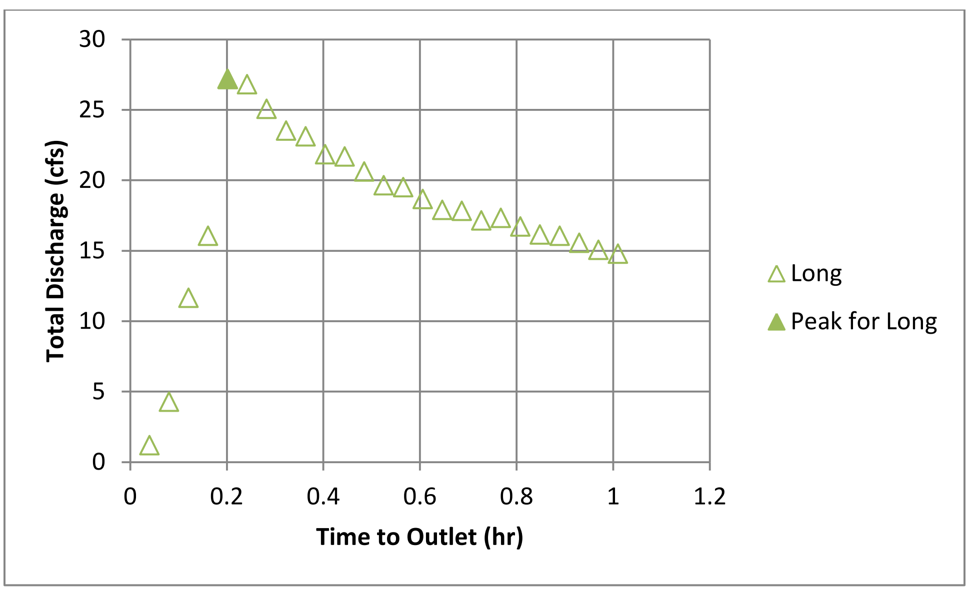


Figure 4.32 Long Basin Shape: Total Discharge vs. Time to Outlet

The results from the normal, wide, and long basin shapes can be compared to assess the general effects of watershed shape on the timing of runoff. All three basin shapes experienced a premature peak when 35 of the 75 cells contributed, but the magnitude of the peak discharge varied as well as the travel time required for runoff to reach the outlet. Of the three basin shapes, the wide shape had the largest Q_{peak} (30.78 cfs) and $Q_{75\ cells}$ (21.50 cfs) with the shortest $T_{t\ peak}$ (0.14 hr) and $T_{t\ 75\ cells}$ (0.59 hr). The long basin shape had the smallest Q_{peak} (26.81 cfs) and $Q_{75\ cells}$ (14.76 cfs) with the longest $T_{t\ peak}$ (0.24 hr) and $T_{t\ 75\ cells}$ (1.01 hr). The observed discharge ratio was the smallest for the wide basin and the largest for the long basin. The predicted discharge ratio was the same for all three basin shapes since it depends on inputs of ΔC, C_L, ΔS, and S_L. A tabular comparison of Q_{peak}, $Q_{75\ cells}$, observed D_R, predicted D_R, $T_{t\ peak}$ and $T_{t\ 75\ cells}$ for all three basin shapes is shown in Table 4.10. A graphical comparison of the total discharge versus time-to-outlet for all three basin shapes is shown in Figure 4.33.

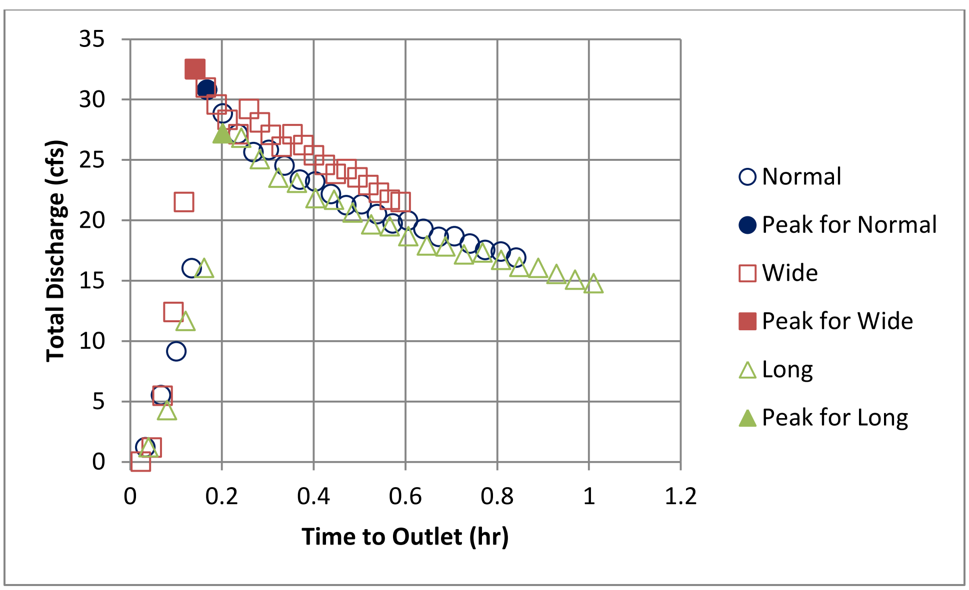


Figure 4.33 Comparison of Normal, Wide, and Long Basin Shapes

Table 4.10 Comparison of Normal, Wide, and Long Basin Shapes

Shape	Q_{peak} (cfs)	$Q_{75\ cells}$ (cfs)	Observed D_R	Predicted D_R	$T_{t\ peak}$ (hr)	$T_{t\ 75\ cells}$ (hr)
Normal	30.78	17.39	1.77	1.67	0.17	0.81
Wide	32.51	21.50	1.51	1.67	0.14	0.59
Long	26.81	14.76	1.82	1.67	0.24	1.01

The results indicate that the basin shape influences the travel time of the runoff through a watershed. The distance the runoff discharge must travel varies depending on the watershed length and width because the travel distance of each cell to the watershed outlet is inherent in the flow identification paths. The distance of each cell to the watershed outlet can be defined as how many cells the water must flow through in order to reach the outlet. The number of cells that the water must flow through to reach the outlet was referred to as the flow path distance. The flow path distances for each cell of the normal, wide, and long basin shapes (see Figure 4.29) are shown in Figure 4.34.

The maximum flow path distance for the normal, wide, and long basin shapes was 11, 8, and 14, respectively (see Figure 4.34). The structure of the flow identification path provides the path that the runoff must travel throughout the watershed, and the flow path distance demonstrates the distance, in terms of the number of cells, that the runoff must travel to reach the watershed outlet. The long basin shape resulted in the greatest maximum flow path distance (i.e., 14 cells) because the cells at the top of the watershed were the furthest away from the outlet and thus would require runoff to travel through more cells in order to reach the outlet. The maximum flow path distance of the wide basin shape (i.e., 8 cells) started at the side of the watershed because the cells on the left-most column of the watershed were the further away from the outlet than the cells on the top row of the watershed. This occurred because of the basin's length-to-width ratio. The long watershed was 15 cells long and the wide watershed was 15 cells wide, but the location of the outlet in the watershed determined the maximum flow path distance.

Based off of these results, a general guideline regarding the effect of basin shape on the timing of runoff was created. The general geometrical properties of basin shape, length and width, impact the distance that runoff must travel through the watershed because the maximum distance depends on the location of the cell that is farthest, in terms of flow path distance, from the outlet.

Normal (12 *x* 8)							
		11	11	11	11		
		10	10	10	10	10	
	9	9	9	9	9	9	
10	9	8	8	8	8	8	8
9	8	7	7	7	7	7	7
6	6	6	6	6	6	6	6
5	5	5	5	5	5	5	6
4	4	5	5	4	4	4	5
4	3	4	3	3	3	4	
	3	2	3	2	3	3	
		2	1	1	2		
			1	**0**	1		

Wide (6 *x* 15)														
		7	7	6	5	5	6	5	5	5	6	6		
8	7	7	6	6	5	4	5	4	4	5	5	5	6	
8	7	6	5	5	4	3	3	3	4	4	4	5	6	7
8	7	6	5	4	3	3	2	3	2	3	4	5	6	7
		6	5	4	2	2	1	1	2	3	4	5		
			4	3	2	1	**0**	1	2	3	4			

Long (15 *x* 6)					
		14	14		
	13	13	13	13	
13	12	12	12	12	12
12	12	11	11	11	12
11	11	10	10	10	10
11	10	9	9	9	9
10	9	8	9	8	8
9	8	7	8	7	8
6	6	6	6	6	6
6	5	5	5	5	5
5	4	4	4	4	5
4	3	3	3	4	5
	2	2	2	3	
	1	1	2		
		0	1		

Figure 4.34 Flow Path Distances for the Normal, Wide, and Long Basin Shapes

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

5.1 CONCLUSIONS

The goal of this research was to improve the understanding of the effects of timing of microwatershed runoff on hydrologic design. This goal was accomplished through the following objectives: (1) to demonstrate the effects of challenging the traditional assumptions that underlie the Rational method; and (2) to develop guidelines for determining the conditions that would produce a peak discharge that was larger than that produced when the entire watershed contributed runoff. The results will improve the existing state of hydraulic design by enhancing the knowledge of when a premature peak can be expected to occur and by how much it differs from the traditionally computed design discharge.

To achieve these objectives, three different watershed trial setups were developed. The initial trials, discussed in Chapter 3, were developed and analyzed for the purpose of understanding the general conditions under which the maximum discharge from a subarea of a watershed could exceed the discharge for the case where the entire watershed contributed runoff. These initial trials were created to better understand how the inputs of travel time and the runoff coefficient affected the occurrence of a premature peak. General guidelines for the occurrence of a premature peak were formulated based on these 20-cell trials. However, because the initial trials were arranged in a linear fashion, it was necessary to create models that consisted of watersheds that were more spatially realistic. The second watershed setup consisted of 24-cell trials that were used to verify the general guidelines. The third watershed setup consisted of 75-cell trials that allowed for simulations with a better spatial representation of hydrologic characteristics. The effects of the following traditional assumptions that underlie the Rational method were analyzed using the three different watershed trial setups:

(1) The runoff coefficient does not vary within the drainage area.

(2) The watershed is relatively homogeneous in terms of factors that affect the runoff coefficient, such as topography and slope.

(3) The maximum discharge occurs when runoff is contributed from the entire drainage area.

(4) The total area of the watershed, but not its shape or spatial distribution, is the characteristic used to define the physical characteristics of the watershed.

The model analyses resulted in the development of general guidelines that would indicate to the design engineer that a premature peak could be expected to occur if the hydrologic characteristics of a watershed are significantly nonhomogeneous. This research assessed how the timing of runoff was affected by the spatial distribution of the runoff coefficient and slope. Using the general guidelines, the design engineer could expect a premature peak to be more likely to occur under conditions when: (1) the difference in runoff coefficients between the upper and lower reaches of the watershed was significantly large; (2) the magnitudes of the C values for the lower and upper portions of the watershed are relatively large; and (3) the contributing area of the smaller C value, which is located in the upper portion of the watershed, is sufficiently large.

Trials from both the 24-cell and the 75-cell watershed scenarios were used to calibrate a model that would predict the dimensionless discharge ratio. The discharge ratio gages the relationship between the magnitudes of a premature peak, if present, and the traditionally defined peak discharge that results when the entire watershed contributes runoff. The discharge ratio could be used by the design engineer to determine whether a premature peak would be expected for an individual watershed. A discharge ratio of about 1.0 would indicate that a premature peak was not expected and that the traditionally used discharge for the entire watershed could be used for design. A large discharge ratio indicates that a more in-depth analysis of the microwatershed is necessary prior to the design of hydraulic infrastructure. This research also indicated that the timing of runoff was affected by watershed shape, and part of the additional in-depth analysis may be that the design engineer takes into account the general geometrical shape of the watershed.

The predicted discharge ratio equation can be used to perform a sensitivity analysis for a watershed. The computed relative sensitivity of ΔS, S_L, ΔC. and C_L provides an indication of the relative importance of the runoff coefficient or slope. This would vary from watershed-to-watershed because it depends on the hydrologic conditions of an individual watershed.

Design engineers will benefit from the guidelines regarding the occurrence of premature peak discharge rates. Identification of the conditions under which a premature peak may occur coupled with the predicted discharge ratio creates indicators for when additional watershed analyses are necessary prior to the design of hydraulic infrastructure for a microwatershed. The design engineer can then assess whether the stormwater system would be underdesigned if the traditional design discharge was used rather than the larger premature peak discharge.

5.2 IMPLICATIONS OF PREMATURE PEAKS

The implication of a premature peak can be demonstrated by showing how the use of the premature peak rather than the traditional peak impacts hydraulic design. The following design example shows the sizes of pipes required by the two peak discharge rates.

5.2.1 Design Example

The pipe size required to convey the runoff from a watershed can be estimated using Manning's equation and the continuity equation. Manning's equation is given as:

$$v = \frac{1.49}{n} {R_h}^{2/3} S^{1/2} \tag{5-1}$$

where, *v* is the mean velocity of the flow (ft/s), *n* is the Manning's roughness coefficient, R_h is the hydraulic radius (ft), and *S* is the slope of the hydraulic grade line (ft/ft). The hydraulic radius is defined as the ratio of the cross-sectional area to the wetted perimeter. Using the assumption of full-flow along with the formulas for area and circumference of a pipe, the hydraulic radius can be calculated in terms of diameter (*d*) as:

$$R_h = \frac{d}{4} \tag{5-2}$$

Coupling the continuity equation (Equation 5-3) with Manning's equation yields Equation 5-4:

$$Q = vA \tag{5-3}$$

$$Q = \frac{1.49}{n} A \, {R_h}^{2/3} S^{1/2} \quad (5\text{-}4)$$

Equation 5-4 can be solved for the pipe diameter as follows by substituting in Equation 5-2 and the formula for the area of a pipe:

$$d = \left[\frac{2.16 Q n}{S^{1/2}}\right]^{3/8} \quad (5\text{-}5)$$

Comparing the pipe diameter necessary to accommodate the runoff discharge for a premature peak to that of when the entire watershed is included will show an implication of using a premature peak for hydraulic design. The 75-cell normal basin shape, with the same flow identification paths as shown in Figure 4.29 was arranged with the following hydrologic characteristics: the lower runoff coefficient, C_L, was equal to 0.6, the upper runoff coefficient, C_U, was equal to 0.2, the difference between the lower and upper runoff coefficients, ΔC, was equal to 0.4, and the lower and upper slopes, S_L and S_U, were equal to 2%. The upper and lower portions were 53% and 47% of the watershed, respectively. Figure 5.1 shows the arrangement of the runoff coefficients in the watershed.

0	0	.2	.2	.2	.2	0	0
0	0	.2	.2	.2	.2	6	0
0	.2	.2	.2	.2	.2	.2	0
.2	.2	.2	.2	.2	.2	.2	.2
.2	.2	.2	.2	.2	.2	.2	.2
.2	.2	.2	.2	.2	.2	.2	.2
.6	.6	.6	.6	.6	.6.	.6	.2
.6	.6	.6	.6	.6	.6.	.6	.6
.6	.6	.6	.6	.6	.6	.6.	0
0	.6	.6	.6	.6	.6	.6.	0
0	0	.6	.6	.6	.6	0	0
0	0	0	.6	**0**	.6	0	0

Figure 5.1 Design Example: Arrangement of Runoff Coefficients in the Watershed

The hydrologic conditions of the watershed resulted in a premature peak of 43.28 cfs that occurred when 35 of the 75 cells contributed. The runoff discharge computed for all 75 cells was 27.11 cfs. The discharge versus travel time results for the design watershed is shown in Figure 5.2. The observed discharge ratio was equal to 1.5 and the predicted discharge ratio (Equation 4-10) was equal to 1.3. The discharge ratio for this example watershed would indicate to the design engineer that a premature peak would be expected, and it is different enough from the entire peak to warrant a more in-depth analysis of the watershed.

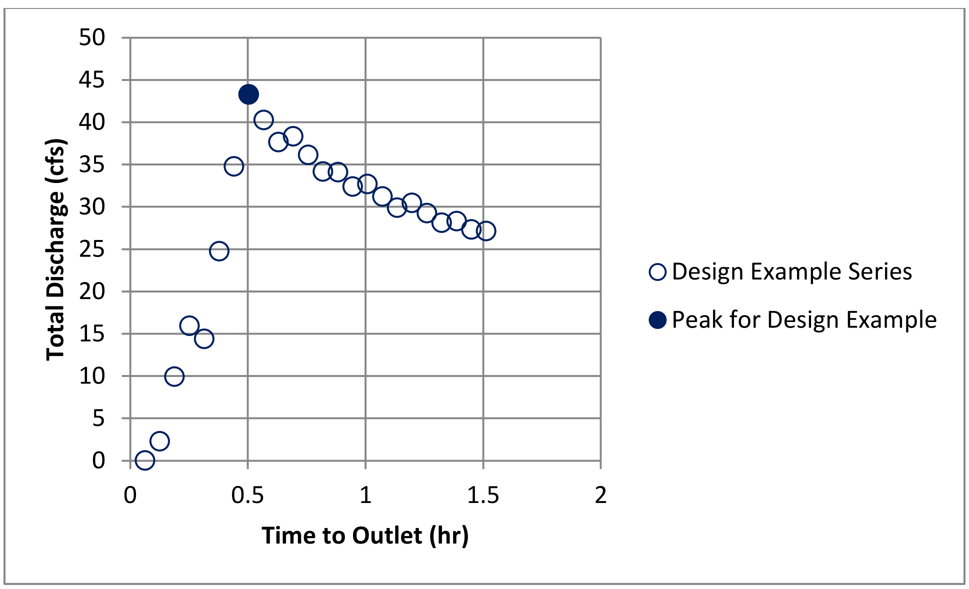


Figure 5.2 Design Example: Total Discharge versus Time-to-Outlet

The premature peak and the discharge for the entire watershed were used as input into Equation 5-5 to calculate the required pipe diameter. The Manning's roughness coefficient for reinforced concrete pipe, n = 0.013, was used for both. A slope of 0.02 was assumed. Using the premature peak (43.28 cfs) to design the pipe size resulted in a pipe diameter of 26.9 in, which would result in the design requirement of a 27 in. pipe.

Using the discharge for the entire watershed (27.11 cfs) resulted in a required pipe diameter of 22.6 in, which would dictate a 24 in. pipe.

The implication of the difference in pipe size depending on which discharge value is used in design calculations, premature peak versus the traditional peak, include the ability of the stormwater infrastructure to handle the runoff from a watershed. The premature peak requires a larger pipe size to effectively manage the runoff. Using the discharge from the entire watershed when a premature peak is present may lead to a pipe size that doesn't have the capacity to manage all of the runoff. Thus, ponding of the excess runoff would result. This can create a safety hazard for the public and increase the potential for flood damage.

5.3 RECOMMENDATIONS

While this research has improved the understanding of the effects of timing of microwatershed runoff on hydrologic design, additional research is still needed. The following are recommended research areas that will further progress in the field of hydrologic design.

5.3.1 Inclusion of Additional Hydrologic Characteristics

In this research, the runoff coefficient and slope were primarily used to define the hydrologic characteristics of a watershed. However, additional watershed characteristics could be used to develop a model to predict the occurrence of a premature peak. One example would be the effect of urbanization. The flow identification path would differ for a more urbanized area in that the runoff would be conveyed in a less meandering manner than that of a natural channel. Additional trials would need to be made to assess this effect and then the model would need to be calibrated based on the results of these trials.

5.3.2 Watersheds with Multiple Nonhomogeneous Portions

The watershed trials in this research were divided into upper and lower portions. The portions represented two parts of a watershed that were nonhomogeneous in terms of runoff coefficient and/or slope. However, more diverse watersheds exist, where the land uses show much greater nonhomogeneity. Multiple sections would better reflect a more diverse watershed. A model could be developed based on parameters that relate to all of the portions, and then a sensitivity analysis could be used to determine which hydrologic characteristics and sections were most important to an individual watershed.

5.3.3 Development of Model Including Basin Shape

Although the general effect of basin shape on the timing on microwatershed runoff was assessed in Section 4.6, it was not included in the equation for the predicted discharge ratio. The development of a model that accounts for watershed shape requires a more comprehensive study on how the geometry of a watershed impacts the timing of the runoff. The watershed has an endless possibility of shapes that would need to be generalized into the shapes that most commonly occur. The watershed length and width, or other shape parameters, could become calibration variables.

APPENDIX A

CALIBRATION DATA FOR EQUATION 4-10

The calibration of the model to predict the discharge ratio (Equation 4-10) was based on data from 156 of the realistic watershed trials, 98 24-cell trials (Table A.1) and 58 75-cell trials (Table A.2)

Table A.1 Calibration Data: 24-cell trials

Trial	# Cells Upper	# Cells Lower	Premature peak?	Upper C	Lower C	ΔC	Lower Slope	ΔSlope	Q_{peak} / Q_{end}
1	13	11	N	0.2	0.3	0.1	2.0000	0.0000	1.0000
2	13	11	N	0.2	0.4	0.2	2.0000	0.0000	1.0000
3	13	11	Y	0.2	0.5	0.3	2.0000	0.0000	1.0513
4	13	11	Y	0.2	0.6	0.4	2.0000	0.0000	1.1536
5	13	11	Y	0.2	0.7	0.5	2.0000	0.0000	1.2499
6	13	11	Y	0.2	0.8	0.6	2.0000	0.0000	1.3434
7	13	11	Y	0.2	0.9	0.7	2.0000	0.0000	1.3611
8	13	11	N	0.3	0.4	0.1	2.0000	0.0000	1.0000
9	13	11	N	0.3	0.5	0.2	2.0000	0.0000	1.0000
10	13	11	N	0.3	0.6	0.3	2.0000	0.0000	1.0000
11	13	11	Y	0.3	0.7	0.4	2.0000	0.0000	1.0263
12	13	11	Y	0.3	0.8	0.5	2.0000	0.0000	1.0577
13	13	11	Y	0.3	0.9	0.6	2.0000	0.0000	1.1322
14	13	11	N	0.4	0.5	0.1	2.0000	0.0000	1.0000
15	13	11	N	0.4	0.6	0.2	2.0000	0.0000	1.0000
16	13	11	N	0.4	0.7	0.3	2.0000	0.0000	1.0000
17	13	11	N	0.4	0.8	0.4	2.0000	0.0000	1.0000
18	13	11	N	0.4	0.9	0.5	2.0000	0.0000	1.0000
19	13	11	N	0.5	0.6	0.1	2.0000	0.0000	1.0000
20	13	11	N	0.5	0.7	0.2	2.0000	0.0000	1.0000
21	13	11	N	0.5	0.8	0.3	2.0000	0.0000	1.0000
22	13	11	N	0.5	0.9	0.4	2.0000	0.0000	1.0000
23	13	11	N	0.6	0.7	0.1	2.0000	0.0000	1.0000
24	13	11	N	0.6	0.8	0.2	2.0000	0.0000	1.0000
25	13	11	N	0.6	0.9	0.3	2.0000	0.0000	1.0000

Table A.1 continued

Trial	# Cells Upper	# Cells Lower	Premature peak?	Upper C	Lower C	ΔC	Lower Slope	ΔSlope	Q_{peak} / Q_{end}
26	13	11	N	0.7	0.8	0.1	2.0000	0.0000	1.0000
27	13	11	N	0.7	0.9	0.2	2.0000	0.0000	1.0000
28	13	11	N	0.8	0.9	0.1	2.0000	0.0000	1.0000
29	12	12	N	0.2	0.4	0.2	2.0000	0.0000	1.0000
30	11	13	N	0.2	0.4	0.2	2.0000	0.0000	1.0000
31	10	14	N	0.2	0.4	0.2	2.0000	0.0000	1.0000
32	9	15	N	0.2	0.4	0.2	2.0000	0.0000	1.0000
33	8	16	N	0.2	0.4	0.2	2.0000	0.0000	1.0000
34	7	17	N	0.2	0.4	0.2	2.0000	0.0000	1.0000
35	6	18	N	0.2	0.4	0.2	2.0000	0.0000	1.0000
36	5	19	N	0.2	0.4	0.2	2.0000	0.0000	1.0000
37	4	20	N	0.2	0.4	0.2	2.0000	0.0000	1.0000
38	3	21	N	0.2	0.4	0.2	2.0000	0.0000	1.0000
39	2	22	Y	0.2	0.4	0.2	2.0000	0.0000	1.0158
40	1	23	Y	0.2	0.4	0.2	2.0000	0.0000	1.0393
41	12	12	N	0.3	0.6	0.3	2.0000	0.0000	1.0000
42	11	13	N	0.3	0.6	0.3	2.0000	0.0000	1.0000
43	10	14	N	0.3	0.6	0.3	2.0000	0.0000	1.0000
44	9	15	N	0.3	0.6	0.3	2.0000	0.0000	1.0000
45	8	16	N	0.3	0.6	0.3	2.0000	0.0000	1.0000
46	7	17	N	0.3	0.6	0.3	2.0000	0.0000	1.0000
47	6	18	N	0.3	0.6	0.3	2.0000	0.0000	1.0000
48	5	19	Y	0.3	0.6	0.3	2.0000	0.0000	1.0185
49	4	20	N	0.3	0.6	0.3	2.0000	0.0000	1.0000
50	3	21	Y	0.3	0.6	0.3	2.0000	0.0000	1.0145
51	2	22	Y	0.3	0.6	0.3	2.0000	0.0000	1.0397
52	1	23	Y	0.3	0.6	0.3	2.0000	0.0000	1.0638
53	12	12	N	0.4	0.9	0.5	2.0000	0.0000	1.0000
54	11	13	N	0.4	0.9	0.5	2.0000	0.0000	1.0000
55	10	14	N	0.4	0.9	0.5	2.0000	0.0000	1.0000
56	9	15	N	0.4	0.9	0.5	2.0000	0.0000	1.0000
57	8	16	N	0.4	0.9	0.5	2.0000	0.0000	1.0000
58	7	17	N	0.4	0.9	0.5	2.0000	0.0000	1.0000
59	6	18	Y	0.4	0.9	0.5	2.0000	0.0000	1.0453
60	5	19	Y	0.4	0.9	0.5	2.0000	0.0000	1.0731

Table A.1 continued

Trial	# Cells Upper	# Cells Lower	Premature peak?	Upper C	Lower C	ΔC	Lower Slope	ΔSlope	Q_{peak} / Q_{end}
61	4	20	Y	0.4	0.9	0.5	2.0000	0.0000	1.0119
62	3	21	Y	0.4	0.9	0.5	2.0000	0.0000	1.0360
63	13	11	N	0.2	0.3	0.1	4.0000	2.0000	1.0000
64	13	11	Y	0.2	0.4	0.2	4.0000	2.0000	1.0085
65	13	11	Y	0.2	0.5	0.3	4.0000	2.0000	1.1313
66	13	11	Y	0.2	0.6	0.4	4.0000	2.0000	1.2431
67	13	11	Y	0.2	0.7	0.5	4.0000	2.0000	1.3494
68	13	11	Y	0.2	0.8	0.6	4.0000	2.0000	1.3764
69	13	11	Y	0.2	0.9	0.7	4.0000	2.0000	1.4737
70	13	11	Y	0.2	0.3	0.1	4.0000	3.0000	1.0097
71	13	11	Y	0.2	0.4	0.2	4.0000	3.0000	1.1912
72	13	11	Y	0.2	0.5	0.3	4.0000	3.0000	1.3553
73	13	11	Y	0.2	0.6	0.4	4.0000	3.0000	1.5131
74	13	11	Y	0.2	0.7	0.5	4.0000	3.0000	1.5584
75	13	11	Y	0.2	0.8	0.6	4.0000	3.0000	1.7067
76	13	11	Y	0.2	0.9	0.7	4.0000	3.0000	1.7331
77	13	11	Y	0.2	0.3	0.1	5.0000	4.0000	1.0657
78	13	11	Y	0.2	0.4	0.2	5.0000	4.0000	1.2600
79	13	11	Y	0.2	0.5	0.3	5.0000	4.0000	1.3435
80	13	11	Y	0.2	0.6	0.4	5.0000	4.0000	1.4998
81	13	11	Y	0.2	0.7	0.5	5.0000	4.0000	1.6573
82	13	11	Y	0.2	0.8	0.6	5.0000	4.0000	1.6931
83	13	11	Y	0.2	0.9	0.7	5.0000	4.0000	1.8494
84	13	11	N	0.5	0.9	0.4	3.0000	1.0000	1.0000
85	13	11	N	0.5	0.9	0.4	3.5000	1.5000	1.0000
86	13	11	N	0.5	0.9	0.4	4.0000	2.0000	1.0000
87	13	11	N	0.5	0.9	0.4	4.5000	2.5000	1.0000
88	13	11	N	0.5	0.9	0.4	5.0000	3.0000	1.0000
89	13	11	N	0.5	0.9	0.4	3.0000	2.0000	1.0000
90	13	11	Y	0.5	0.9	0.4	3.5000	2.5000	1.0223
91	13	11	Y	0.5	0.9	0.4	4.0000	3.0000	1.0166
92	13	11	Y	0.5	0.9	0.4	4.5000	3.5000	1.0117
93	13	11	Y	0.5	0.9	0.4	5.0000	4.0000	1.5540
94	13	11	Y	0.5	0.9	0.4	3.0000	2.5000	1.1808
95	13	11	Y	0.5	0.9	0.4	3.5000	3.0000	1.1739

Table A.1 continued

Trial	# Cells Upper	# Cells Lower	Premature peak?	Upper C	Lower C	ΔC	Lower Slope	ΔSlope	Q_{peak} / Q_{end}
96	13	11	Y	0.5	0.9	0.4	4.0000	3.5000	1.2420
97	13	11	Y	0.5	0.9	0.4	4.5000	4.0000	1.2364
98	13	11	Y	0.5	0.9	0.4	5.0000	4.5000	1.2317

Table A.2 Calibration Data: 75-cell trials

Trial	# Cells Upper	# Cells Lower	Premature peak?	Upper C	Lower C	ΔC	Lower Slope	ΔSlope	Q_{peak} / Q_{end}
1	40	35	Y	0.2	0.9	0.7	5	4	2.4343
2	40	35	Y	0.2	0.2	0	2	0	1.0473
3	40	35	Y	0.2	0.3	0.1	2	0	1.0665
4	40	35	Y	0.2	0.4	0.2	2	0	1.1846
5	40	35	Y	0.2	0.5	0.3	2	0	1.3362
6	40	35	Y	0.2	0.6	0.4	2	0	1.4777
7	40	35	Y	0.2	0.7	0.5	2	0	1.5194
8	40	35	Y	0.2	0.8	0.6	2	0	1.6466
9	40	35	Y	0.2	0.9	0.7	2	0	1.7775
10	40	35	Y	0.3	0.3	0	2	0	1.0085
11	40	35	Y	0.3	0.4	0.1	2	0	1.0482
12	40	35	Y	0.3	0.5	0.2	2	0	1.0689
13	40	35	Y	0.3	0.6	0.3	2	0	1.1523
14	40	35	Y	0.3	0.7	0.4	2	0	1.2412
15	40	35	Y	0.3	0.8	0.5	2	0	1.3422
16	40	35	Y	0.3	0.9	0.6	2	0	1.4484
17	40	35	Y	0.4	0.4	0	2	0	1.0154
18	40	35	Y	0.4	0.5	0.1	2	0	1.0418
19	40	35	Y	0.4	0.6	0.2	2	0	1.0449
20	40	35	Y	0.4	0.7	0.3	2	0	1.0815
21	40	35	Y	0.4	0.8	0.4	2	0	1.1243
22	40	35	Y	0.4	0.9	0.5	2	0	1.2141
23	40	35	Y	0.5	0.5	0	2	0	1.0027
24	40	35	Y	0.5	0.6	0.1	2	0	1.0361
25	40	35	Y	0.5	0.7	0.2	2	0	1.0384

Table A.2 continued

Trial	# Cells Upper	# Cells Lower	Premature peak?	Upper C	Lower C	ΔC	Lower Slope	ΔSlope	Q_{peak} / Q_{end}
26	40	35	Y	0.5	0.8	0.3	2	0	1.0401
27	40	35	Y	0.5	0.9	0.4	2	0	1.0676
28	40	35	N	0.6	0.6	0	2	0	1.0000
29	40	35	Y	0.6	0.7	0.1	2	0	1.0308
30	40	35	Y	0.6	0.8	0.2	2	0	1.0324
31	40	35	Y	0.6	0.9	0.3	2	0	1.0337
32	40	35	N	0.7	0.7	0	2	0	1.0000
33	40	35	Y	0.7	0.8	0.1	2	0	1.0256
34	40	35	Y	0.7	0.9	0.2	2	0	1.0267
35	40	35	N	0.8	0.8	0	2	0	1.0000
36	40	35	N	0.8	0.9	0.1	2	0	1.0000
37	40	35	N	0.9	0.9	0	2	0	1.0000
38	40	35	Y	0.2	0.3	0.1	5	4	1.3266
39	40	35	Y	0.2	0.4	0.2	5	4	1.5844
40	40	35	Y	0.2	0.5	0.3	5	4	1.8299
41	40	35	Y	0.2	0.6	0.4	5	4	1.9097
42	40	35	Y	0.2	0.7	0.5	5	4	2.1421
43	40	35	Y	0.2	0.8	0.6	5	4	2.1895
44	40	35	Y	0.2	0.3	0.1	4	3	1.3379
45	40	35	Y	0.2	0.4	0.2	4	3	1.4803
46	40	35	Y	0.2	0.5	0.3	4	3	1.7021
47	40	35	Y	0.2	0.6	0.4	4	3	1.9246
48	40	35	Y	0.2	0.7	0.5	4	3	1.9830
49	40	35	Y	0.2	0.8	0.6	4	3	2.2048
50	40	35	Y	0.2	0.9	0.7	4	3	2.2403
51	40	35	Y	0.2	0.3	0.1	3	2	1.2571
52	40	35	Y	0.2	0.4	0.2	3	2	1.4961
53	40	35	Y	0.2	0.5	0.3	3	2	1.5941
54	40	35	Y	0.2	0.6	0.4	3	2	1.7945
55	40	35	Y	0.2	0.7	0.5	3	2	2.0026
56	40	35	Y	0.2	0.8	0.6	3	2	2.0455
57	40	35	Y	0.2	0.9	0.7	3	2	2.2603
58	40	35	Y	0.2	0.2	0	3	2	1.0781

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