A Computerized Master Plan of Drainage, I: Development

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Abstract

Since the development and availability of low cost microcomputers, the last half decade has seen a rapid growth in the demand for computers in municipal flood control system analysis. In this paper, a computerized master plan of drainage software system is developed for use on microcomputers. The data entry requirements are presented in a user-friendly format which enables the program user to become productive immediately.

Keywords

Water Resources; Hydrologic Modeling; Master Plans of Drainage; Urban Drainage

Rational Method Equation

The most widely used hydrologic model for estimating watershed peak runoff rates is the rational method. This approach is frequently used to estimate runoff rates from small urban areas of variable size. Some older versions of this method are directly applied to watersheds with sizes in excess of several square miles. Modern versions of this approach generally limit the watershed size to about 1 square mile (Hromadka and Guymon, 1983b).

Basically, the rational method equation relates rainfall intensity, a runoff coefficient, and drainage area size to the direct peak runoff rate. This relationship is expressed by the equation

$$Q = CIA \qquad (1)$$

where

- Q = the peak runoff rate in cubic feet per second (cfs) at the point of concentration
- C = a runoff coefficient representing the area-averaged ratio of runoff to rainfall rates
- I = the time-averaged rainfall intensity in inches per hour corresponding to the time of concentration
- A = the drainage area (acres)

The values of the runoff coefficient and rainfall intensity are based on a study of drainage area characteristics such as type and condition of the runoff surfaces and the time of concentration. These factors and the limitations of the rational method equation are discussed in the following sections. The drainage area may be determined by planimetering a suitable topographic map of the tributary watershed area.

Data required for the computation of peak discharge by the rational method include (1) rainfall intensity for a storm of specified duration and selected return frequency; (2) drainage area characteristics of size, shape, slope; and (3) an estimate of rainfall-runoff relationships.

The duration of the storm rainfall required in the rational method equation is based on the time of concentration of the tributary drainage area. The time of concentration (Tc) is usually defined as the duration (in minutes) required for runoff at the point of concentration to become a maximum under a uniform and constant rainfall intensity. This occurs when all parts of the drainage area are contributing to the flow. Generally, the time of concentration is the interval of time from the beginning of rainfall for water from the hydraulically most remote point of the drainage area to reach the point of concentration (e.g. a drainage structure). The Tc is a function of several variables including the length of the flow path from the most remote point of the watershed to the concentration point, the slope of the flowpath, characteristics of natural and improved channels within the drainage area, the infiltration properties of the soil, and the extent and type of development.

Rainfall intensity (I) is determined from the local precipitation intensity-duration curves of the desired return frequency (see Fig. 1). Intensity duration curves for a particular region can be developed using the log-log paper of Fig. 1, plotting the area-averaged point rainfall value for the one hour duration, and drawing a straight line through the one hour value with a slope based on shorter duration rainfall intensity values.

The runoff coefficient (C) is the ratio of peak rate of runoff to the rate of rainfall at an average intensity when the total drainage area is contributing runoff to the point of concentration. The selection of the runoff coefficient depends on rainfall intensity, drainage area slope, type and amount of vegetative cover, distribution and magnitude of the soil infiltration capacity, and various other factors. Since one acre-inch/hour is equal to 1.008 cfs, the rational method is generally assumed to estimate a peak flowrate in cfs. For urban design studies, the runoff coefficient is usually assumed to be a function of the impervious and pervious area fractions, a characteristic infiltration rate (F_p) for the pervious area fraction, and the effects of watershed detention in the estimation of traveltime of the peak runoff rate through the watershed channel system.

Limitations of the Rational Method

The validity of the relationship expressed by the rational method equation holds true only if certain assumptions are reasonably correct and limitations are observed. Three basic assumptions are that (1) the frequency of the storm runoff is the same as the return frequency of rainfall producing the runoff (that is, a 25 year recurrence interval rainfall will result in a 25 year recurrence interval storm runoff); (2) the peak runoff rate occurs when all parts of the drainage area are contributing to the runoff; and (3) the design rainfall is uniform over the watershed area tributary to the point of concentration, and the intensity is essentially constant during the storm duration equal to the time of concentration.

The rational method equation is only applicable where the rainfall intensity can be assumed uniformly distributed over the drainage area at a uniform rate throughout the storm duration. This assumption applies fairly well to small drainage areas of less than about one square mile. Beyond this limit, the rainfall distribution may vary considerably from the point values given in rainfall isohyetal maps.

The selection of the runoff coefficient is another major limitation for the method. For small urban areas, the runoff coefficient can be reasonably estimated from field investigations and studies of aerial photographs. For larger areas where the determination of the runoff coefficient is to be based on vegetation type, cover density, the infiltration capacity of the soil, and the slope of the drainage area, an estimate of the

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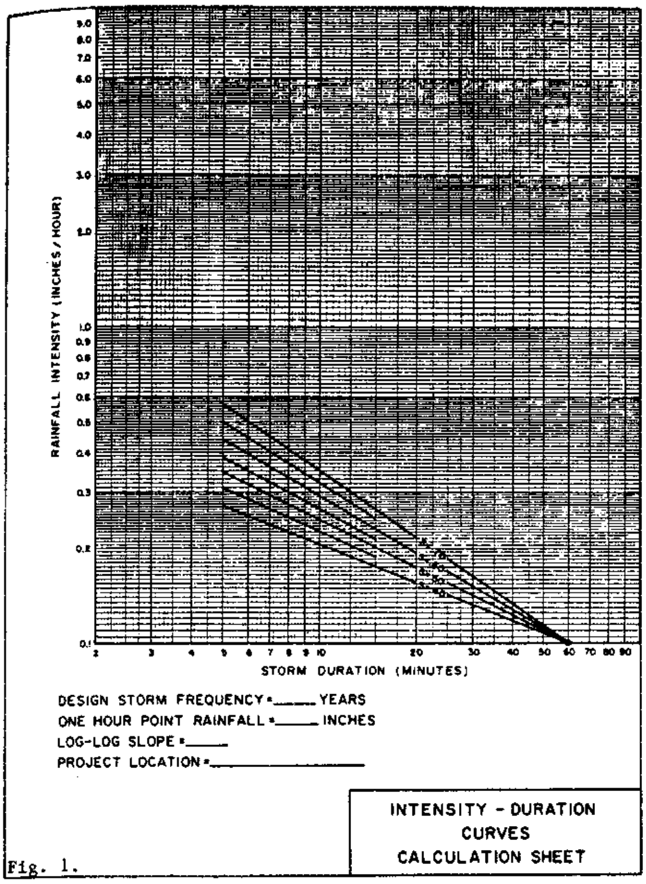


Fig. 1.

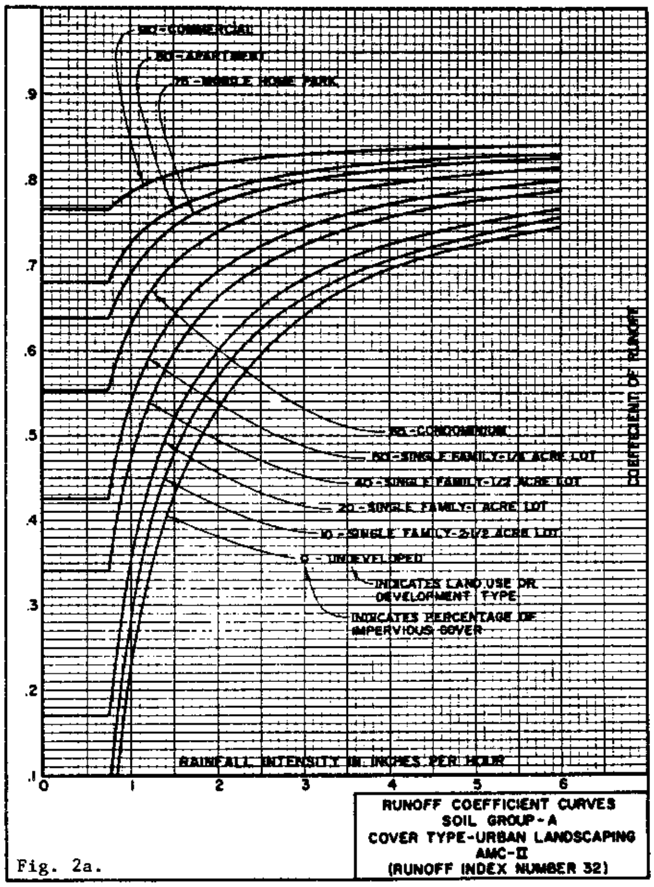


Fig. 2a.

runoff coefficient may be subject to a much greater error due to the variability of the drainage area characteristics. Rainfall losses due to evaporation, transpiration, depression and channel storage cannot be properly evaluated, and may appreciably affect the estimate of the watershed peak rate of runoff.

Runoff Coefficient

For calculation purposes, the runoff coefficient is defined to be either (1) a constant value depending on soil cover type and quality, or (2) a function of rainfall intensity, soil cover type and quality. Table 1, lists typical C values for direct substitution into the rational method equation.

TABLE 1. TYPICAL CONSTANT RUNOFF COEFFICIENTS
(County of San Diego Hydrology Manual, Calif., 1975)

Development Type and Cover	S.C.S. Soil Group A	B	C	D
single family residential	0.40	0.45	0.50	0.55
multi-unit residential	0.45	0.50	0.60	0.70
mobile home park	0.45	0.50	0.55	0.65
rural(lots greater than .25 acre)	0.30	0.35	0.40	0.45
commercial	0.70	0.75	0.80	0.85
industrial	0.80	0.85	0.90	0.95

The second class of runoff coefficient representations relate the C value to the rainfall intensity. One approach used for urban design purposes is to assume that the watershed loss rate is equal to the infiltration loss rate which corresponds to the limiting value of the infiltration capacity curve of the soil. For design storm conditions, it can be argued that the impervious area runoff rate is independent of the rainfall intensity and that the pervious area infiltration loss rate is a constant which is determined from emperical equations. Then estimates for runoff coefficients are developed using a relationship of the form

$$C = \left(A + \frac{(I - F_p)A_p}{I}\right) \qquad (2)$$

where

C = runoff coefficient
I = rainfall intensity (inch/hour)
F_p = infiltration rate for pervious area fraction
A_i = impervious area fraction
A_p = pervious area fraction

The infiltration rate for the pervious area (F_p) can be estimated for various combinations of soil type, cover, and antecedent moisture conditions and from rainfall-runoff data. For the most common types of urban development and soil covers, typical runoff coefficient curves based on (2) are shown in Figs. 2a through 2d for S.C.S. soil groups A through D, respectively.

When the drainage area is composed of several types of runoff surfaces, an area-averaged runoff coefficient can be developed as demonstrated by the following example problem.

Example 1, Area-Averaged Runoff Coefficient

The watershed is composed of 3.5 acres of parking lot pavement and associated street system. Also included is 35.6 acres of a condominium development and 12.5 acres of a neighboring apartment complex. The area-averaged runoff coefficient is estimated by tabulating each area fraction's contribution.

Area(acres)	Type of Surface	C	CA
3.5	concrete pavement	1.0	3.5
35.6	condominiums	.67	23.85
12.5	apartments	.77	9.63
51.6 (sum)			36.98

area-averaged C = 36.98/51.6 = 0.72

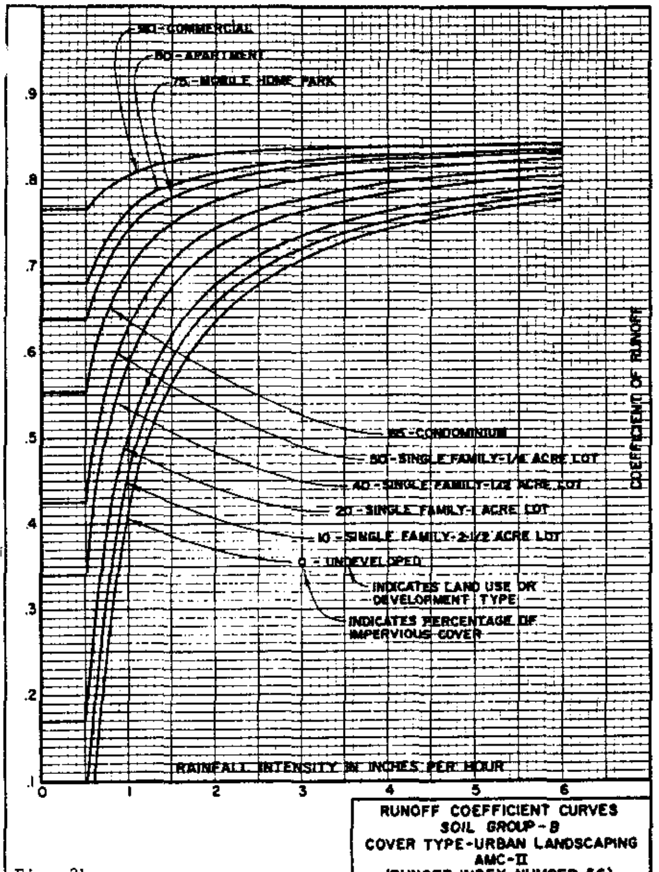


Fig. 2b.

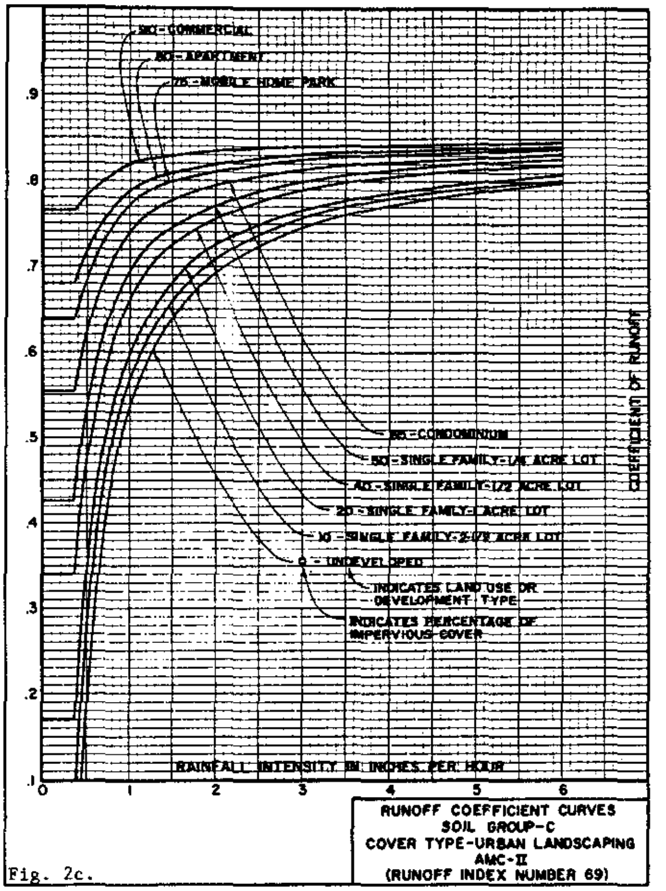


Fig. 2c.

Time of Concentration

For urban watersheds, the time of concentration (Tc) can be estimated by summing the runoff travel time (T_t) through the several flow regimes as the flood peak travels downstream to the watershed outlet or point of concentration. These flow regimes include overland flow, streetflow, pipeflow, and open channel flow in natural or improved channels, and must include the *effects of the flood peak rate (Q) increasing in magnitude as the* tributary area to the main collection stream increases. The general rational method procedure is to define the total watershed tributary to the point of concentration where the peak runoff rate is to be estimated. The main flowpath is identified such that the watershed can be subdivided into m subareas with each subarea tributary to the collection stream (see Figure 7).
From the figure, it is noted that the initial (the most upstream) subarea is relatively small (about 10 acres) and has an associated overland flowpath length of less than about 1000 feet. The subareas gradually increase in size in the downstream direction along the collection stream. Additionally, nodal points (i = 1, 2, ..., m) are defined along the collection stream so that each *subarea has an associated upstream and downstream node number.* The initial subarea time of concentration for the overland flow between nodes number 1 and 2 is estimated by some overland flow formula or an assumed average flow velocity for the runoff traveling along the main flowpath within the initial subarea. Subsequent T_c values are determined by

$$Tc(i+1) = Tc(i) + T_t(i+1) \qquad (3)$$

where

Tc(2) = initial subarea time of concentration (minutes)
Tc(i) = Tc at node number i
$T_t(i+1)$ = traveltime of Q between nodes i and i+1

To estimate the traveltime values $T_t(i+1)$, the Manning's formula is used to calculate a normal depth for the runoff flowing in the channel linking nodes i and i+1 and the corresponding flow velocity is used to estimate the time for the peak Q to move from node i to node i+1. Then

$$T_t(i+1) = L(i+1)/V(i+1) \qquad (4)$$

where

L(i+1) = length of channel linking nodes i and i+1
V(i+1) = normal depth flow velocity for Q(i)
Q(i) = peak runoff rate concentrated at node i

Over two dozen overland flow formulas have been proposed in the literature to estimate Tc(2). A variation of the Kirpich (1940) formula which is widely used throughout Southern California has the form

$$Tc = k(L^3/H)^E \qquad (5)$$

where

L = length of initial subarea flowpath (feet)
H = drop in elevation along flowpath (feet)
k = coefficient depending on development type and other factors
E = constant exponent, usually 0.385

A nomograph for the solution of (5) with E=0.20 is given in Fig. 4. Due to the inherent inaccuracy in the determination of a generalized overland flow Tc, (5) should be used on initial *subareas of less than about 10 acres.*

In comparison, the Federal Aviation Agency (1970) proposes

$$Tc = 1.8(1.1 - C)L^{0.50}S^{-.0333} \qquad (6)$$

where

C = runoff coefficient
S = average surface slope (percent)

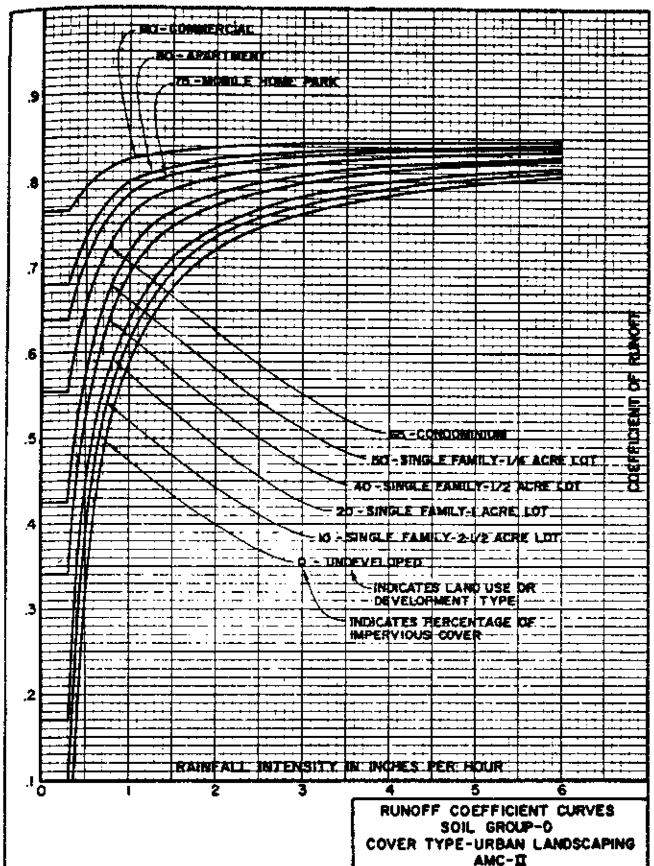


Fig. 2d.

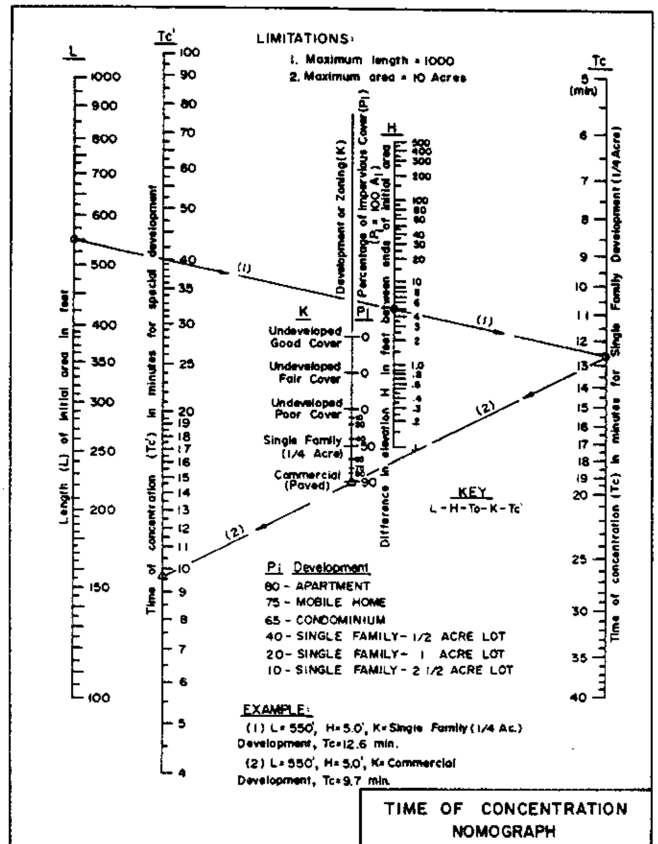


Fig. 3.

Rational Method Modeling Approaches

A review of frequently used rational method approaches for analysis of urban watersheds indicates that a great majority of these techniques fall into four categories:

NOMOGRAPH MODEL. The simplest rational method approach is to estimate a Tc at the point of concentration by using a generalized overland flow formula or corresponding nomograph. For example, Fig. 4 provides a nomograph for the California Culvert Practice (1942) formula of (5) with k = 0.0078. After estimating a Tc, an average rainfall intensity (I) and corresponding runoff coefficient (C) are determined, and peak discharge is computed from the equation Q = CIA.

TIME OF CONCENTRATION MODEL. This approach estimates the peak runoff at a watershed point of concentration by the following algorithm steps:

(1). Subdivide the watershed into m subareas such as shown in Fig. 7. The subareas are chosen such that the initial subarea is relatively small and subsequent subareas gradually increase in size in the downstream direction. Each subarea has an associated runoff coefficient function C_i, and a tributary drainage area A_i.

(2). Estimate a time of concentration Tc' at the point of concentration of the watershed (that is, the most downstream nodal point, node m+1).

(3). Using Tc', determine a corresponding rainfall intensity (I') from the local precipitation intensity-duration curves.

(4). If the C_i are assumed to be functions of rainfall intensity, determine appropriate C_i' values for the intensity I'.

(5). Calculate a total watershed peak runoff (Q') by

$$Q' = (C_1'A_1 + C_2'A_2 + \dots + C_m'A_m)I'$$

(6). Distribute Q' throughout the watershed according to the area proportion of runoff $Q_i' = C_i'A_iI'$, where Q_i' is the assumed runoff estimate at nodal point i in the estimation of the peak Q for node m+1.

(7). Estimate the time of concentration for the initial subarea, Tc(2). Generally, a nomograph is used or an overland flow velocity (such as 3 feet/second) is assumed.

(8). In the next downstream subarea, calculate the traveltime $T_t(3)$ for the runoff Q_2' to flow to the next nodal point and determine $Tc(3) = Tc(2) + T_t(3)$.

(9). In each subsequent downstream subarea, use Q_i' to estimate the traveltime $T_t(i+1)$ between nodes i and i+1, and estimate $Tc(i+1) = Tc(i) + T_t(i+1)$.

(10). Using step 9, determine the final subarea Tc(m+1).

(11). Compare Tc(m+1) to the estimated Tc'.

(12). Calculate a new Tc'' by Tc'' = (Tc(m+1) + Tc')/2.

(13). Return to step 3 where Tc'' is substituted for Tc'.

This rational method link-node model attempts to estimate a Tc(m+1) by accounting for the several flow regimes through which the runoff peak flow rate must travel through, and provides a considerable improvement over the nomograph method when studying larger watersheds.

SUBAREA MODEL. This approach attempts to reduce the calculation effort required by the previous Time of Concentration Model iteration method. The procedure for the Subarea Model is as follows:

(1). Subdivide the watershed into m subareas such as shown in Fig. 7. Similar to the Time of Concentration Model, the subareas are selected such that the initial subarea is less than 10 acres and the subsequent downstream subareas gradually increase in size in order to reduce the computational effort in dealing with small subareas.

(2). Estimate an initial subarea Tc(2) for the overland flow between nodes 1 and 2.

(3). Using Tc(2), estimate the corresponding rainfall intensity I_2 = I(Tc(2)) and the runoff coefficient C_2. Then $Q(2) = C_2I_2A_1$.

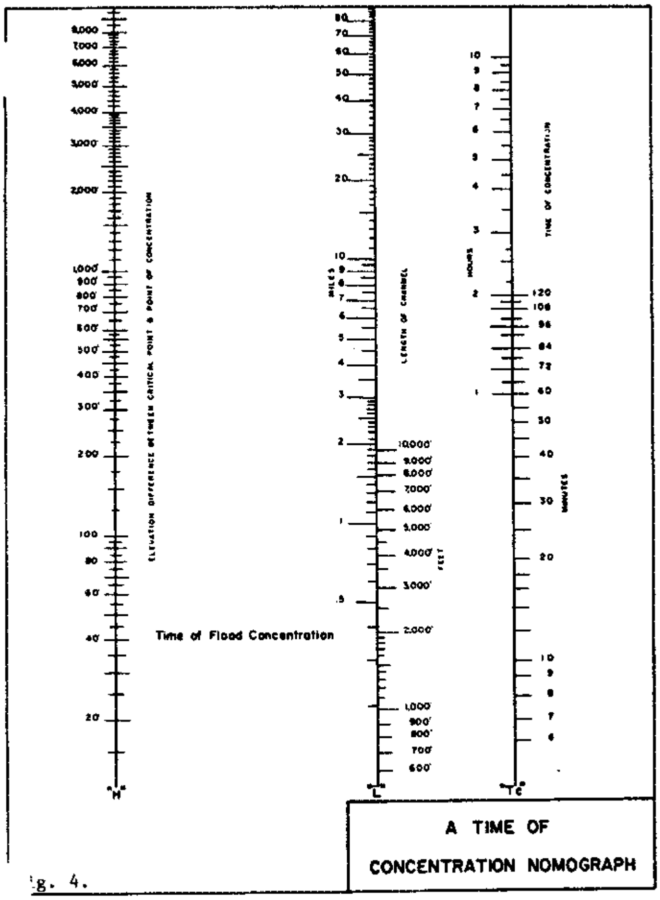


Fig. 4.

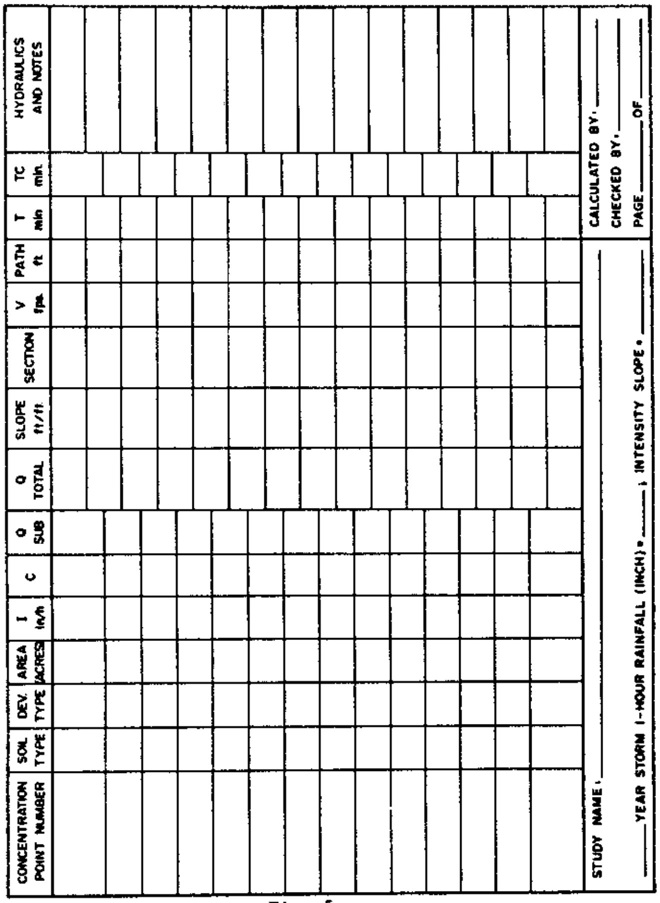


Fig. 5.

(4). Using Q(2), estimate the traveltime $T_t(3)$ between nodes 2 and 3 of the next downstream subarea.

(5). Calculate $Tc(3) = Tc(2) + T_t(3)$. Determine the rainfall intensity $I_3 = I(Tc(3))$. Using I_3, determine an area-averaged runoff factor for the entire watershed tributary to node 3 by

$$(CA)_3 = (C_2A_1 + C_3A_2)$$

Then $Q(3) = (CA)_3 I_3$.

(6). Repeat steps 4 and 5 for each subsequent downstream subarea as the study proceeds in the downstream direction. At each node, the area-averaged runoff factor $(CA)_i$ is calculated based on the new Tc and I values.

A computational advantage of the Subarea Model over the Time of Concentration Model is that the entire watershed is analyzed for peak flow rate estimates at each watershed nodal point with only one pass of the method. Consequently, computational effort is considerably reduced. A disadvantage of the model is that it is possible to estimate a downstream peak runoff rate (at a node i+1) which is less than the preceeding nodal point (at node i). This is due to the variation of the runoff factor $(CA)_i$ with rainfall intensity and is easily accomodated by some smoothing of computed results.

SUBAREA SUMMATION MODEL. This rational method modeling approach is widely used due to its simplicity in application, and the capability for estimating peak runoff rates throughout the interior of a study watershed analogous to the Subarea Model. The procedure for the Subarea Summation Model is as follows:

(1). Subdivide the watershed into subareas with the initial subarea being approximately 10 acres in size, and the subsequent subareas gradually increasing in size. Assign upstream and downstream nodal point numbers to each subarea in order to correlate calculations to the watershed map (see Fig. 7).

(2). Estimate a Tc(2) by using a nomograph or overland flow velocity estimation.

(3). Using Tc(2), determine the corresponding values of I_2 and C_2. Then $Q(2) = C_2I_2A_1$.

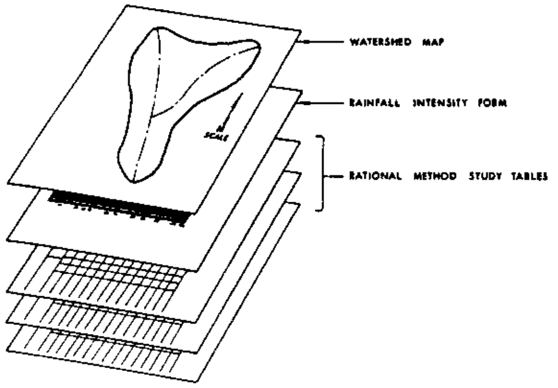
(4). Using Q(2), estimate the traveltime between nodes 2 and 3 by Manning's equation as applied to the particular channel or conduit linking nodes 2 and 3.

(5). Then $Tc(3) = Tc(2) + T_t(3)$. Using Tc(3), estimate the rainfall intensity $I_3 = I(Tc(3))$ and the runoff coefficient corresponding to both I_3 and the properties of the subarea between nodes 2 and 3. Then

$$Q(3) = Q(2) + C_3I_3A_2$$

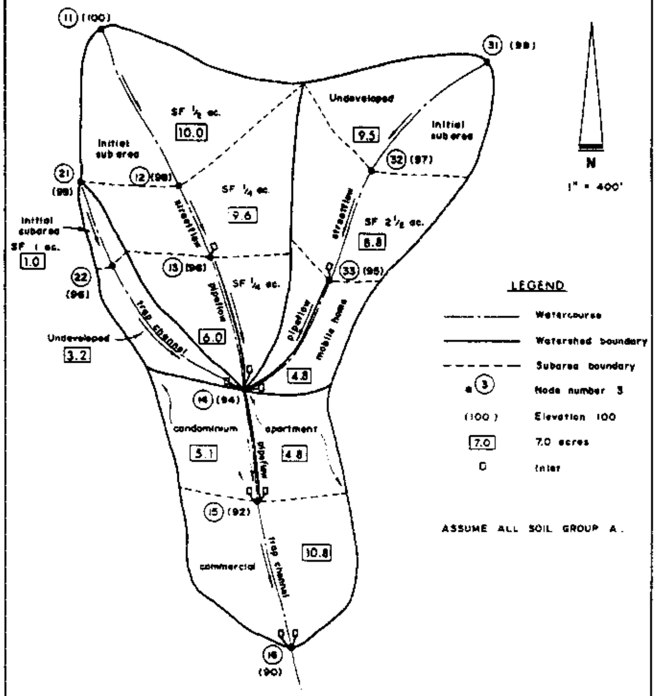
(6). Repeat steps 4 and 5 as the analysis proceeds in the downstream direction along the principal collection stream.

Of the subarea models, the Subarea Summation Model is generally the easiest to use and formulate into a digital computer program. Computer applications of this model include the master plannings of several large cities in the urbanized regions of Southern California using the computer capacity of currently available small microcomputers (Hromadka et al., 1983c). Because the calculations proceed in the downstream direction exclusively, the entire watershed tributary to each nodal point is characterized by only three variables: Q(i), Tc(i), and total area. The Subarea Model is also easily programmable for use in master planning and design purposes. The Time of Concentration Model, however, involves considerably more computational effort and computer memory allocation, and is generally restricted for use on the larger mini- or mainframe computer systems.



TYPICAL REPORT FORMAT FOR RATIONAL METHOD STUDY

Fig. 6.



EXAMPLE PROBLEM DRAINAGE SYSTEM

Fig. 7.

Confluence of Streams (Junction Analysis)

Each of the above rational method modeling approaches determines peak runoff rates for major collection streams within a watershed. At the confluence of two or more collection streams, a procedure for adjusting the total summation of peak flow rates is required in order to account for each stream's time of concentration at the junction. The following procedure provides an estimate of the confluence peak flow rate assuming that each stream's runoff hydrograph is triangular in shape (Riverside County Hydrology Manual, 1978).

Let Q_a, T_a, I_a be the peak runoff flowrate, time of concentration, and rainfall intensity that corresponds to the collection stream with the longer time of concentration. Let Q_b, T_b, I_b correspond to the collection stream with the shorter time of concentration. Let Q_p and T_p correspond to the confluence peak Q and time of concentration, respectively. Then the following situations are possible:

(1). If the collection streams have the same time of concentration, then the Q values are directly summed,

$$Q_p = Q_a + Q_b; \quad T_p = T_a = T_b$$

(2). If the collections streams have different times of concentration, the smaller of the tributary Q values may be adjusted as follows:

(i) The most frequent case is where the collection stream with the longer time of concentration has the larger Q. Then the smaller Q value is adjusted by the ratio of rainfall intensities

$$Q_p = Q_a + Q_b(I_a/I_b); \quad T_p = T_a$$

(ii) In some cases, the collection stream with the shorter time of concentration has the larger Q. Then the smaller Q is adjusted by a ratio of the Tc values

$$Q_p = Q_b + Q_a(T_b/T_a); \quad T_p = T_b$$

Presentation of Product

Of interest to many civil engineers in both the private and public sectors is a standardized means of presenting rational method calculations. Many local governmental flood control agencies at the city and county levels use a standard tabulation form such as shown in Fig. 5. Consequently, a computer program prepared to perform master planning and design studies within a flood control district should be designed to produce a product which exactly satisfies the local agency requirements for study submittals. A typical study format is shown in Fig. 6 which includes the necessary hydrologic and geographic data requirements as well as the overall presentation of the product.

References

California Culverts Practice, (1942); Formula is essentially the Kirpich equation; developed from small mountainous basins in California; U.S. Bureau of Reclamation, 1973, pp. 67-71.

County of Riverside, "Hydrology Manual", Riverside, Ca. 1978.

County of San Diego, "Hydrology Manual", San Diego, Ca. 1975.

Federal Aviation Agency, "Dept. of Trans. Adv. Circular on Airport Drainage", Rep. A/C 050-5320-5B, Washington, D.C., (1970).

Hromadka II, T.V., and Guymon, G.L., "San Bernardino County Hydrology Manual", County of San Bernardino, Ca., (1983b).

Hromadka II, T.V., Clements, J.C., and Guymon, G.L., "Guidelines for Interactive Software in Water Resources Engineering", Water Resources Bulletin, February, (1983c).

Kirpich, Z.P., "Time of Concentration of Small Agricultural Watersheds", Civ. Eng., 10(6), (1940).