FINAL REPORT

DETERMINATION OF RAINFALL LOSSES IN VIRGINIA: THE EFFECTS OF URBANIZATION

by

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Virginia Highway & Transportation Research Council
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SUMMARY

The effects of urbanization on the Corps of Engineers' HEC-1 rainfall-runoff model parameters were examined. Data on rainfall events and corresponding streamflow hydrographs were gathered for five watersheds in rural and highly urbanized areas in Virginia. These data were used in the HEC-1 program to obtain optimal estimates for the loss rate, unit hydrograph, and runoff hydrograph parameters. Regression analyses were then performed to derive prediction equations for these parameters in terms of the percent imperviousness of a watershed, which was used as the index of urbanization.

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INTRODUCTION

In an earlier study entitled "Determination of Rainfall Losses in Virginia, Phase II" (Cruise and Yu 1982), optimal rainfall loss and hydrograph parameters were determined for 28 watersheds scattered in 7 hydrologic regions throughout the state of Virginia. These loss rate and hydrograph parameters were computed for 160 storm events and corresponding streamflow data using the Corps of Engineers' HEC-1 hydrologic simulation program. Parameter selection curves were then developed for each hydrologic region relating these parameters to watershed characteristics such as drainage area, length of main channel, and average slope. These "localized" parameters are used to make accurate estimates of the amount of runoff from storm events.

The present study, suggested by engineers with the Location and Design Division of the Virginia Department of Highways and Transportation, concentrated on the effects of urbanization on the loss rate and hydrograph parameters. Urbanization was not considered in the earlier study as the watersheds were mostly rural or forested. Since many current highway construction activities are in urban or urbanizing areas, it is imperative to quantify the effects of urbanization on the hydrologic response of a watershed so that adequate drainage can be provided for areas with projected urban development.

OBJECTIVE AND SCOPE OF WORK

The objective of this study was to examine the effects of urbanization on rainfall loss and hydrograph parameters for selected watersheds in Virginia using the Corps of Engineers HEC-1 computer program. The HEC-1 rainfall-runoff model is used by the Department of Highways and Transportation to determine design peak discharges. It is expected the input parameters developed will allow a highway engineer to estimate storm runoff from a given design even when using the HEC-1 model for urban watersheds.

Major work elements in this project were –

1. selecting test watersheds and taking an inventory of all test data;
2. determining an index for the degree of urbanization; and
3. evaluating the effects of urbanization on loss rate and hydrograph parameters.

Presented in this report are:

1. A review of the effects of urbanization on storm runoff dynamics.
2. The HEC-1 computer program and the parameters studied.
3. The procedure for HEC-1 parameter optimization and the collection of the HEC-1 input data.
4. Results, discussions, and recommendations.

EFFECTS OF URBANIZATION ON STORM RUNOFF DYNAMICS

An urban or urbanizing watershed can be defined as an area in which all or part of the watershed will be covered by impervious structures such as roads, sidewalks, parking lots, and houses. Urban stream channels may also be supplemented by some form of artificial drainage system such as paved gutters and storm sewers (Soil Conservation Service 1975).

The urbanization of a watershed changes its hydrologic response to precipitation and hence changes the dynamics of storm runoff. Peak discharges generally increase with urbanization. Not only is infiltration, or rainfall loss, reduced, but lag time, or the time of response of runoff to precipitation, is usually decreased as increasingly larger percentages of the basin are made impervious and as drainage channels are lined, paved, or replaced by pipes. The net result is an increase in the concentration of storm runoff in a drainage system and an increase in peak flows. The degree to which peak flows are increased is also dependent on the manner in which runoff from impervious surfaces reaches the drainage channels or collector systems.

The following is a discussion on the effect of urbanization on the two most important hydrograph parameters runoff volume (peak discharge rate) and watershed lag time (Soil Conservation Service 1975).

Factors Affecting Runoff Volume

Soil Type

Since urban areas are seldom completely covered by impervious structures, soil properties are an important factor in estimating the total volume of direct runoff. The infiltration and percolation rates of soils indicate their potential to absorb rainfall and thereby reduce the amount of direct runoff. Soils having a high infiltration rate (sands or gravels) have a low runoff potential, and soils having a low infiltration rate (clays) have a high runoff potential. Urbanization on soils with a high infiltration rate increases the volume of runoff and peak discharge more than urbanization on soils with a low infiltration rate.

Cover Type

The type of cover and its hydrologic condition affects runoff volume through its influence on the infiltration rate of the soil. Fallow land yields more runoff than forested land for a given soil type. Covering areas with impervious material reduces surface storage and infiltration and increases the volume of runoff.

Figure 1 illustrates the effect of imperviousness, usually used as an index of urbanization, on the storm runoff rate and lag time. The watershed, Kimages Creek near Richmond, Virginia, was assumed to have imperviousness ranging from 0 to 80%. A storm event occurring on February 27, 1977, was analyzed using the Corps of Engineers" computer model STORM. The resulting hydrographs clearly showed how imperviousness would cause a significant increase in runoff rate and decrease in lag time.

Most investigators agree that the percent of imperviousness, although permanently affecting the basin hydrology, does not have the same effect on the rainfall available for runoff for all storm events. As storm intensities and magnitudes increase, the percentage of rainfall that infiltrates into the ground, is trapped in surface depressions, or is lost by evaporation becomes less and less (even for a rural basin), until the amount of loss has little or no effect on the volume of rainfall available for runoff. This implies that at some point in the flood frequency distribution there is virtually no difference between an urban and a rural flood magnitude. However, in general, the improvements in the hydraulic efficiency of a drainage system remain in effect and continue to speed the runoff in a stream. This means that the bulk of the runoff passes a point in a stream in a shorter period of time. The rate of flow, and consequently the peak flow, is appreciably higher than that for an undeveloped condition.

The initial abstraction, i.e., the sum of interception, depression storage, and infiltration before runoff begins, occurs on all types of cover, from pasture in good condition to concrete pavement. However, the amount of initial abstraction is less on concrete pavement than on pasture. Therefore, it is expected that the rainfall loss parameters would decrease as urbanization increases.

Factors Affecting Lag Time

Slope

Urbanization can change the effective slope of a watershed if flow paths are altered by channelizing and by terracing areas for building lots, parking lots, roads, and diversion ditches. The slopes of storm sewers, street gutters, roads, and overland flow areas, as well as stream channels, are significant in determining travel times through urban watersheds.

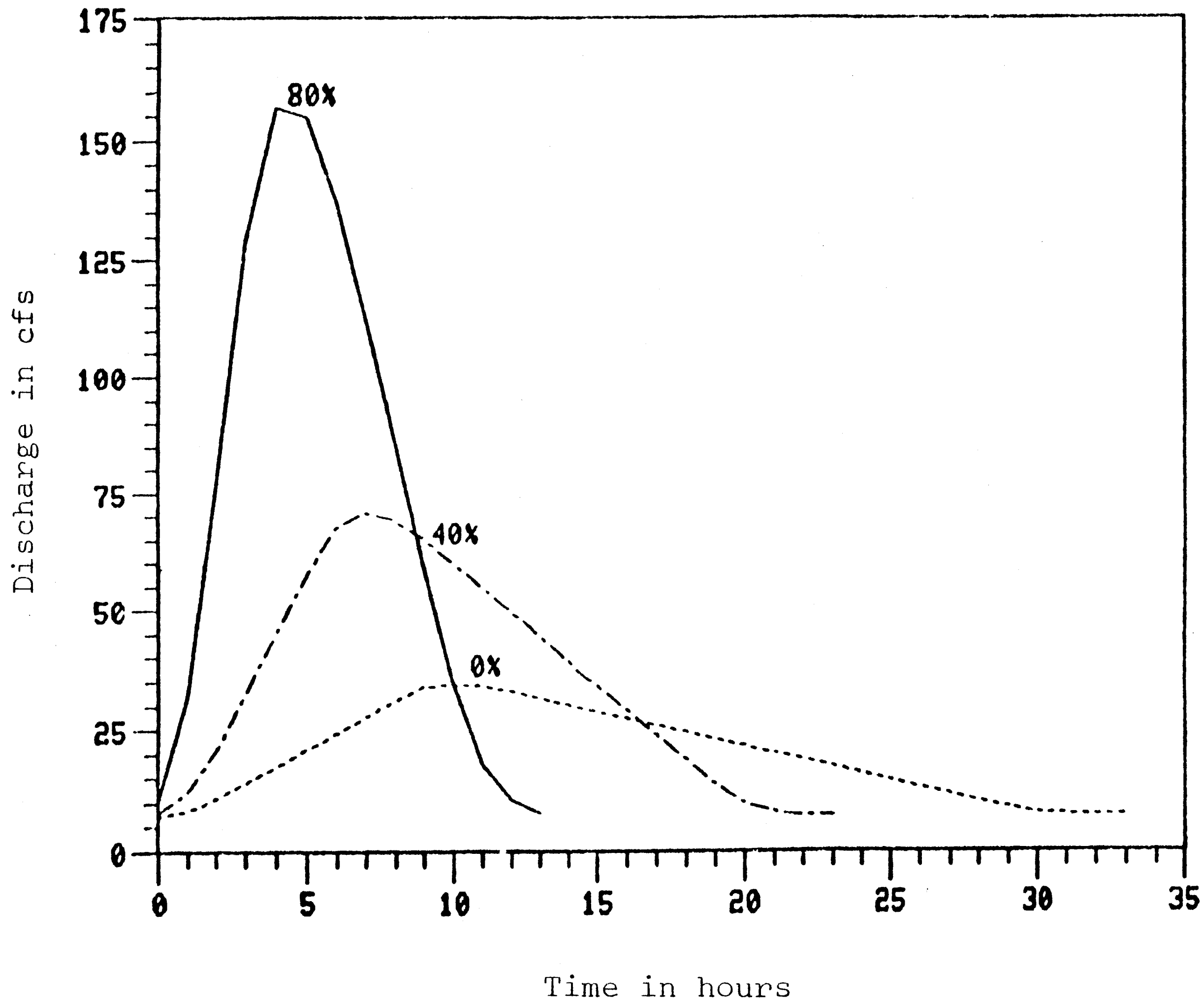


Figure 1. Effect of imperviousness on discharge, Kimages Creek - February 22, 1977 to March 1, 1977.

Flow Length

Flow length may be reduced if naturally meandering streams are changed to straight channels. It may be increased if overland flows are diverted through diversions, storm sewers, or street gutters to large collection systems.

Surface Roughness

Flow velocity normally increases significantly when the flow path is changed from flow over rough surfaces of woodland, grassland, and natural channels to sheet flow over smooth surfaces of parking lots, diversions, storm sewers, gutters, and lined channels.

Figure 2 illustrates the combined effects of increased imperviousness and storm sewerage on the mean annual flood for a drainage area of 1 mi.^{2} (2.6 km^2) as reported by Leopold (1968).

The U.S. Geological Survey and other agencies have attempted to quantify the effects of urbanization on watershed hydrology since the early 1960s using numerous urban flood studies such as those by Carter (1961), Anderson (1967), and Stankowski (1974), in which multiple regression analyses were used to relate flood discharge to the following factors:

- watershed area
- stream length and slope
- measures of urbanization (% of imperviousness and % sewered)
- recurrence intervals
- climatic factors

A flood peak adjustment factor or, "urban flood ratio" expressed as "$Q_{urban}/Q_{natural}$", was suggested as a result of such studies. Tables 1 and 2 show the urban flood ratios reported for New Jersey and Pennsylvania, respectively.

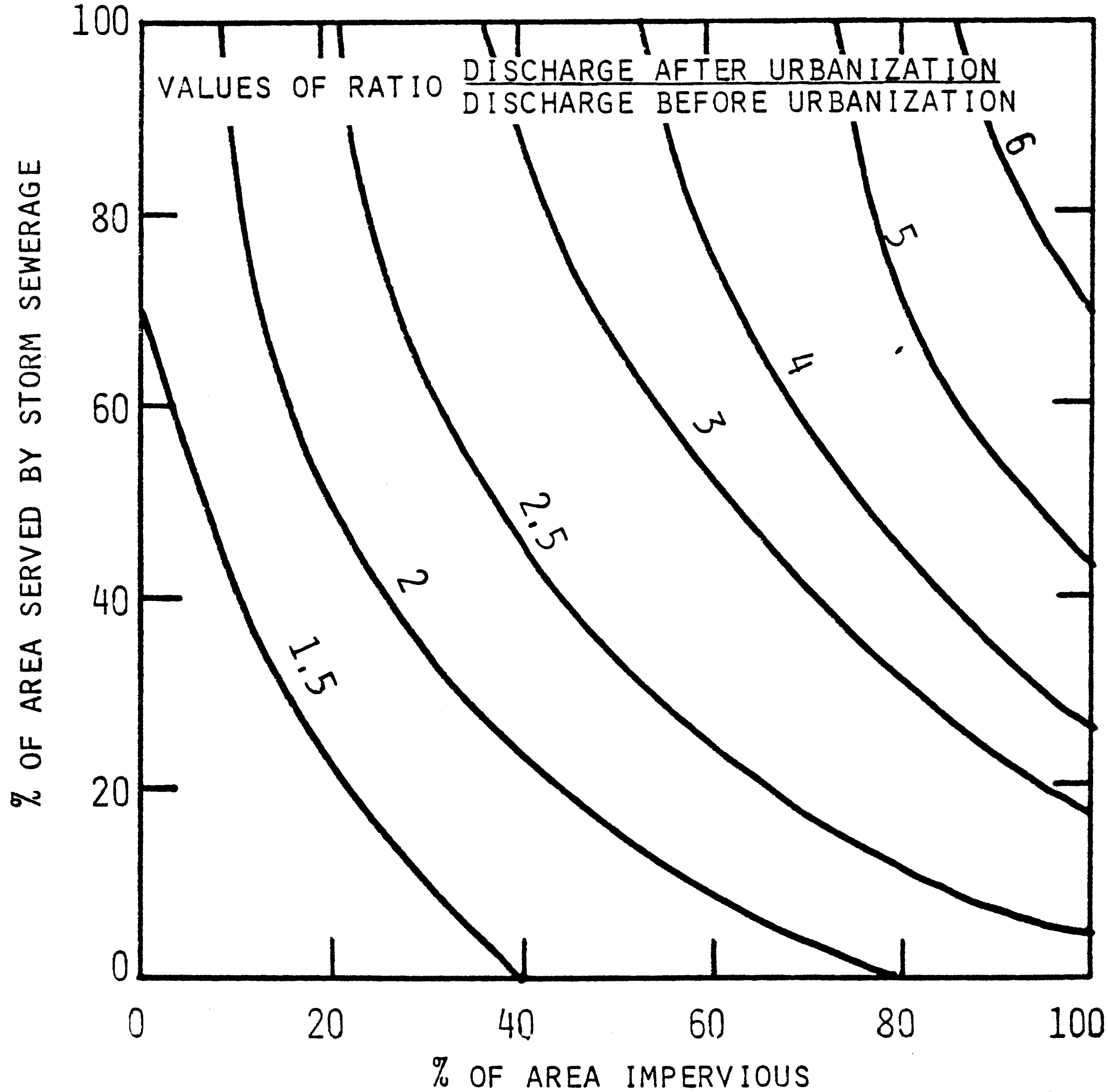


Figure 2. Effect of urbanization on mean annual flood. (After Leopold 1968).

Table 1

New Jersey ratios: $\frac{Q_{urban}}{Q_{nat}}$ (Stankowski 1974).

Recurrence interval (years)	Index of man-made impervious cover (percent)				
	1	10	25	50	80
2	1.0	1.8	2.2	2.7	3.0
5	1.0	1.7	2.0	2.4	2.6
10	1.0	1.6	1.9	2.2	2.4
25	1.0	1.5	1.8	2.0	2.2
50	1.0	1.4	1.7	1.9	2.0
100	1.0	1.4	1.6	1.7	1.8

Table 2

Pennsylvania ratios: $\frac{Q_{urban}}{Q_{nat}}$ based on regression and historical flood peaks (Aron and Kibler 1981)

Recurrence interval (years)	Impervious drainage area (percent)				
	1	10	25	50	80
2	1.0	1.4	1.6	1.7	1.8
5	1.0	1.3	1.4	1.5	1.6
10	1.0	1.2	1.3	1.4	1.5
25	1.0	1.2	1.2	1.3	1.4
50	1.0	1.1	1.2	1.3	1.3
100	1.0	1.1	1.2	1.2	1.2

HEC-1 Program and Computational Procedure

Numerous rainfall-runoff simulation models have been developed since the early 1970s. These models are generally complex and require extensive data for the calibration and verification which often preclude their usage. HEC-1, a general purpose rainfall-runoff simulation model developed by the Corps of Engineers, is one of the most widely used of these packages, as well as SWMM (the EPA's storm management model) STORM (Corps of Engineers), UCURM (University of Cincinnati urban runoff model), ILLUDAS (Illinois urban drainage simulation), MITCAT (MIT catchment model), and TR20 (Soil Conservation Service runoff model).

A survey of the highway drainage design practices of nearby states has indicated that the classical rational formula, as well as multiple regression methods, is also widely used in rainfall-runoff predictions as highlighted below.

1. Maryland

   Rational formula
   TR20 model
   USGS multiple regression method - some flood frequency data are synthesized by using the USGS rainfall-runoff model (Dawdy et al. 1972).

2. North Carolina

   USGS multiple regression method

3. Pennsylvania

   Rational formula
   Federal Highway Administration Method (FHWA 1977)
   Penn State University rainfall-runoff model (PSU III)

4. Tennessee

   Rational formula
   USGS multiple regression method

5. West Virginia

   Rational formula
   Soil Conservation Service method
   FHWA method

Although the effect of change in land use and cover on flood discharges is implicitly reflected in the runoff coefficient of the rational formula, the method is too simplistic to handle the complex watershed response to urbanization. Multiple regression methods such as those proposed by the USGS and modified to include urbanization and attenuation effects appear to be an adequate approach (Aron and Kibler 1981). On the other hand, the detailed computer simulation models such as HEC-1, SWMM, and TR20, if properly calibrated and verified, provide the most accurate and complete description of the rainfall-runoff process in urban areas. HEC-1 was selected by the Virginia Department of Highways and Transportation to be examined in this study.

The HEC-1 flood hydrograph package (Corps of Engineers 1973) is a general purpose, lumped parameter, rainfall-runoff event simulation model consisting of a main program and six subroutines. Two of the subroutines determine the optimal unit hydrograph, loss rate, or streamflow routing parameters by matching recorded and simulated hydrograph values. The other routines perform snowmelt, unit hydrograph, hydrograph routing and combining, and hydrograph balancing computations. HEC-1 is one of the most widely used event simulation models for determining runoff from a given storm event.

In order to apply HEC-1, or any unit hydrograph procedure, to a given watershed, certain parameters must be supplied. These include loss rate and unit hydrograph parameters, so that the program can obtain the precipitation excess and the runoff hydrograph. The loss rate for the HEC-1 model is an exponential decay function that depends on the rainfall intensity and the antecedent losses as illustrated below.

$$ALOSS = (AK + DLTK)(RAIN)^{ERAIN}, \qquad (1)$$

where

ALOSS = loss rate in inches per hour,

AK = basin loss coefficient,

DLTK = incremental loss coefficient,

RAIN = rainfall in inches per hour,

ERAIN = exponent of the rainfall relative to how storms occur over subarea;

and

$$AK = STRKR/(RTIOL)^{.1CUML}, \qquad (2)$$

in which

STRKR = basin loss index for start of storm in inches per hour,

RTIOL = ratio of loss coefficient (AK) to that AK after 10 inches (25.4 cm) or more of accumulated loss occurs, and

CUML = accumulated loss in inches.

Also

$$DLTK = .2\ DLTKR[1-(CUML/DLTKR)]^2$$
for (CUML/DLTKR)<1, otherwise zero;

and

DLTKR = amount of accumulated rain loss during which the loss coefficient is initially increased.

The HEC-1 program employs the instantaneous unit hydrograph concept and linear routing scheme proposed by Clark (1945) to compute the runoff hydrograph from excess rainfall and to route it through the basin. The excess rainfall is determined by using the loss rate function as given in equation (1).

The HEC-1 parameters investigated in this study are listed in Table 3. These include the loss rate and unit hydrograph parameters as studied for rural watersheds (Cruise and Yu 1982) and two hydrograph parameters, QRCSN and RTIOR, as suggested by engineers of the Location and Design Division of the Virginia Department of Highways and Transportation.

Table 3

HEC-1 Parameters Investigated

Loss Rate Parameters

ERAIN	-	Exponent of the rainfall relative to how storms occur over the subarea; varies between 0.0 and 1.0.
STRKR	-	Basin loss index for start of storm; depends on basin characteristics such as soil type, land use, vegetation cover, etc.
RTIOL	-	Ratio of loss coefficient (AK) to that AK after 10 inches (25.4 cm) or more accumulated loss occurs; a function of the ability of the basin to absorb precipitation.
DLTKR	-	Amount of initial accumulated rain loss during which the loss coefficient is increased; depends primarily on antecedent soil moisture deficit.

Clark Unit Hydrograph Parameters

TC	-	Time of concentration; depends on basin size and shape, length of channel, land cover, etc.
R	-	Clark's storage constant; can be taken as a fraction of TC.

Runoff Hydrograph Parameters

QRCSN	-	Discharge at which recession flow begins; may be a function of peak discharge, precipitation intensity, drainage area, or other watershed characteristics.
RTIOR	-	Recession coefficient that is the ratio of flow at time t to that 10 computational periods later during recession; may be a function of soil type, land use, water table level, soil profile, and permeability.

PROCEDURE TO CALCULATE PARAMETERS

HEC-1 determines the optimum values for unit hydrograph and loss rate parameters by a sequentially successive approximation. These optimum values represent the minimum weighted squared deviations between the observed hydrograph and the reconstituted hydrograph. Weightings are computed to give greater importance to higher flows.

The following multistage procedure was adopted from the HEC-1 User's Manual and utilized throughout the study to optimize unit hydrograph and loss rate parameters.

1. Set ERAIN to 0.50 for all optimization runs.

2. Conduct an optimization run, allowing HEC-1 to determine optimal values for RTIOL, STRKR, DLTKR, TC, and R.

3. Based on the results of the optimization run conducted in step 2, fix value for RTIOL and conduct an optimization run, allowing HEC-1 to determine optimal values for STRKR, DLTKR, TC, and R.

4. Based on the results of the optimization run conducted in step 3, fix value for STRKR and conduct an optimization run, allowing HEC-1 to determine optimal values for DLTKR, TC, and R.

5. Based on the results of the optimization run conducted in step 4, fix value for R/TC+R and conduct an optimization run, allowing HEC-1 to determine optimal values for DLTKR, TC, and R.

6. Based on the results of the optimization run conducted in step 5, fix values for TC and R and conduct an optimization run, allowing HEC-1 to determine optimal values for DLTKR (note: values for DLTKR are never fixed, because DLTKR varies from storm to storm).

7. Based upon the acceptability of (i) the hydrograph reproduction and (ii) the values for TC and R from step 6, repeat step 6 if results are unacceptable. Otherwise, continue on to step 8.

8. Using previously fixed values for RTIOL, STRKR, TC, and R as starting points, conduct an optimization run, allowing HEC-1 to determine optimal values for these parameters.

9. Based upon the acceptability of the hydrograph reproduction, reset starting point for RTIOL, STRKR, TC, and/or R, and rerun optimization of HEC-1 if hydrograph reproduction is unacceptable. Otherwise, continue on to step 10.

10. Determine final values for RTIOL, STRKR, DLTKR, TC, and R based on (i) the value ranges for different storms and (ii) the quality of the hydrograph reproduction.

TEST WATERSHEDS AND INVENTORY OF DATA

Five watersheds were chosen according to the availability of stream gage data, precipitation data, and land use characteristics. As shown in Figure 3, one watershed was located in Region C1378-C25, one in RV12, and three in the highly urbanized area of Northern Virginia in Region P138.

Subsequent to the selection of the watersheds, their physical characteristics were determined. Watersheds were sketched on USGS topographic maps according to drainage divides, and their areas were calculated from the topographic maps using the grid method. Basin slopes were also calculated from the maps on an incremental basis and averaged for the total channel length. Channel lengths were measured directly from the maps, and lengths to the watershed centroid were developed from incremental basin lengths, widths, and areas. These data are given in Table 4.

Daily discharge records were examined for candidate runoff events. Isolated single peaked events were selected wherever possible and, in turn, hourly digital or continuous strip chart discharge records were obtained from the United States Geological Survey in Richmond, or the Virginia State Water Pollution Control Board in Charlottesville. At least ten of the best candidate events were chosen to be used for the HEC-1 optimization procedure.

Rainfall data were obtained from the National Oceanic and Atmospheric Administration publications "Climatological Data" and "Hourly Precipitation Data" (1952-1980). One or more rain gages were chosen for each watershed according to the availability of hourly data and the proximity of the gage to the watershed. Where more than one gage was available, the gages were weighted according to location to produce a single set of hourly data for each storm event and watershed.

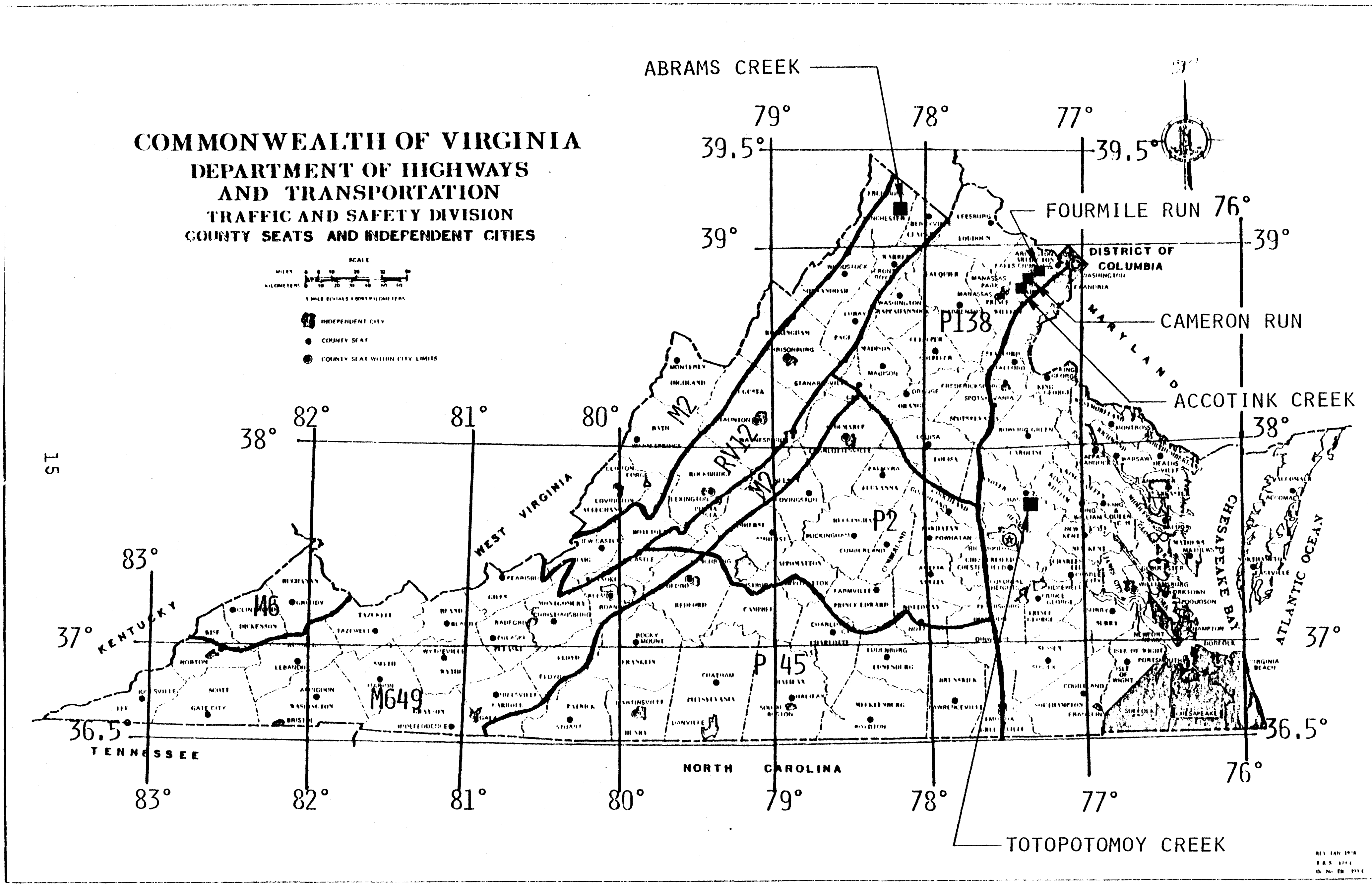


Figure 1. Location map of test watersheds.

317

Table 4

Watershed Characteristics

Watershed	Drainage Area ($mi.^2$)	L (ft.)	L_{ca} (ft.)	S_{st} (ft./mi.)	$\frac{(LL_{ca})^{.3}}{\sqrt{S_{st}}}$ $(mi.^2)^{.3}/(ft./mi.)^{.5}$	Average Imperviousness (percent)
Abrams Creek	16.50	55,750	38,625	33.24	0.6390	12
Accotink Creek	23.50	59,136	29,700	18.90	0.7972	24
Cameron Run	33.70	63,600	23,700	38.64	0.5326	26
Fourmile Run	14.80	41,184	18,638	45.94	0.3989	33
Totopotomoy Creek	5.89	27,200	10,900	13.31	0.5571	3

Conversions: 1 $mi.^2$ = 2.6 km^2

1 ft. = 0.30 m

1 mi. = 1.6 km

The percent imperviousness of a watershed was chosen as the index of urbanization. Various methods for estimating the imperviousness available in the literature were reviewed and the procedure developed by Stankowski (1974) was used, as described by the following equation.

$$I = 9.6\ PD^{(0.573\ -\ 0.0391\ \log_{10}PD)}, \qquad (3)$$

where

I = imperviousness, percent, and

PD = population density, persons/acre (1 acre = 0.40 hectare).

The equation, together with some other literature data, are also shown in Figure 4.

Population data for each watershed were obtained from county planning offices and from agencies such as the Northern Virginia Planning Commission.

The average percent imperviousness data for the watershed were also given in Table 4.

RESULTS AND DISCUSSION

Loss Rate and Unit Hydrograph Parameters

Fourty-two storm events were successfully analyzed for the five watersheds using the HEC-1 optimization procedure. Table 5 presents the date of the storm event, peak rainfall intensity, peak flow rate, and the "optimal" values of the unit hydrograph and loss rate parameters for all storm events.

The results obtained for the essentially rural watershed of Totopotomoy Creek (I = 3%) and those for Fourmile Run (I = 33%), illustrate that in general TC and R decrease as I increases. Both STRKR and DLTKR also decrease with I, even though large variations exist in both parameters. RTIOL, the rate of exponential decrease of the loss-rate coefficient with accumulated loss, does not seem to exhibit any definite trend.

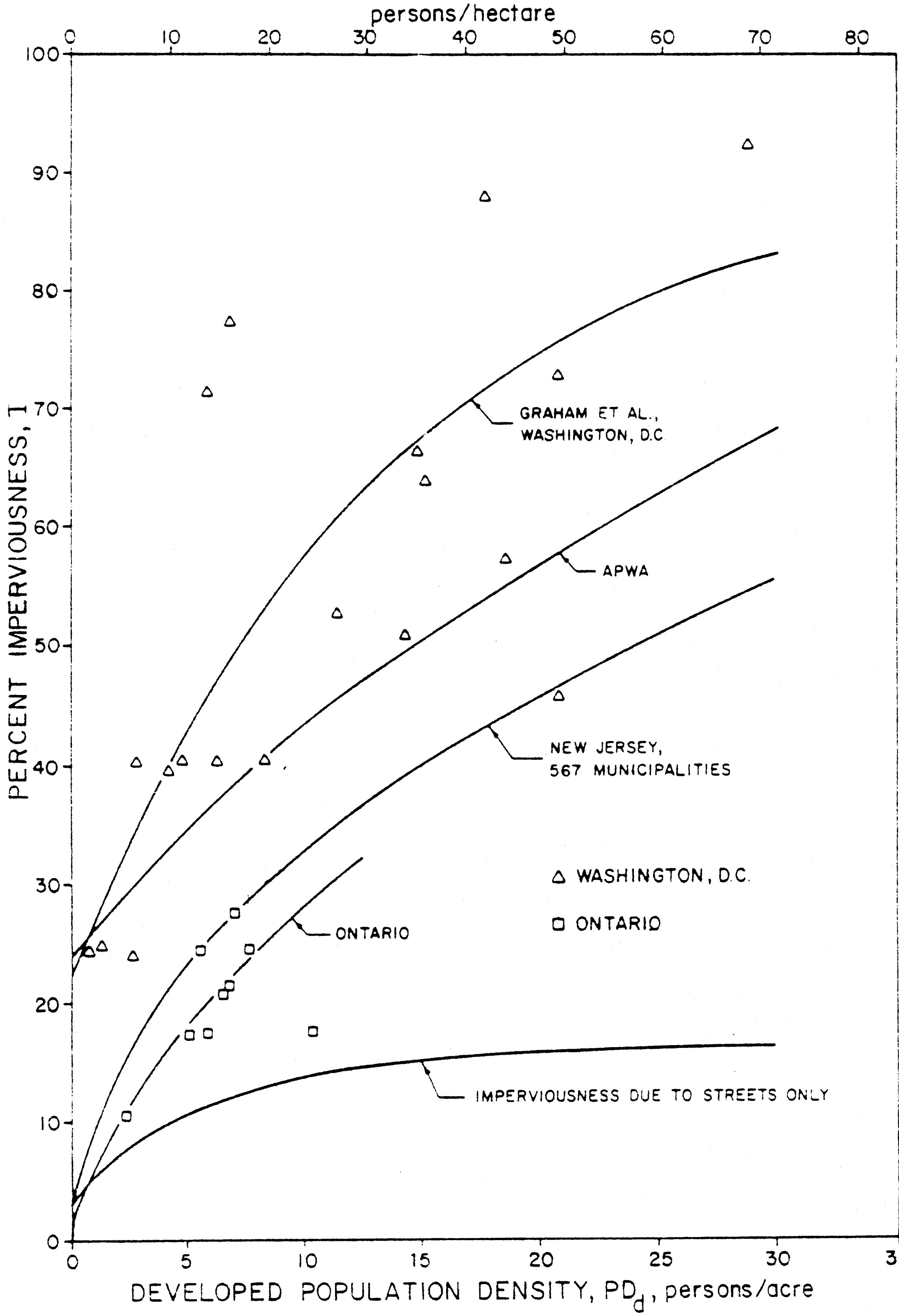


Figure 4. Imperviousness as a function of developed population density. (After Heaney et al. 1976).

Table 5

Results of HEC-1 Optimization by Watershed

Date	TC (hr.)	R (hr.)	STRKR (in./hr.)	DLTKR (in.)	RTIOL	Peak Precipitation (in./hr.)	Peak Flow (ft.^{3}/s)
			Abrams Creek				
24/11/50	3.55	6.50	0.51	1.83	2.83	0.61	602
12/04/51	6.06	3.63	0.29	0.85	2.81	0.37	528
1/06/59	5.92	4.04	0.53	1.78	2.65	0.55	549
9/10/79	5.70	11.01	0.28	0.30	7.39	0.23	408
20/05/80	4.93	8.43	0.19	0.94	4.50	0.20	483
12/06/82	6.44	6.98	0.31	0.70	5.42	0.30	582
			Accotink Creek				
12/04/61	2.56	9.24	0.19	0.02	1.00	0.29	980
6/11/63	12.14	5.55	0.41	0.01	1.73	0.40	1,146
4/03/65	8.55	3.06	0.25	0.01	1.00	0.52	1,492
18/10/66	3.26	5.86	0.19	1.77	5.29	0.34	1,312
6/05/67	5.00	6.00	0.23	0.45	1.99	0.22	704
14/04/70	4.47	6.66	0.22	0.04	1.00	0.49	1,319
28/10/72	8.19	2.17	0.22	2.26	7.63	0.75	1,966
13/07/75	11.35	1.19	0.34	0.18	1.00	0.87	603
17/12/77	3.97	4.45	0.18	0.01	1.00	0.39	1,315
			Cameron Run				
2/06/69	5.59	1.38	0.49	0.63	1.43	0.73	1,678
22/07/69	4.15	3.69	0.90	2.45	1.92	3.29	3,644
13/04/70	1.03	3.80	0.34	0.18	1.00	0.49	2,531
12/05/71	2.43	5.00	0.24	1.19	3.71	0.41	1,740
9/10/71	1.03	4.64	0.49	0.38	3.13	0.67	1,934
3/02/72	2.05	3.42	0.07	0.53	1.00	0.18	1,670
14/09/73	1.03	4.05	0.13	0.40	1.00	0.20	1,599
15/12/74	1.03	4.03	0.15	0.29	3.70	0.22	1,526
12/07/75	1.03	2.78	0.35	0.73	1.00	0.87	4,830
6/12/76	1.03	2.44	0.18	0.55	4.22	0.22	2,153
17/12/77	1.03	3.47	0.29	0.09	1.15	0.39	3,075
			Fourmile Run				
28/11/66	2.22	3.14	0.18	0.04	1.00	0.12	304
2/05/67	2.71	1.42	0.26	0.68	9.73	0.15	264
2/02/68	1.29	2.69	0.14	0.33	1.00	0.14	260
1/12/68	3.16	1.50	0.13	0.33	6.17	0.06	171
23/10/79	1.29	1.23	0.31	0.68	2.00	0.24	239
13/03/80	2.93	1.32	0.20	0.72	7.37	0.34	1,217
18/05/80	2.42	1.10	0.29	0.91	7.37	0.34	660
9/04/81	2.07	1.14	0.18	0.43	2.98	0.13	208
25/07/81	1.77	1.19	0.31	0.73	3.06	0.37	825
			Totopotomoy Creek				
23/11/59	20.99	14.00	0.30	1.25	2.31	0.60	118
2/06/63	14.92	15.72	0.70	0.01	3.38	0.68	111
4/03/65	13.78	27.11	0.21	0.84	2.24	0.34	69
5/07/72	9.68	16.67	0.28	1.17	5.10	0.36	93
5/09/74	6.30	20.14	0.19	1.85	2.38	0.22	187
16/03/75	10.61	16.77	0.02	1.19	1.33	0.32	199
9/09/77	18.50	37.56	0.47	0.34	1.46	0.93	99

Conversions: 1 in. = 2.5 cm
1 ft.^{3} = .03 m^3

An attempt was made first to examine how the parameter selection curves as proposed by Cruise and Yu (1982) should be modified to reflect urbanization effects. This was done by plotting the averages of the parameters on the regional parameter selection curves as shown in Figures 5 through 16.

Three of the test watersheds – namely, Accotink Creek (I = 24%), Cameron Run (I = 26%), and Fourmile Run (I = 33%) – are in Region P138-M13. Figures 5 and 6, given earlier, show the average TC and R for these basins, plotted with the parameter selection curves proposed for rural watersheds. In those figures, it can be seen that both TC and R were lower for urbanized watersheds, especially R, for which a lower parameter selection curve was drawn (Figure 6). A similar trend was obtained for STRKR (Figure 7), but for RTIOL the trend was unclear (Figure 8).

Abrams Creek (I = 12%) is in Region RV12, and Totopotomoy Creek (I = 3%) in Region C1378-C25. As can be seen in Figures 9 through 16, no trends could be detected. This is due to a lack of data and also to the fact that both watersheds are not significantly urbanized.

It was decided to group all data from the five watersheds together and to utilize statistical analyses to examine the possible relationship between the parameters and the index of urbanization, i.e., the percent imperviousness. Since a larger data base was used, the results would be expected to be more definitive.

The results are presented in Figures 17 through 23. The analyses were made using the general purpose statistics program MINITAB available at the University of Virginia Computer Center.

It was found that the composite variable (TC+R) correlated fairly well with the slope and stream length (Figures 17 and 18). This was done because a technique for estimating TC and R, which was based on the same regression analysis, was proposed for Illinois streams (USGS 1982).

$$(TC + R) = 35.2\ L^{0.39} S^{-0.78}, \qquad (4)$$

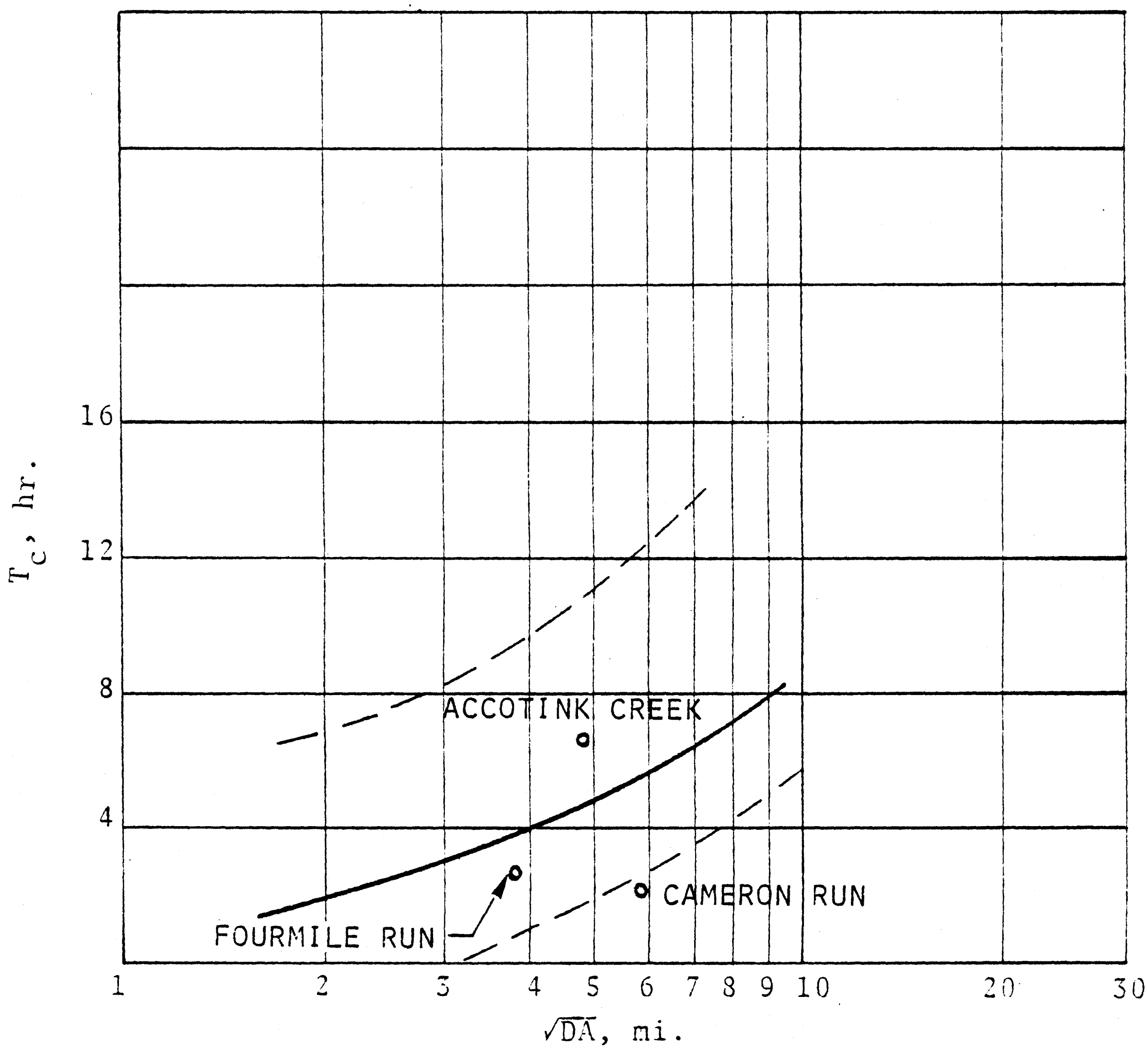


Figure 5. Parameter selection curve T_c for Region P138-M13.
1 mi. = 1.6 km.

324

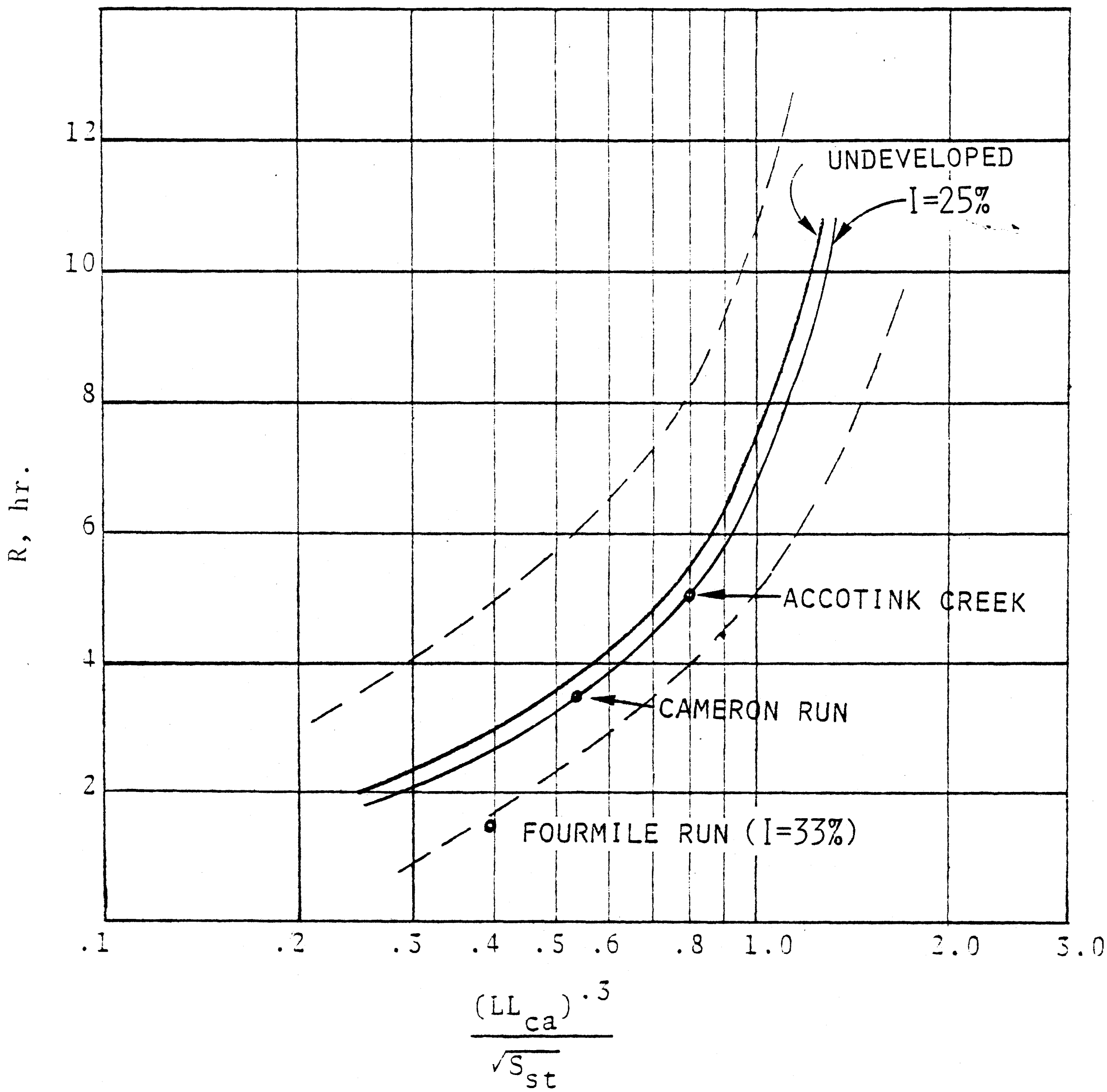


Figure 6. Parameter selection curve R for Region P138-M13. I = watershed imperviousness.

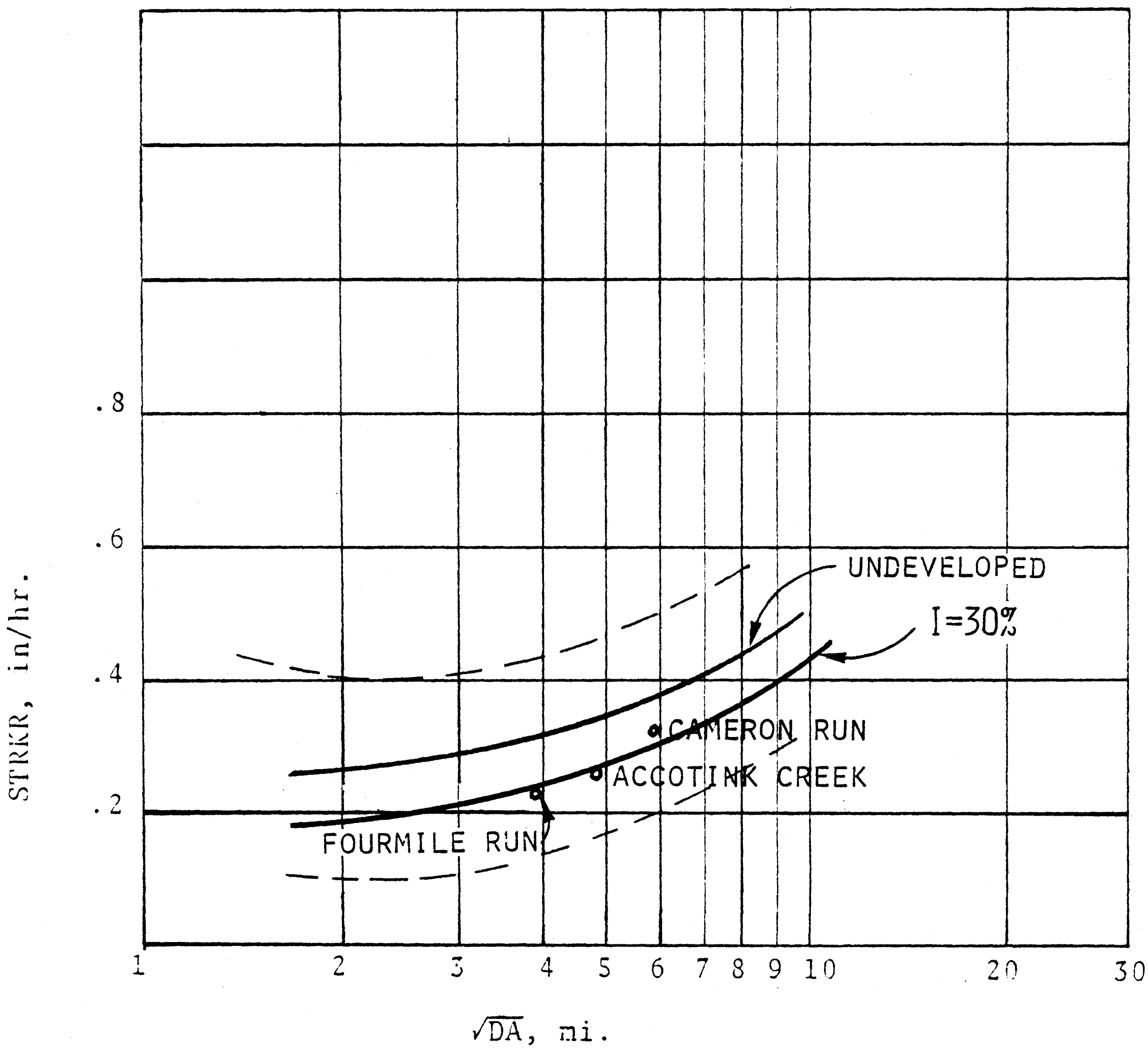


Figure 7. Parameter selection curve STRKR for Region P138-M13. I = watershed imperviousness. 1 mi. = 1.6 km, 1 in. = 2.54 cm.

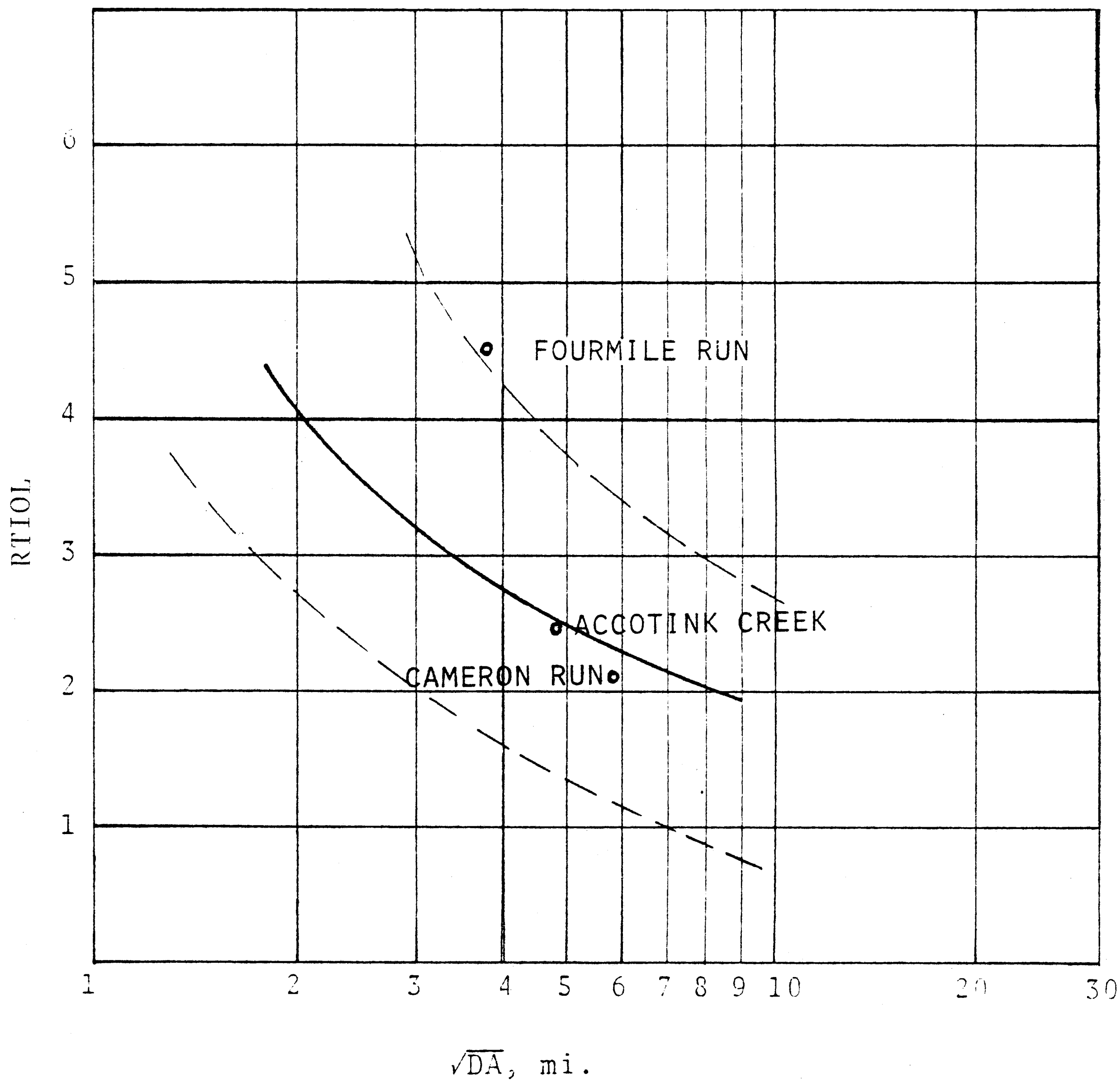


Figure 8. Parameter selection curve RTIOL for Region P138-M13. 1 mi. = 1.6 km.

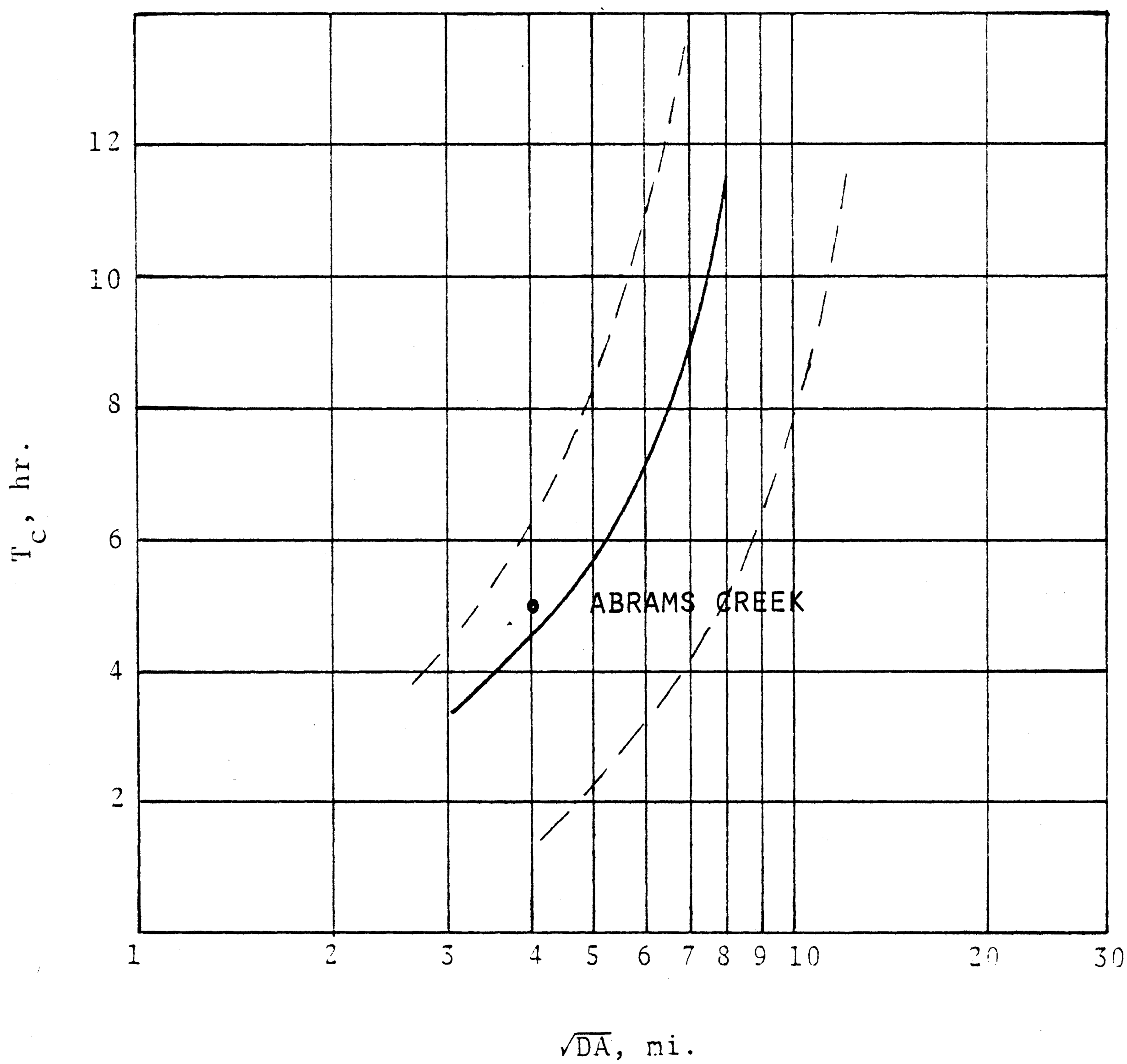


Figure 9. Parameter selection curve T_c for Region RV12.
1 mi. = 1.6 km.

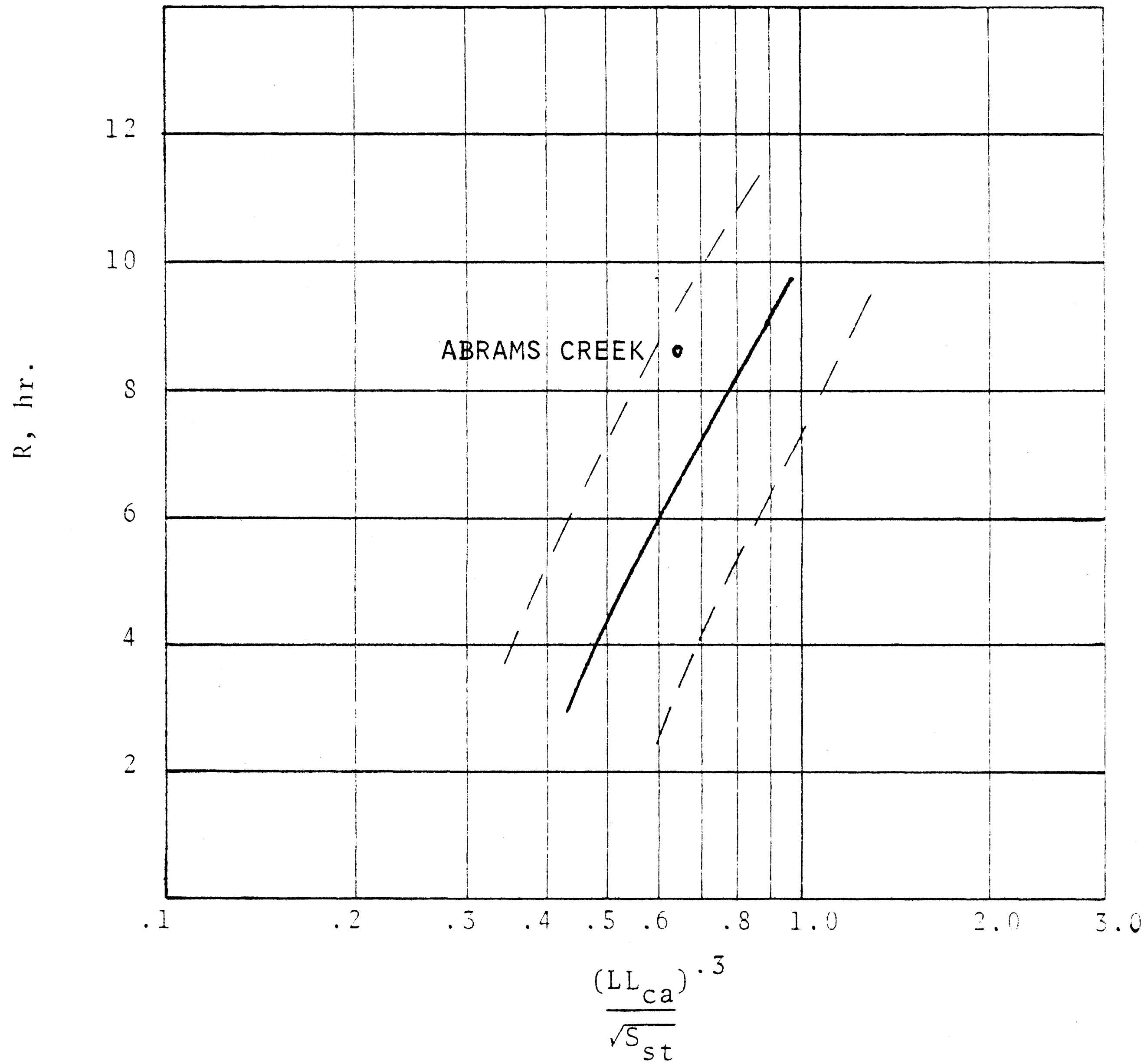


Figure 10. Parameter selection curve R for Region RV12.

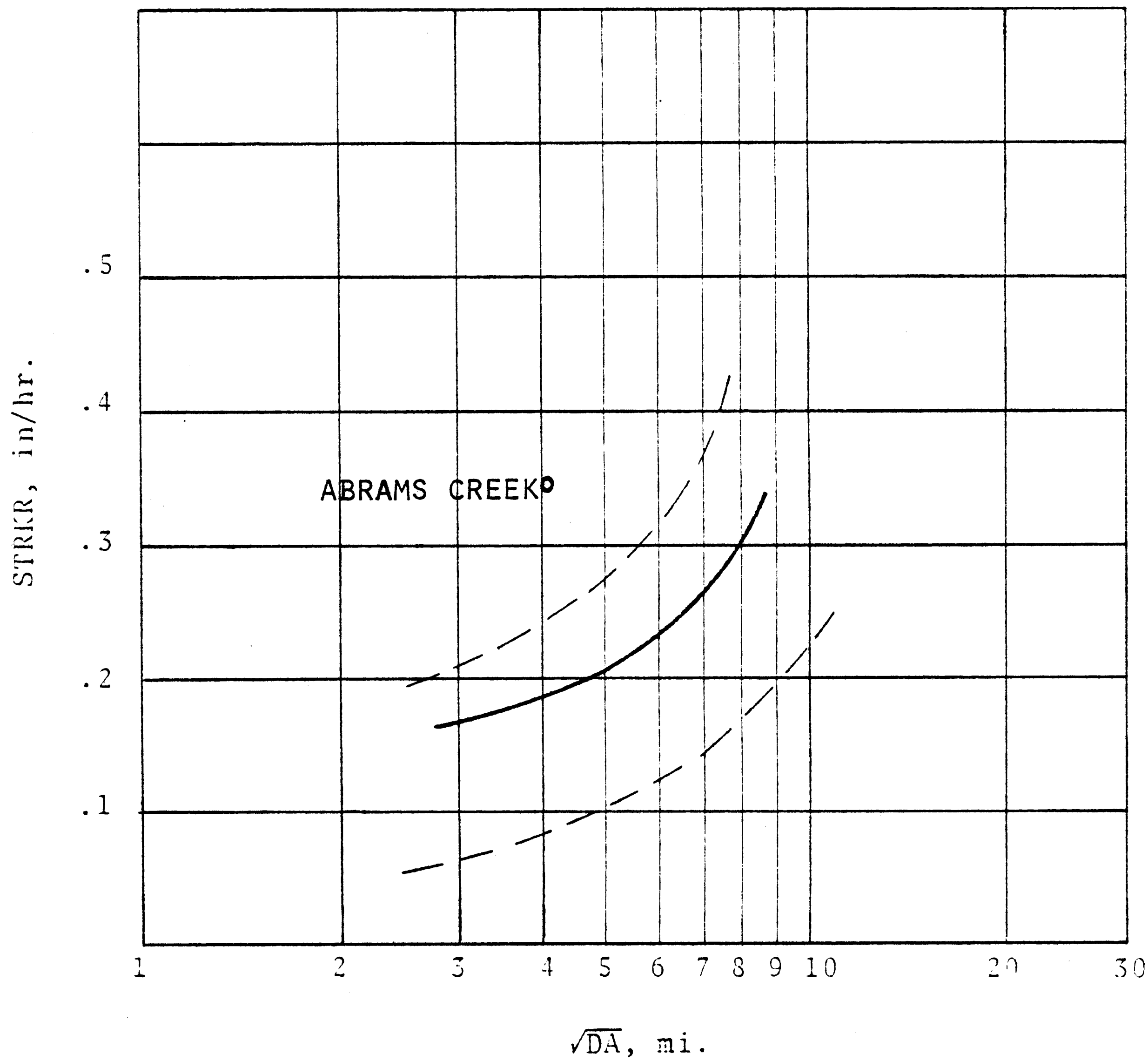


Figure 11. Parameter selection curve
STRKR for Region RV12.
1 mi. = 1.6 km, 1 in. = 2.54 cm.

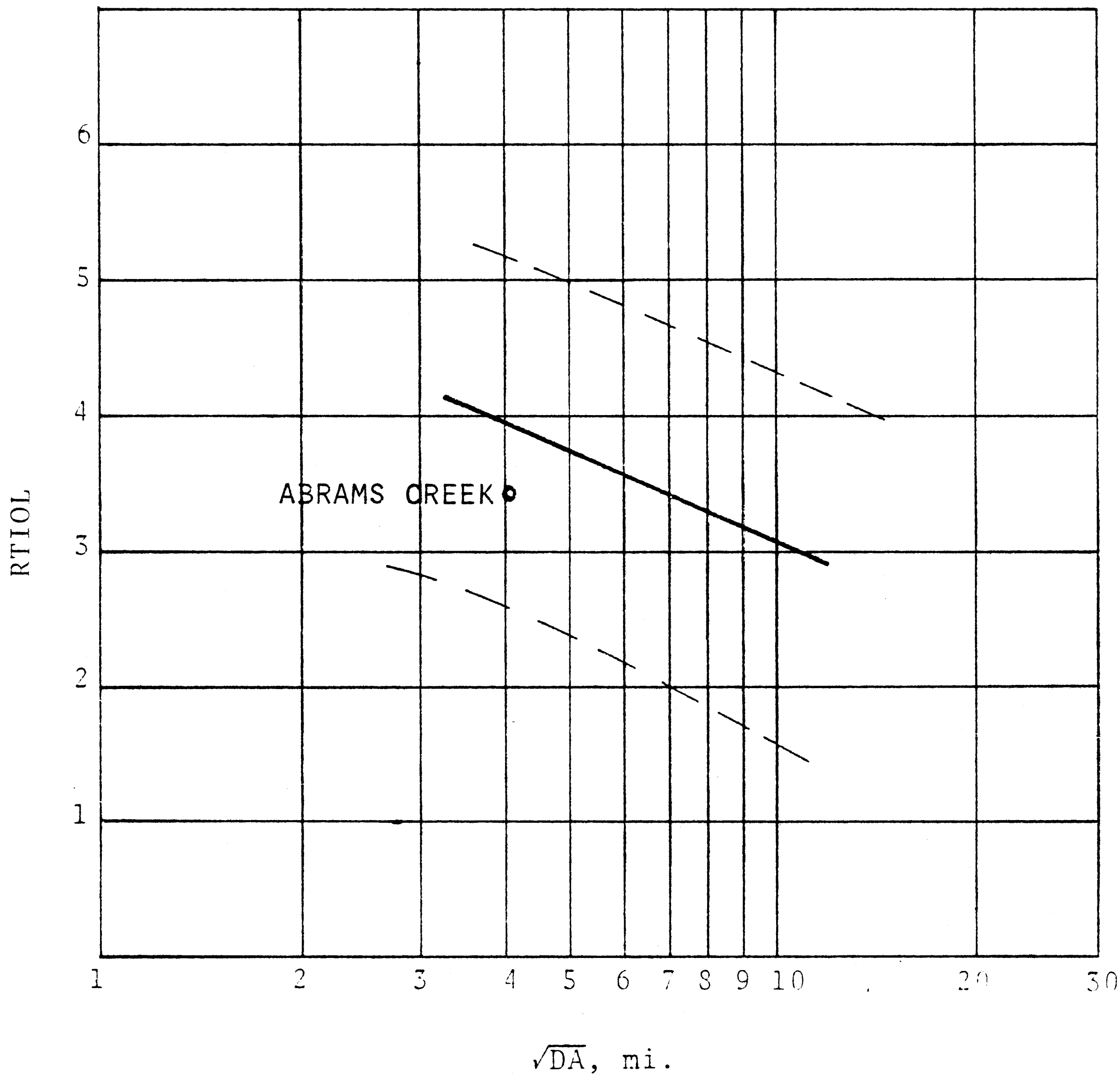


Figure 12. Parameter selection curve RTIOL for Region RV12. 1 mi. = 1.6 km.

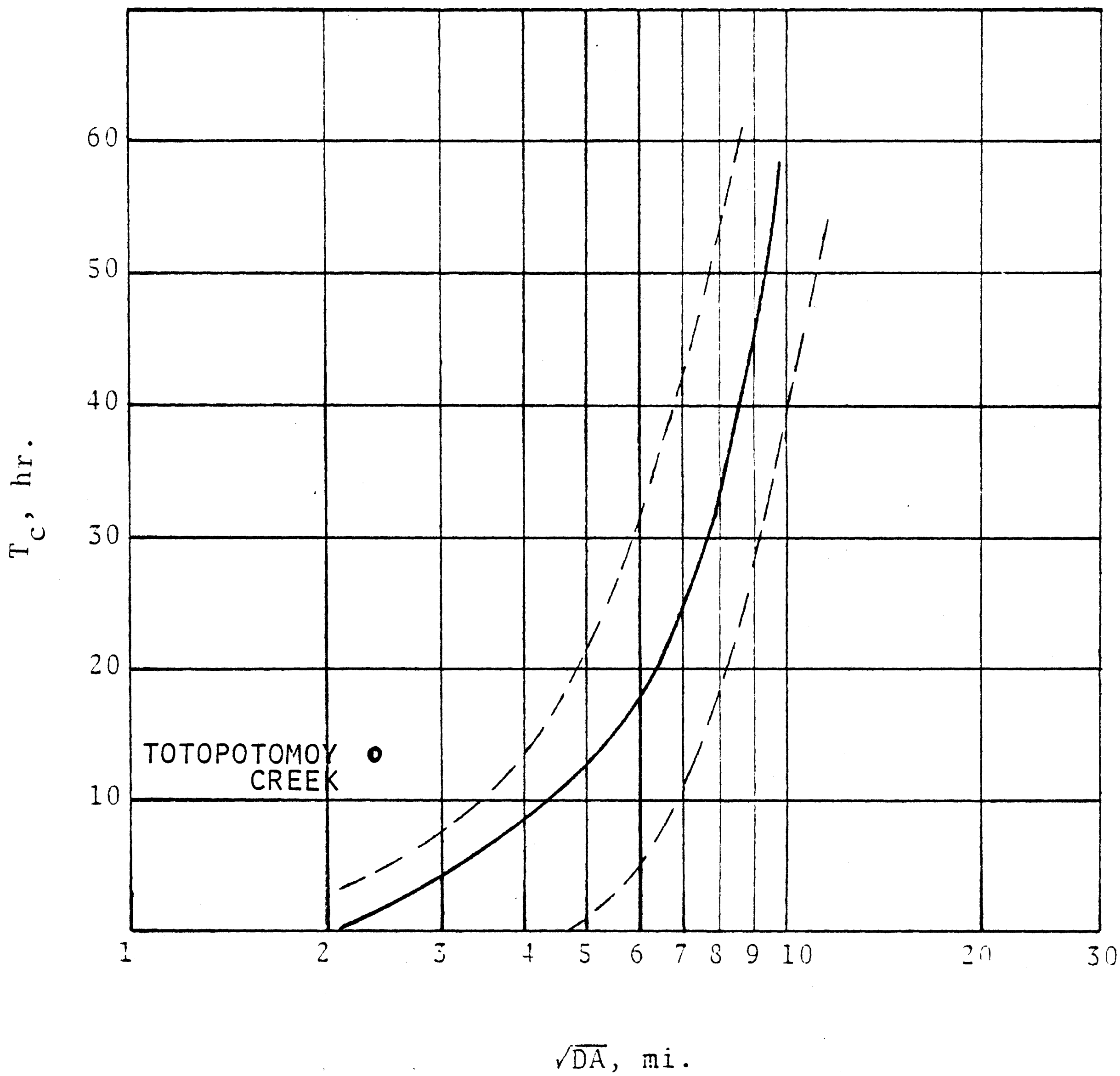


Figure 13. Parameter selection curve T_c for Region C1378-C25.
1 mi. = 1.6 km.

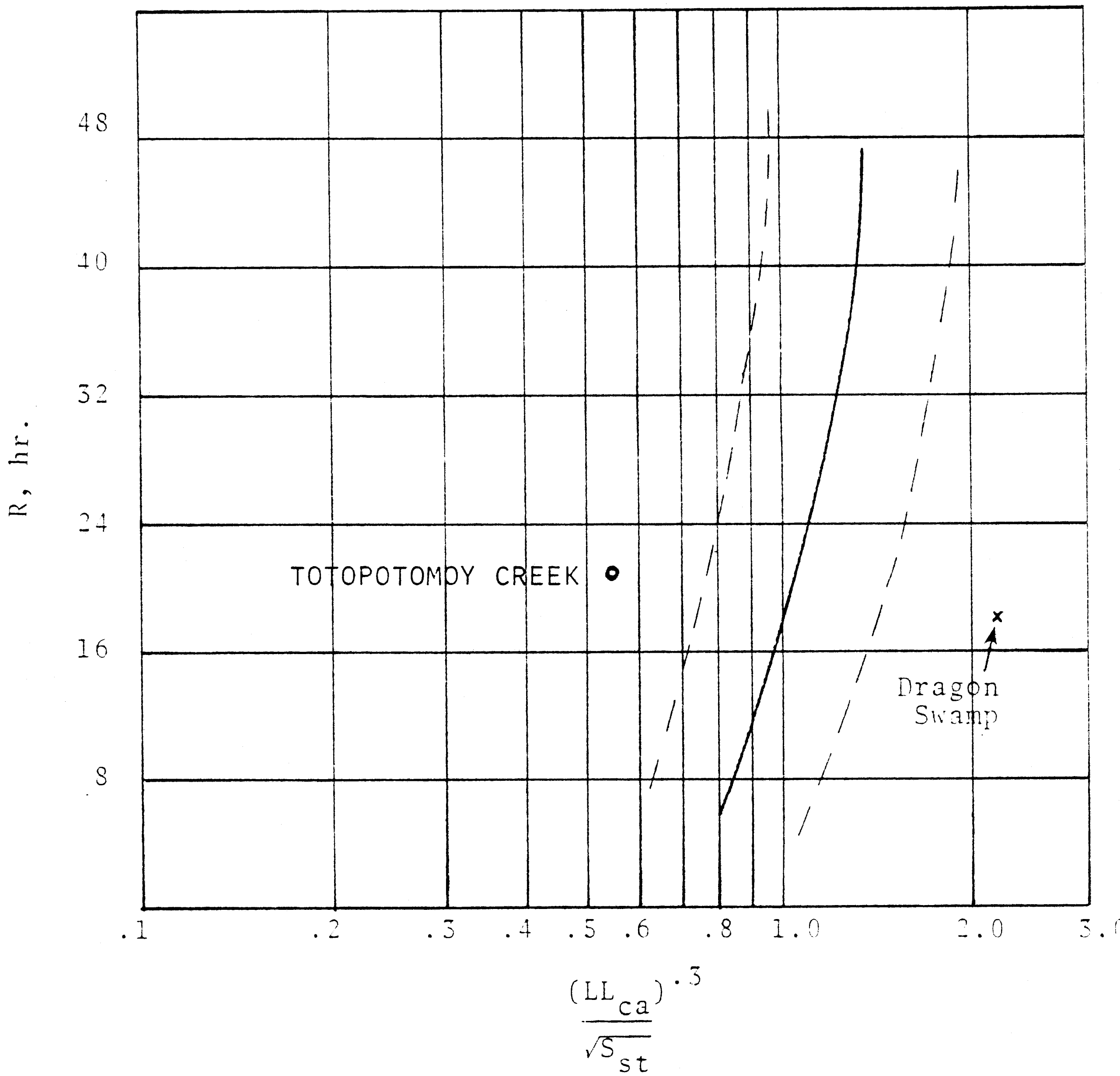


Figure 14. Parameter selection curve R for Region C1378-C25.

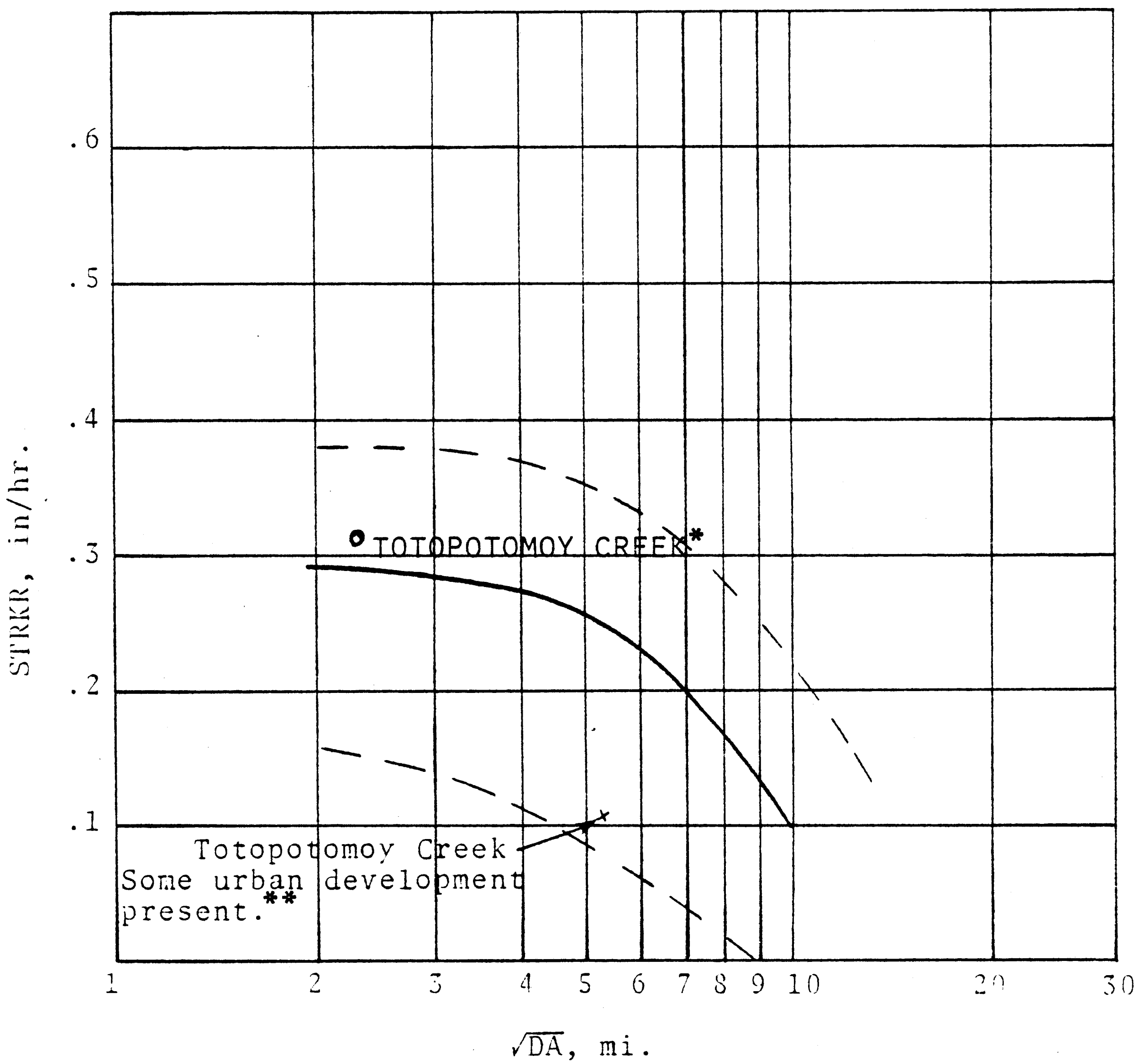


Figure 15. Parameter selection curve STRKR for Region C1378-C25. 1 mi. = 1.6 km, 1 in. = 2.54 cm.

*Gage at US 301, watershed with 3% imperviousness.
**Gage at SR 606.

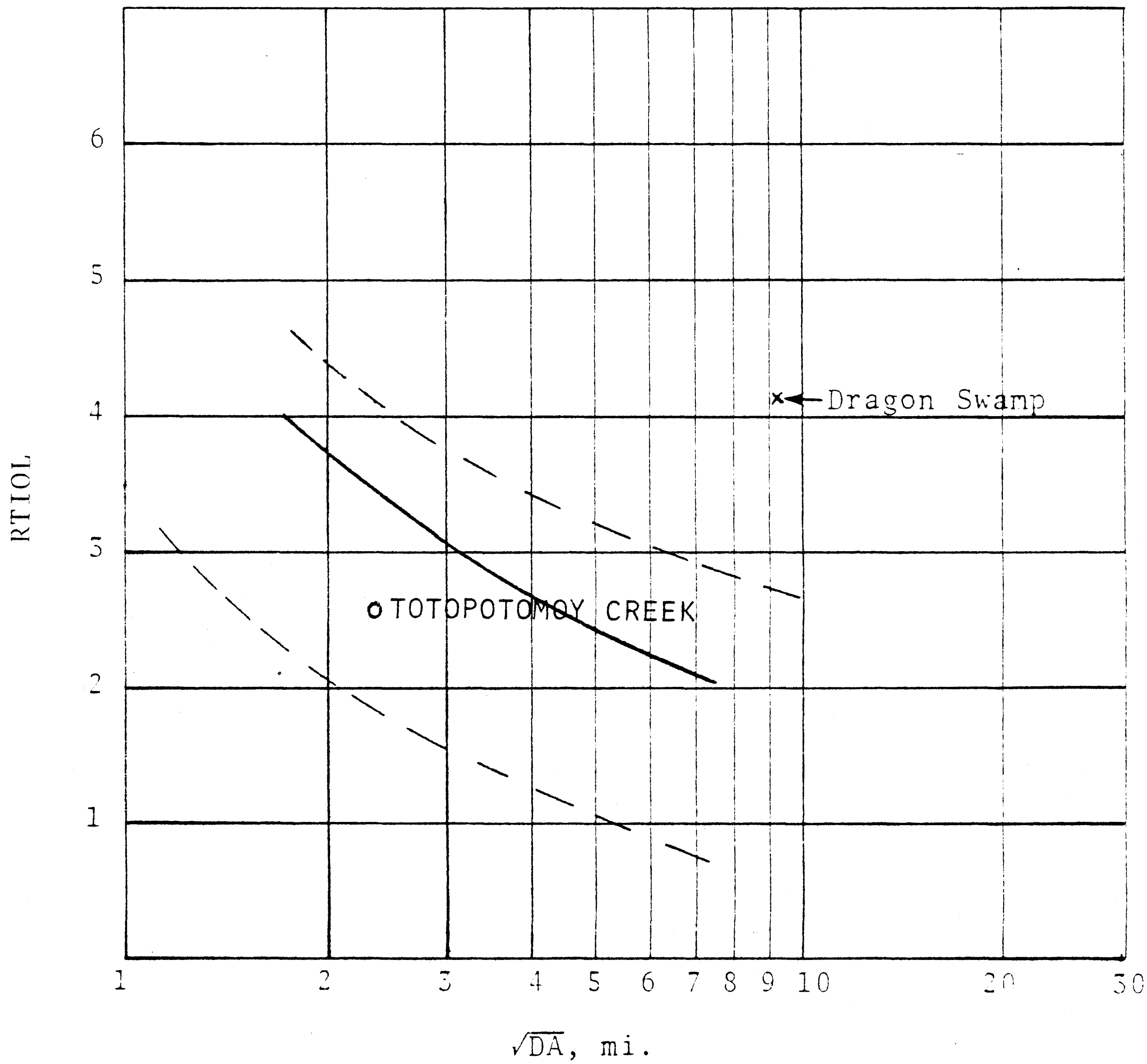


Figure 16. Parameter selection curve
RTIOL for Region C1378-C25.
1 mi. = 1.6 km.

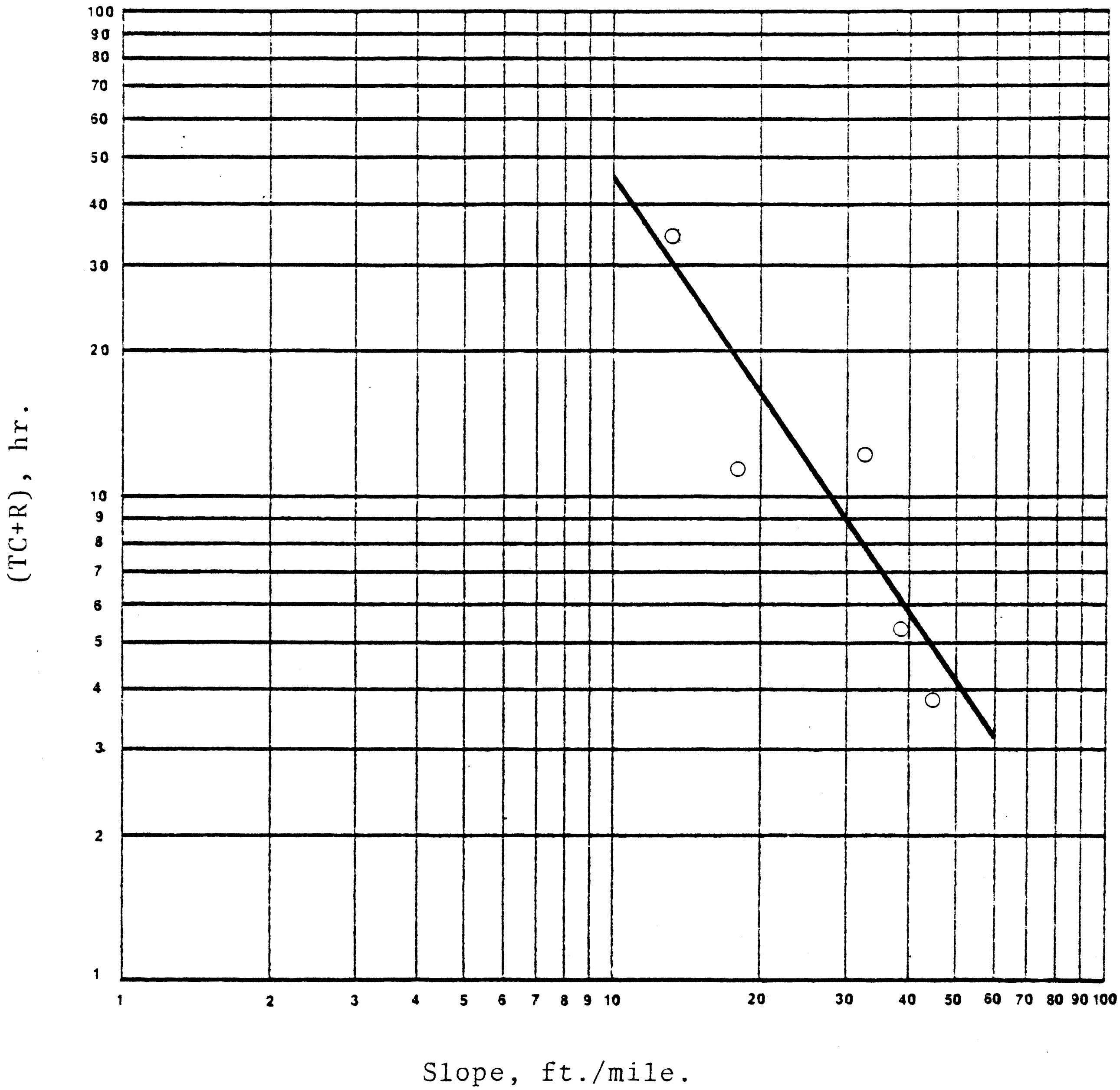


Figure 17. (TC+R) versus channel slope.
1 ft. = 0.305 m
1 mi. = 1.6 km

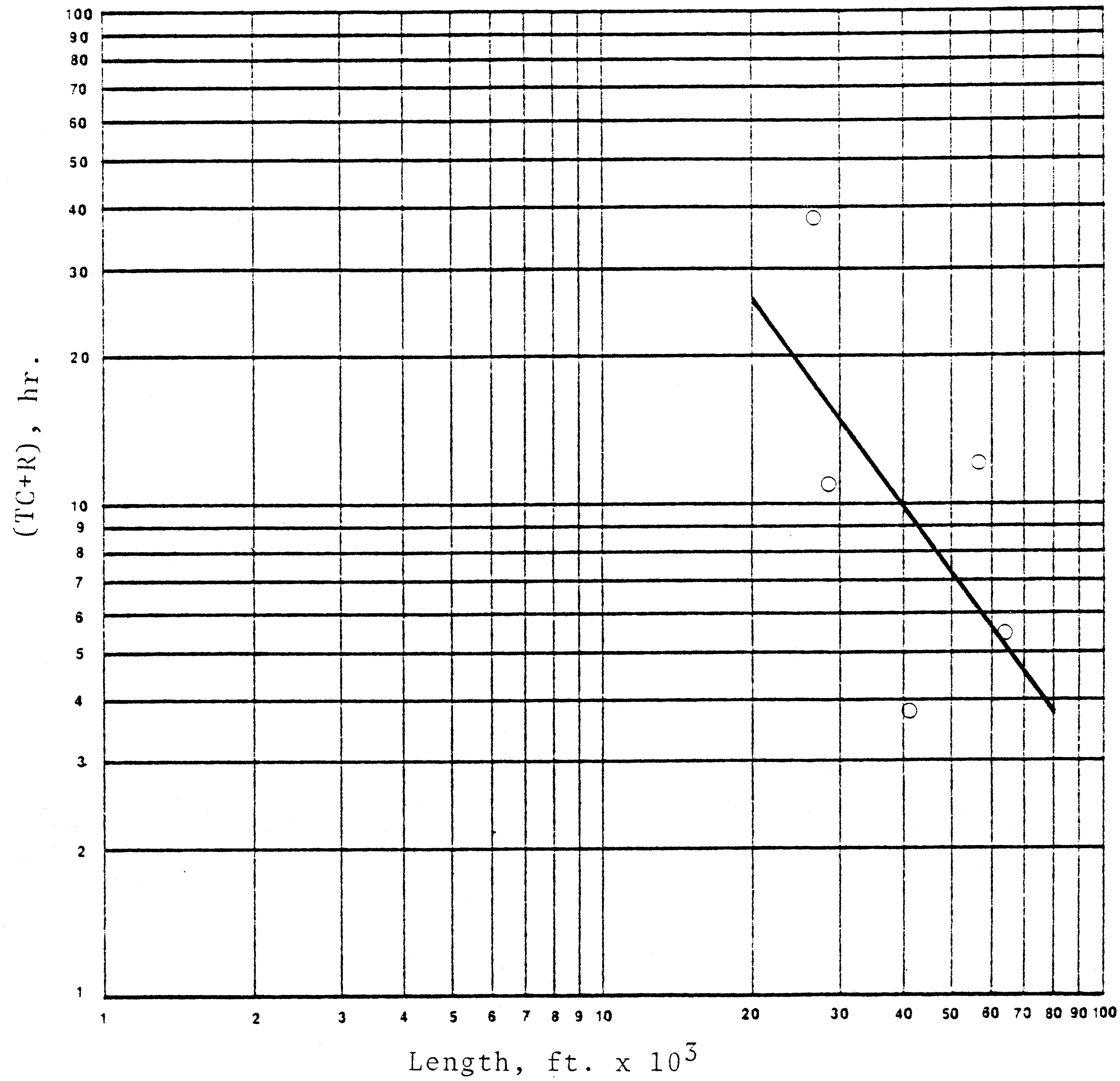


Figure 18. (TC+R) versus stream length.
1 ft. = 0.30 m.

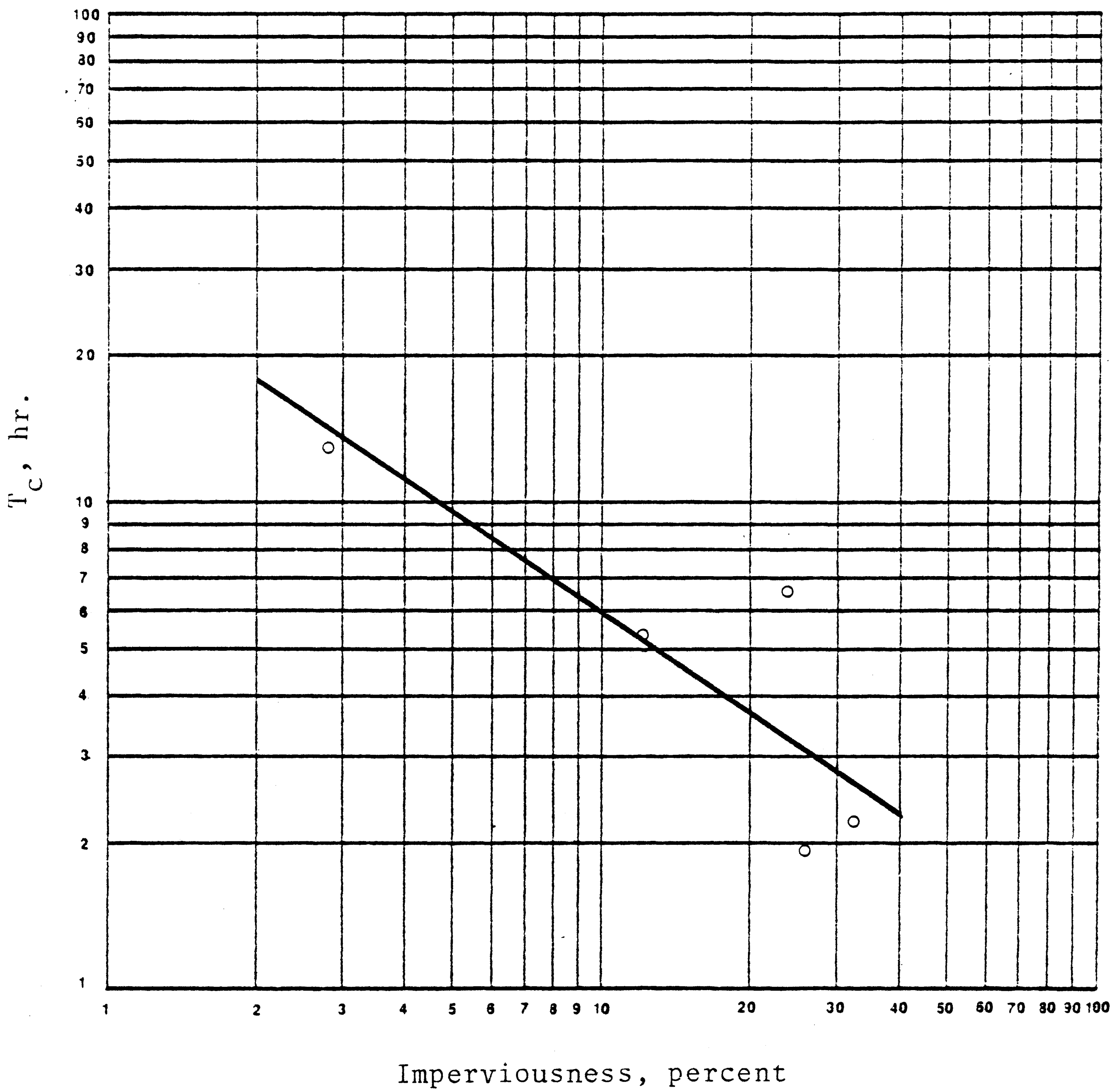


Figure 19. The relation of T_c to watershed imperviousness.

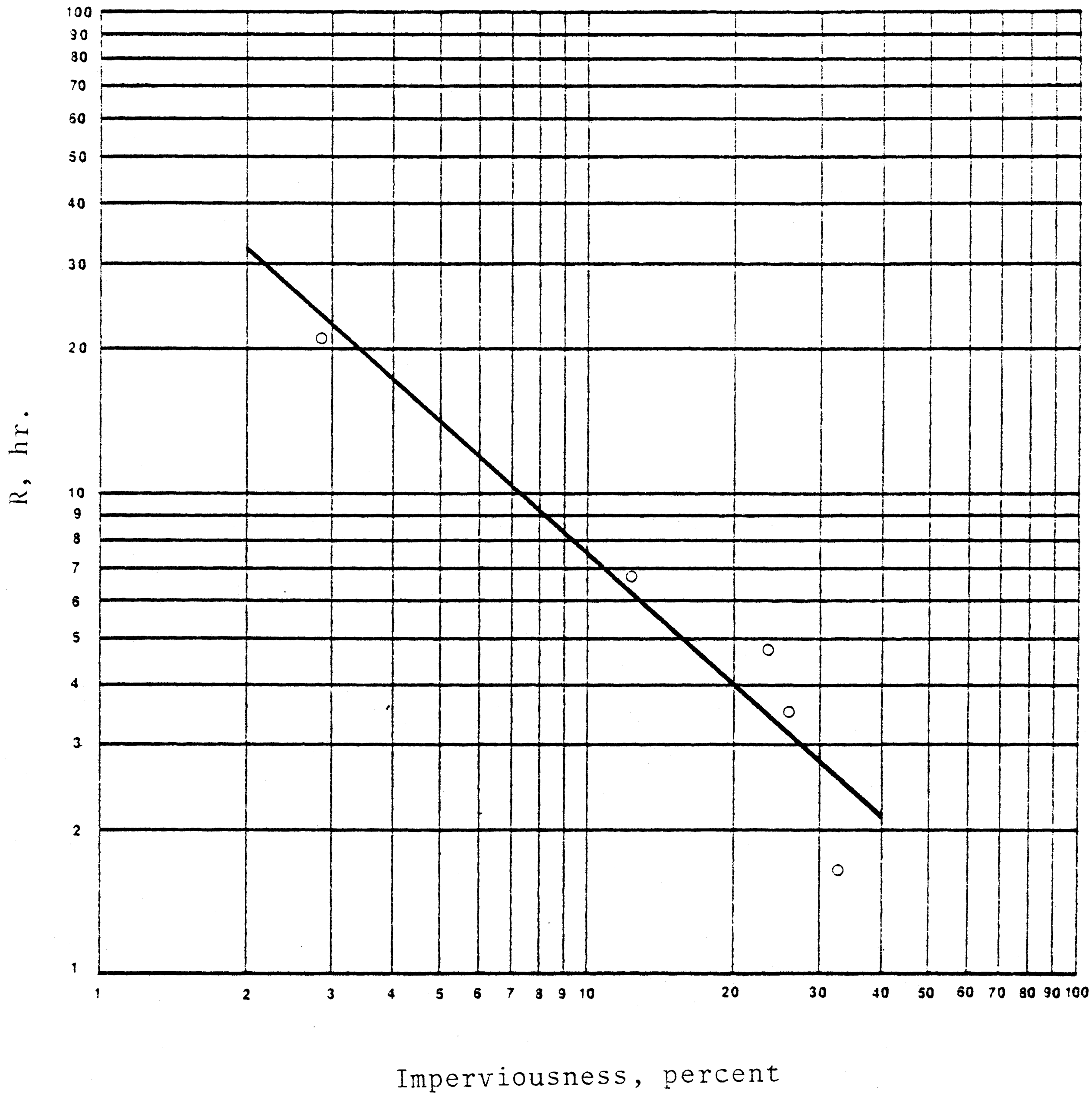


Figure 20. The relation of R to watershed imperviousness.

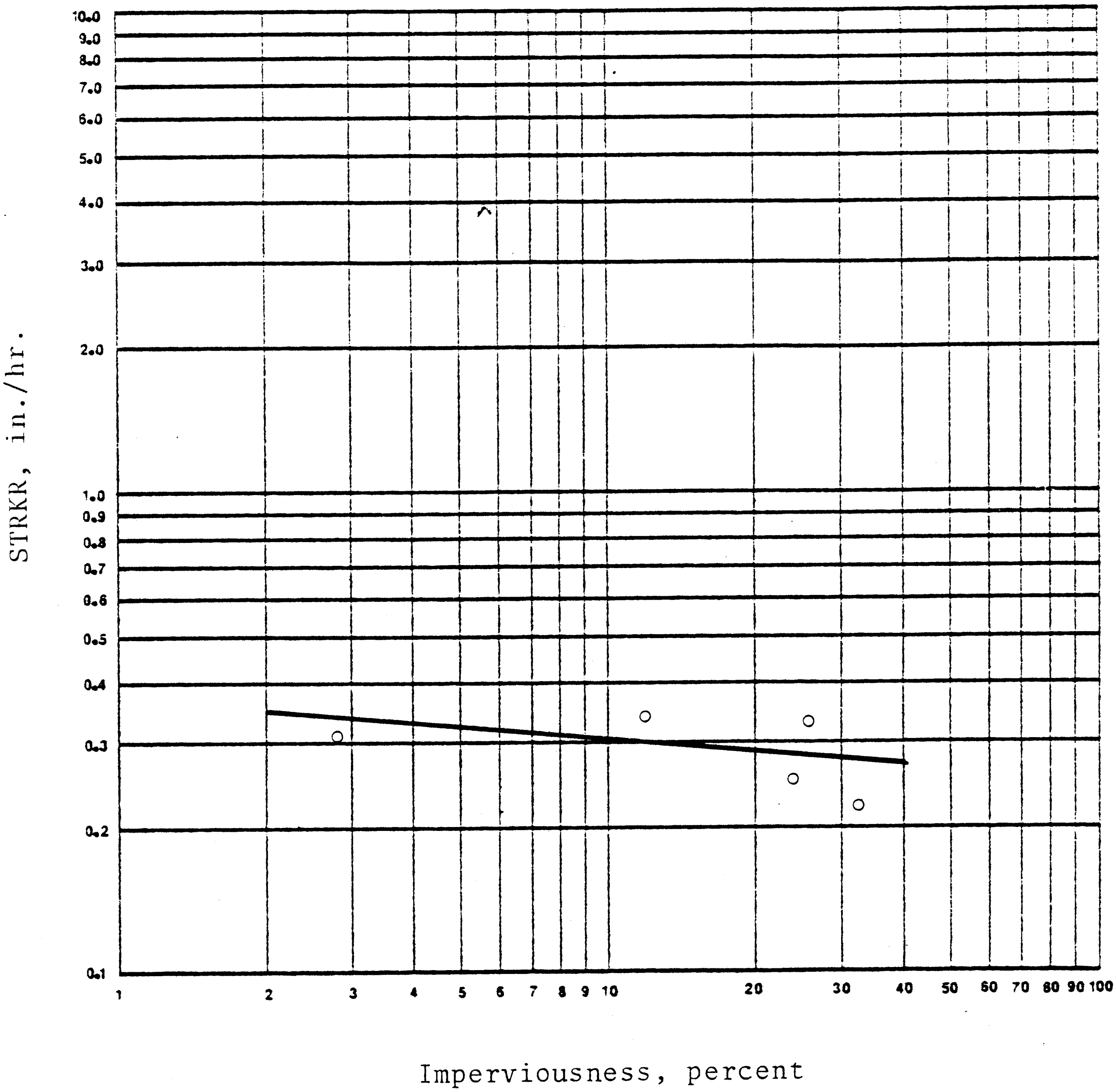


Figure 21. The relation of STRKR to watershed imperviousness.
1 in. = 2.54 cm.

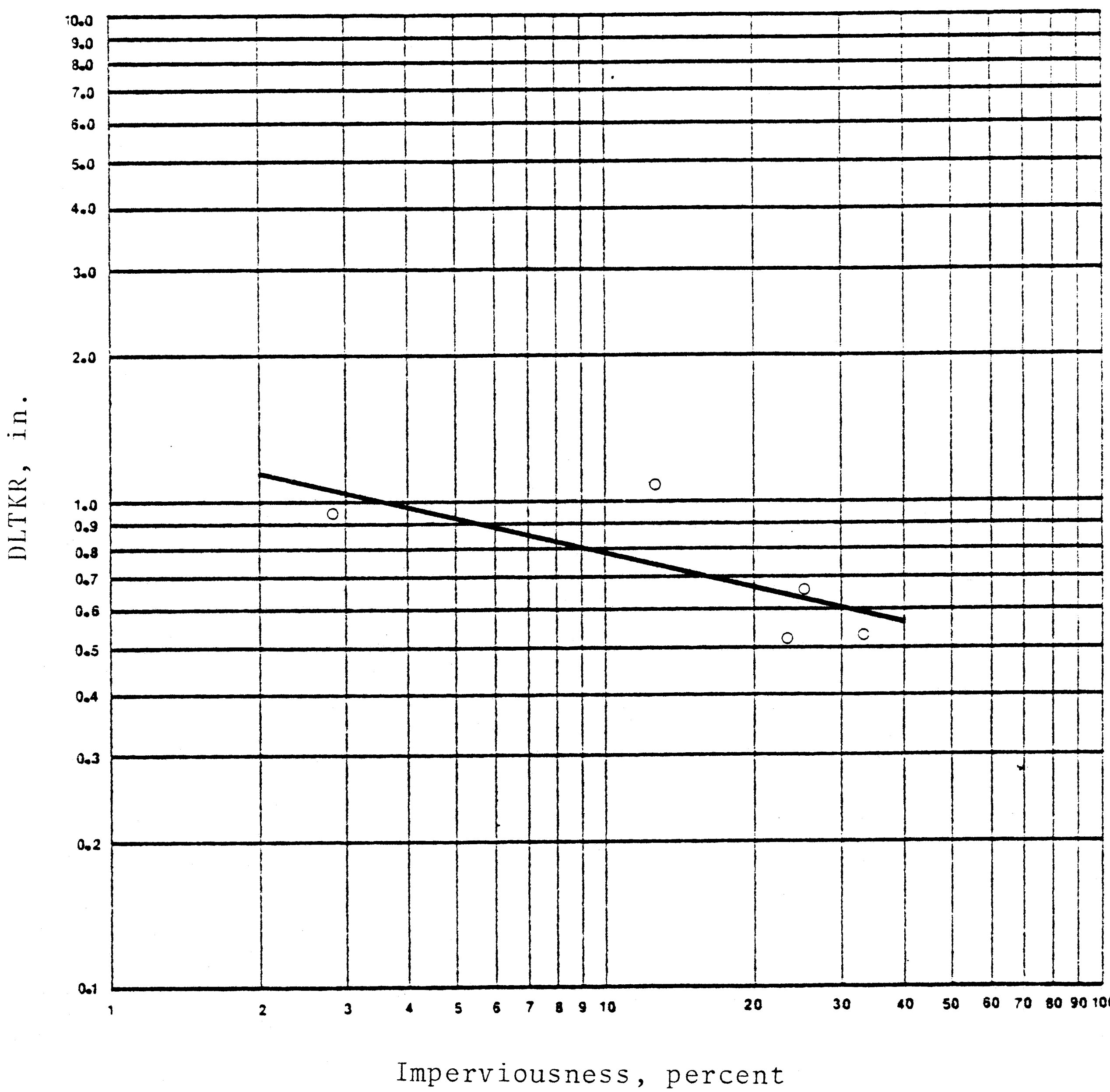


Figure 22. The relation of DLTKR to watershed imperviousness.
1 in. = 2.54 cm.

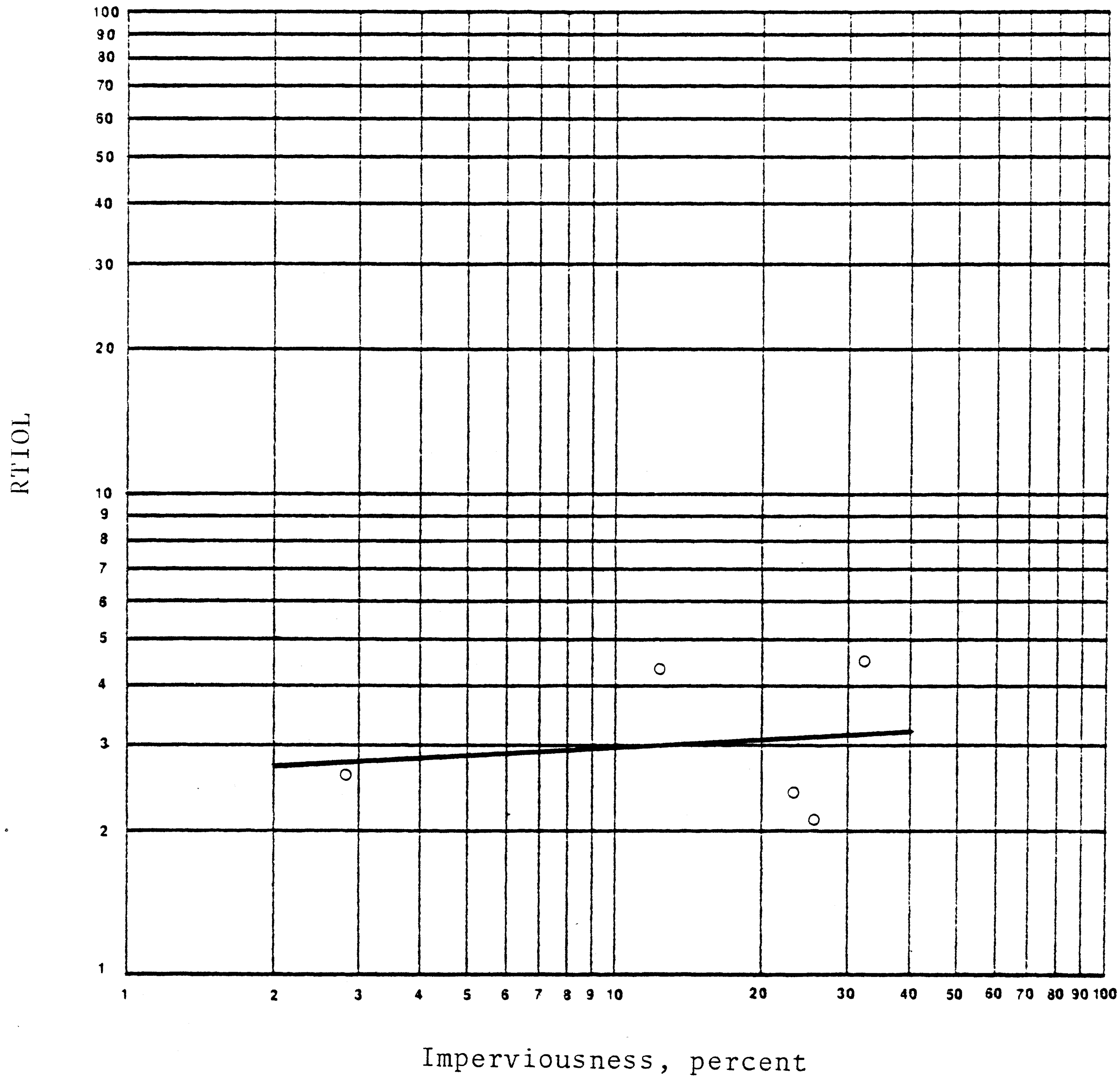


Figure 23. The relation of RTIOL to watershed imperviousness.

where

RC and R are in hours,

L in miles (1 mi. = 1.6 km), and

S in ft./mile (1 ft. = 0.305 m).

The equations obtained in this study are given in Table 6.

Table 6

Equations Relating HEC-1 Parameters to Watershed Imperviousness, Stream Slope, and Length

TC	=	$28.90\ (\text{IMPERVIOUSNESS})^{-0.68}$	(1)
R	=	$57.88\ (\text{IMPERVIOUSNESS})^{-0.89}$	(2)
STRKR	=	$0.37\ (\text{IMPERVIOUSNESS})^{-0.09}$	(3)
DLTKR	=	$1.37\ (\text{IMPERVIOUSNESS})^{-0.24}$	(4)
RTIOL	=	$2.65\ (\text{IMPERVIOUSNESS})^{0.05}$	(5)
QRCSN	=	$42.52\ (\text{IMPERVIOUSNESS})^{-0.44}$	(6)
RTIOR	=	$1.24\ (\text{IMPERVIOUSNESS})^{0.12}$	(7)
(TC+R)	=	$1382.88\ (\text{SLOPE})^{-1.49}$	(8)
(TC+R)	=	$20.13 \times 10^{6}\ (\text{LENGTH})^{-1.37}$	(9)
(TC+R)	=	$0.58\ (\text{SLOPE})^{-2.01}(\text{LENGTH})^{0.89}$	(10)

<u>Units</u>

TC, hours
R, hours
STRKR, inch/hr. (1 in. = 2.54 cm)
DLTKR, inches (1 in. = 2.54 cm)
RTIOL, dimensionless
Imperviousness, percent
Length, ft. (1 ft. = 0.305 m)
Slope, ft./mile (1 ft. = 0.305 m, 1 mi. = 1.6 km)
QRCSN, percent
RTIOR, dimensionless

$$(TC + R) = 0.58 L^{0.89} S^{-2.01}. \quad (5)$$

The Illinois technique includes estimating regional values for the ratio R/(TC+R) and then computing TC and R by using equation (4). Equation (5) needs to be further examined with more data before it is recommended for use in Virginia, but it does provide a convenient alternative to the method currently being used.

Figures 19 through 23 depict the relationship between the loss rate and unit hydrograph parameters and the percent imperviousness of the watershed. It can be seen that TC and R are both closely related to imperviousness (Figures 19 and 20). DLTKR also relates well to I (Figure 22), whereas STRKR decreases slightly as I increases (Figure 21), and RTIOL appears to increase somewhat with I (Figure 23).

MINITAB was used to obtain regression equations for all parameters with respect to imperviousness. These equations are given in Table 6.

In summary, the relationship between the parameters and imperviousness could be described as follows:

- Strongly related: TC, R
- Moderately related: DLTKR
- Weakly related: RTIOL, STRKR

Runoff Hydrograph Parameters

The HEC-1 computer model was used to establish the relation between the two runoff hydrograph variables, QRCSN and RTIOR, and watershed imperviousness, drainage area, peak discharge, and precipitation intensity. As discussed previously (Table 3), the QRCSN value is the percentage of peak flow at which the regression curve begins; RTIOR is the recession coefficient that is the ratio of flow at time t to ten computational periods later during recession. Initially, to reproduce the observed hydrographs, QRCSN and RTIOR were estimated as 5% and 1.5, respectively, for all HEC-1 optimization runs (Cruise and Yu 1982) and values for TC, R, and loss rate parameters were optimized for each storm according to the procedure previously described. To improve the reconstituted hydrographs, these optimized values were then held constant in subsequent optimization runs, and QRCSN and RTIOR were allowed to vary from their original estimates of 5% and 1.5. The final values of QRCSN and RTIOR that produced the best fit were averaged for Abrams Creek, Cameron Run, Fourmile Run and Totopotomoy Creek as shown in Table 7.

Although the changes in QRCSN and RTIOR improved the reconstituted hydrograph, they did not significantly alter the final loss rate parameters. The changes did seem to impact the Clark unit hydrograph values; TC generally increased and the R values decreased, although significantly so only in Abrams Creek and Cameron Run (see Table 7).

Both QRCSN and RTIOR seemed to be a function of the imperviousness of the watershed (Figures 24 and 25). By raising QRCSN and lowering RTIOR, a milder recession to the reconstituted hydrograph was produced, which is typical of a recession in a rural watershed. Conversely, lowering QRCSN and raising RTIOR produced a steeper recession, typical of urbanized areas. Therefore, QRCSN varied inversely with imperviousness; the largest values were found in Totopotomoy (a rural watershed) and the smallest values in Fourmile Run (an urban watershed). The relationship is expressed as

$$\text{QRSCN} = 42.5\ (\text{Imperviousness})^{-0.44}. \qquad (5)$$

The RTIOR values varied directly with imperviousness, with the largest values obtained for Fourmile Run. The relationship is expressed as

$$\text{RTIOR} = 1.24\ (\text{Imperviousness})^{.12}. \qquad (6)$$

The recommended best values for QRCSN and RTIOR would be approximately the following:

	QRCSN,%	RTIOR
Rural Watersheds	10-20	<1.5
Urban Watersheds	5-10	1.5-2

Table 7

Suggested QRCSN and RTIOR Values by Watershed

Watershed	QRCSN (percent)	RTIOR	TC (hr.)	R (hr.)	STRKR (in./hr.)	DLTKR (in.)	RTIOL
Abrams Creek	18.50	1.40	*	**	-	-	-
Cameron Run	13.00	1.63	-	**	-	-	-
Fourmile Run	6.10	2.34	-	-	-	-	-
Totopotomoy	32.50	1.38	-	-	-	-	-

*A significant increase (greater than 10%) occurred using suggested QRCSN and RTIOR values in the HEC-1 optimization.

**A significant decrease (greater than 10%) occurred using suggested QRCSN and RTIOR values in the HEC-1 optimization.

Conversion : 1 in. = 2.54 cm.

346

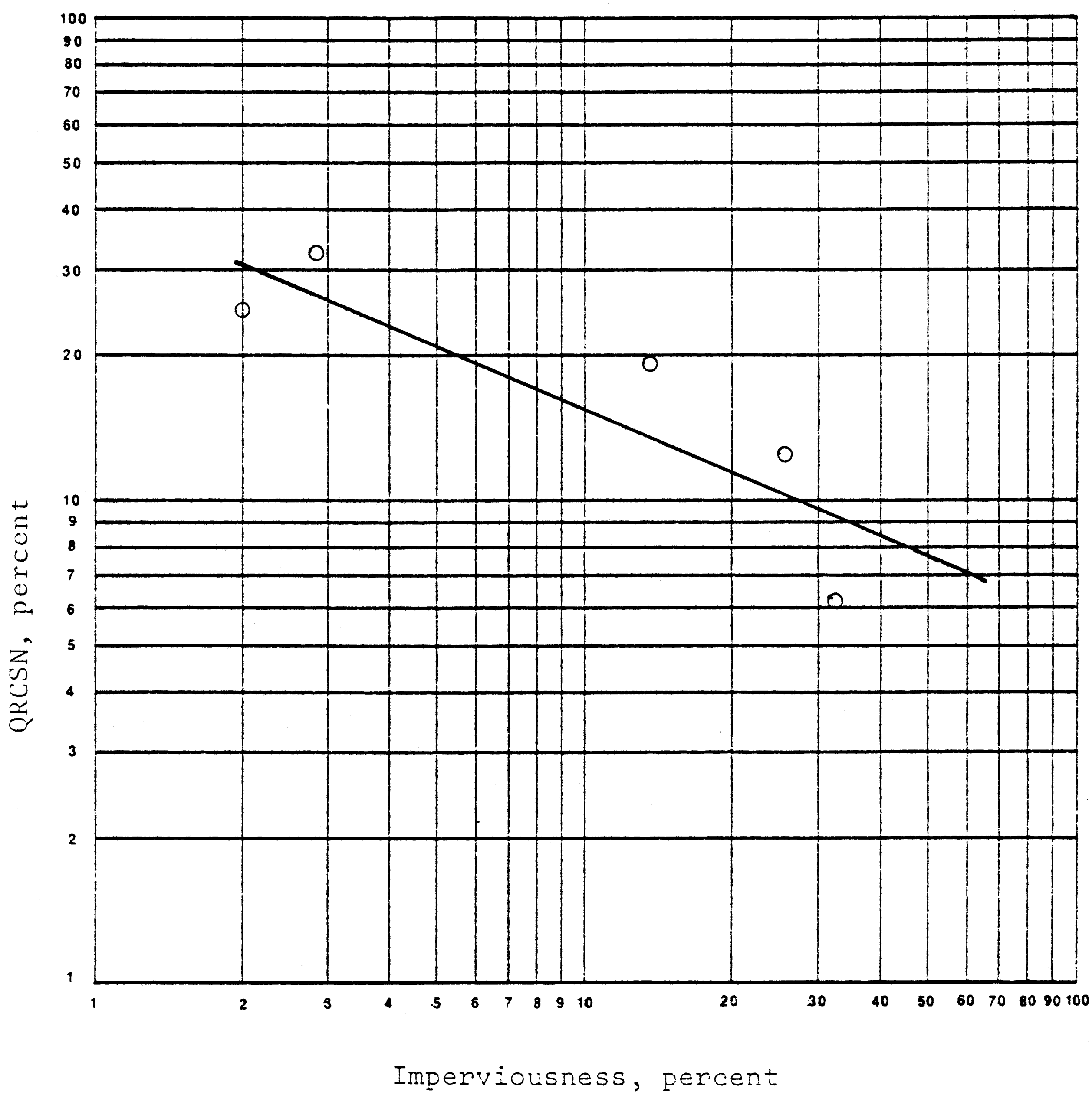


Figure 24. The relation of QRCSN to watershed imperviousness.

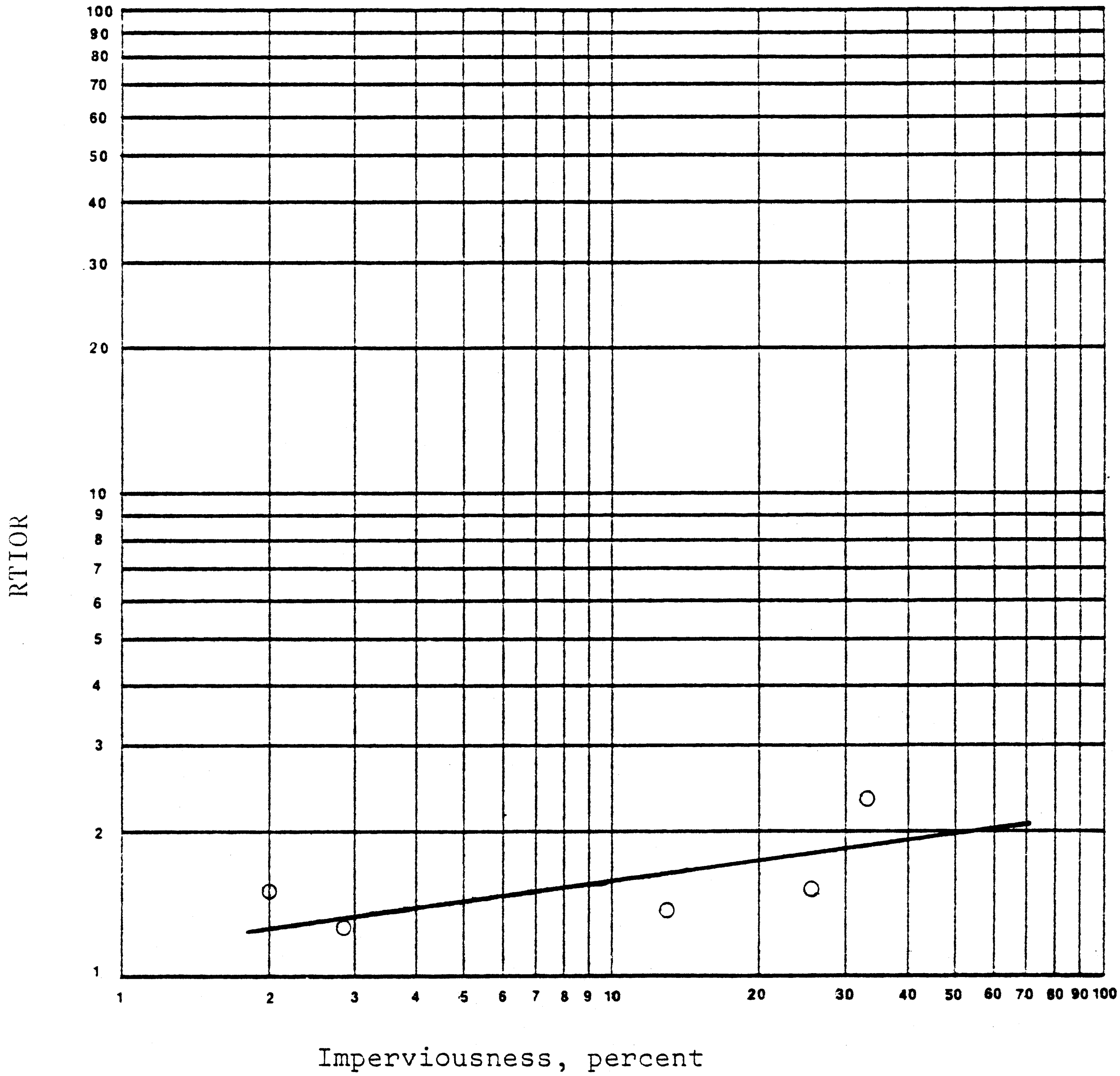


Figure 25. The relation of RTIOR to watershed imperviousness.

The U.S. Army Corps of Engineers has suggested a possible correlation of QRCSN to drainage area, peak discharge, or precipitation intensity (Corps of Engineers 1973). No such correlations were demonstrated in this study (Figures 26, 27, and 28). A possible relation between RTIOR and drainage area, peak discharge, and precipitation was also sought but none was found (Figures 29, 30, and 31).

RECOMMENDATIONS

Based on the information gathered for this study and the results obtained in the data analyses, the following recommendations are made regarding the urbanization effects on HEC-1 loss rate and hydrograph analysis.

1. It is recommended that the percent imperviousness be used as the index of urbanization for a watershed. Further refinement could be made by including area sewered in the index. The imperviousness can be determined from the population density as expressed in equation (3) and shown in Figure 4.

2. Modifications of the loss rate and unit hydrograph parameters for urbanization effects can be made by using the equations listed in Table 6 or the regression lines shown in Figures 19 through 23.

3. The runoff hydrograph parameters QRCSN and RTIOR can be adjusted to improve the accuracy of the HEC-1 model. It is suggested that QRCSN be set at about 5% to 10% of the peak discharge for urban watersheds and 10% to 20% for rural watersheds. RTIOR can be set at 1.5 or less for rural watersheds and 1.5 to 2.0 for urban watersheds.

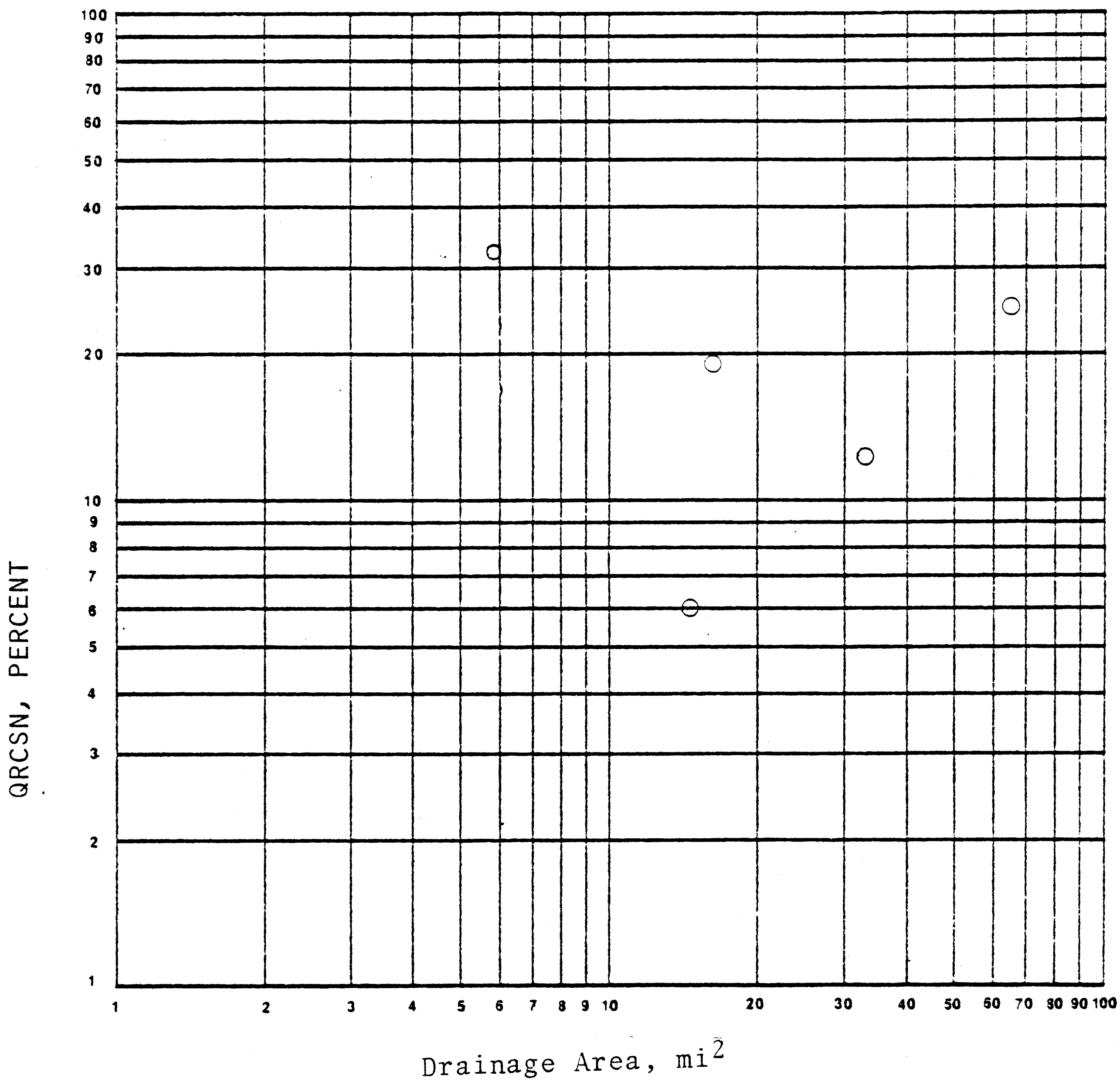


Figure 26. QRCSN versus watershed drainage area. 1 mi.^{2} = 2.6 km^2.

350

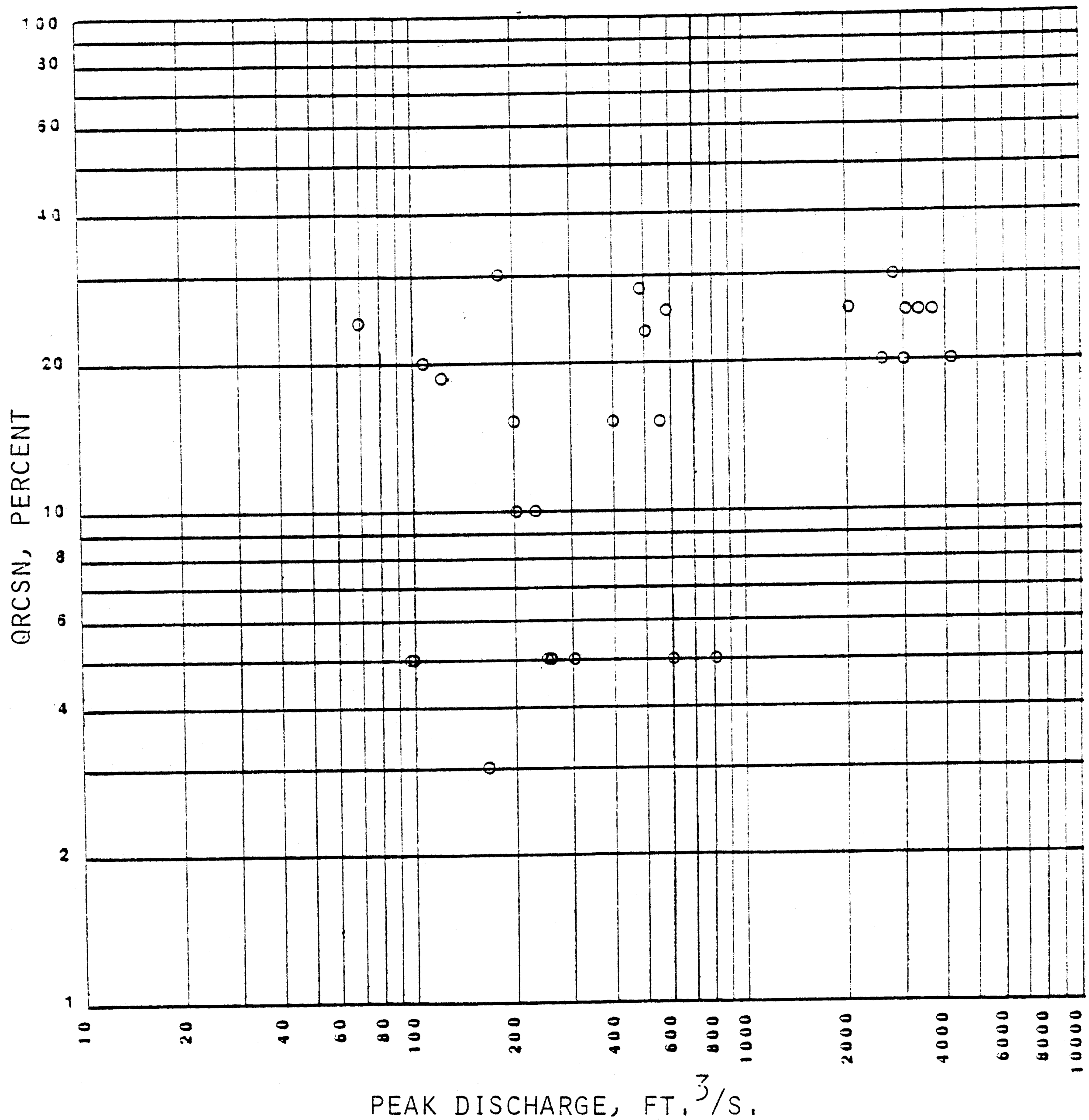


Figure 27. QRCSN versus peak discharge.
1 ft.^{3} = 0.03 m^3.

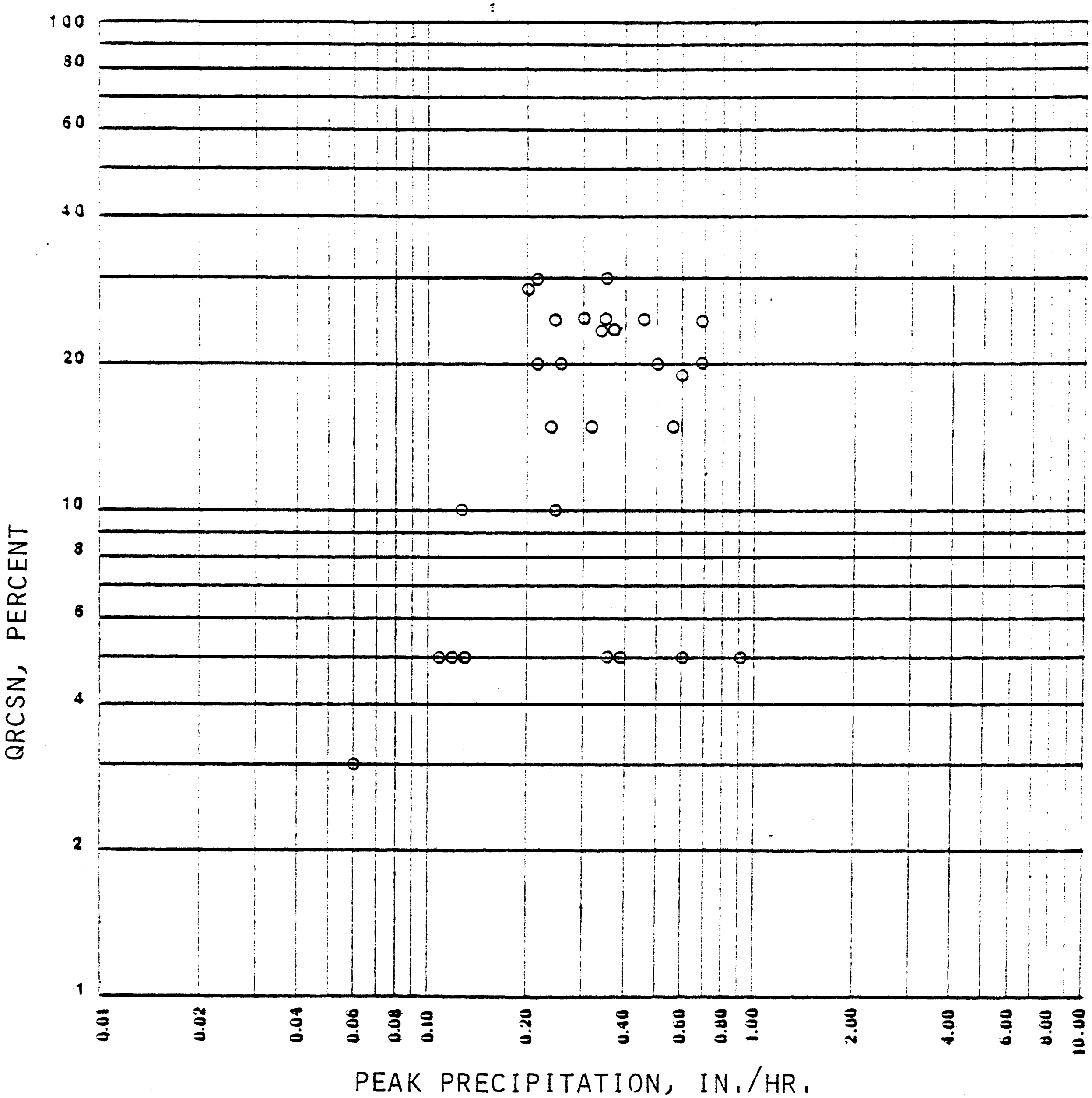


Figure 28. QRCSN versus peak precipitation.
1 in. = 2.54 cm.

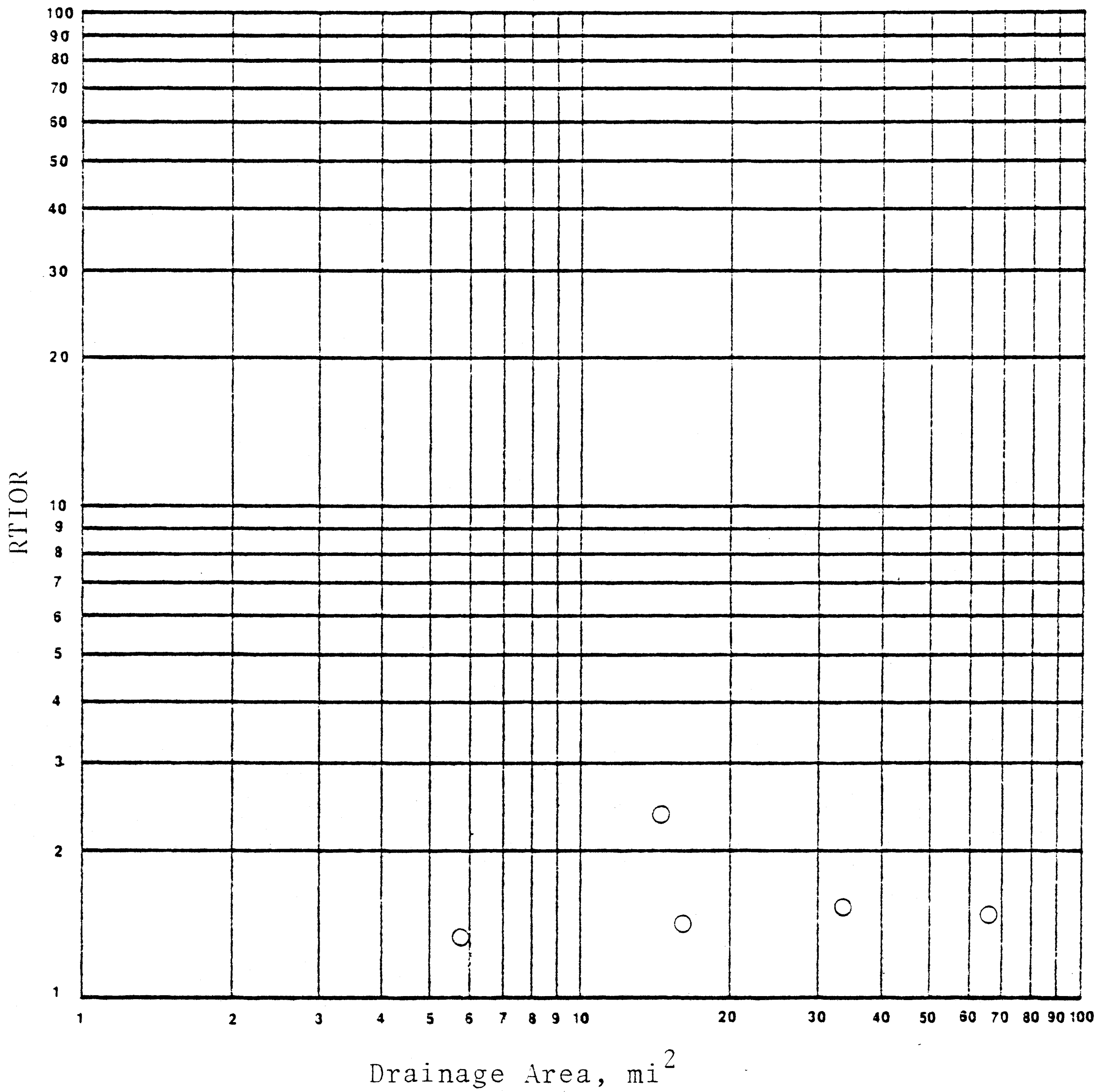


Figure 29. RTIOR versus watershed drainage area. 1 mi.^{2} = 2.6 km^2.

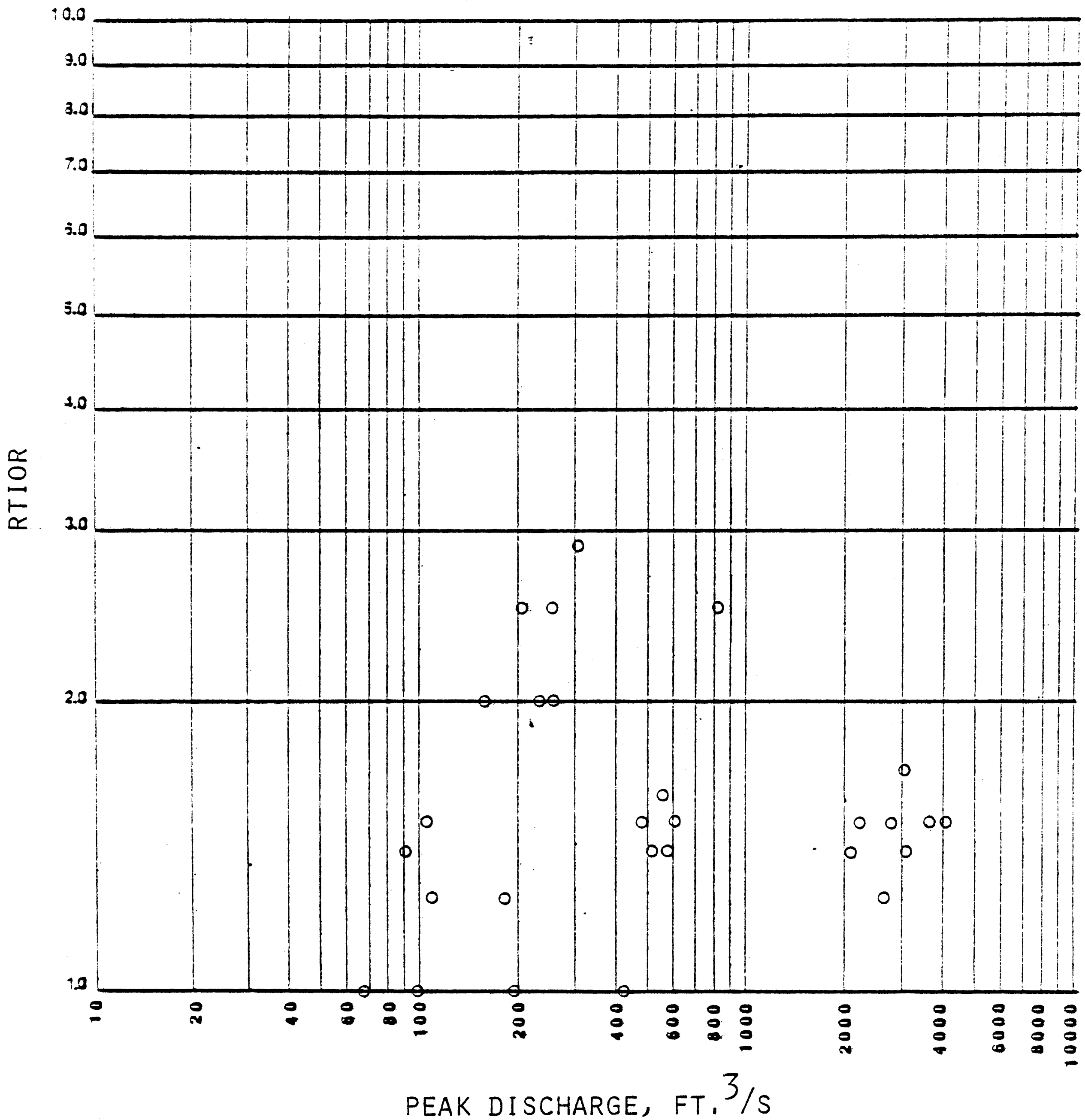
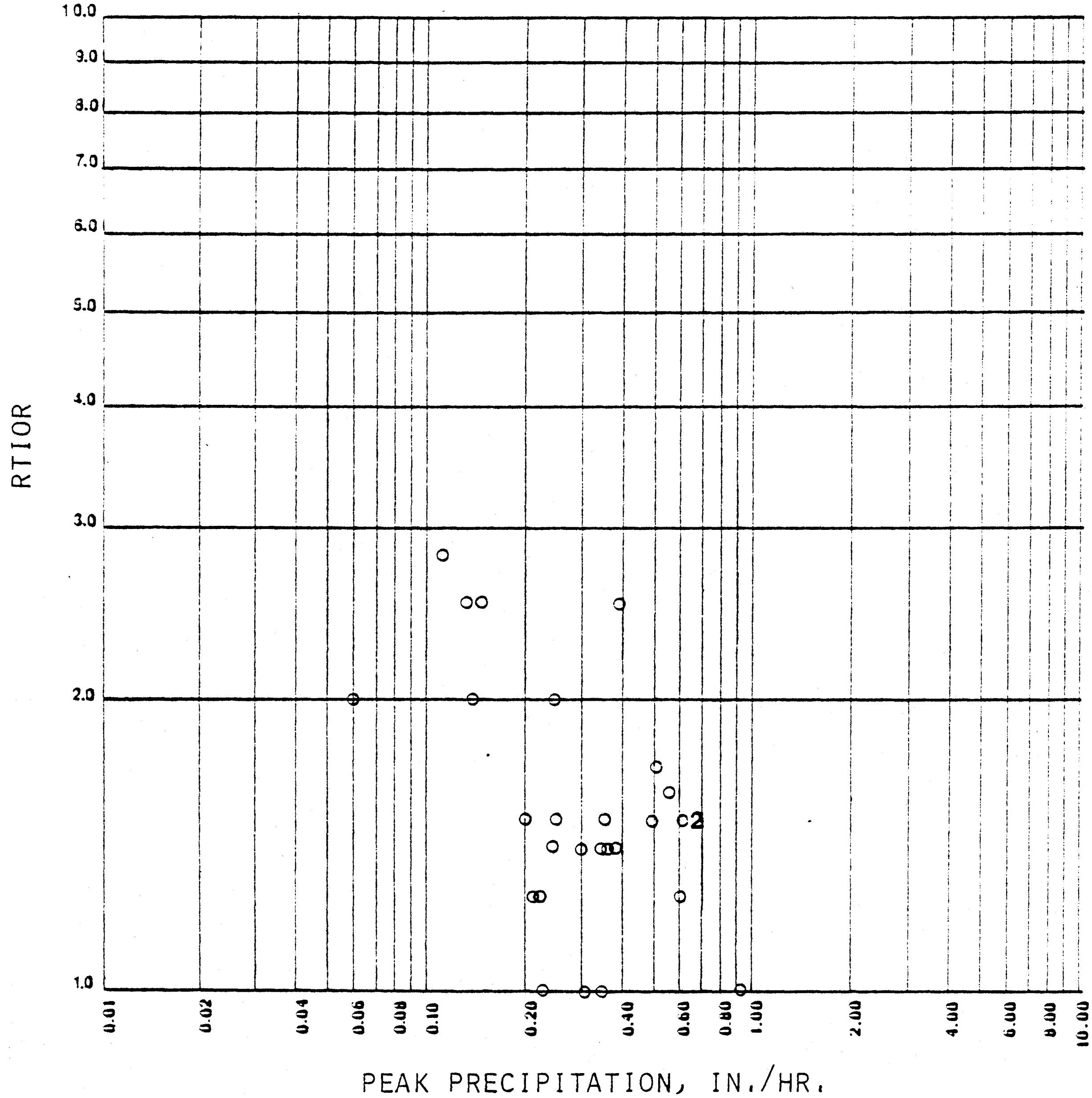


Figure 30. RTIOL versus peak discharge.
1 ft.^{3} = 0.03 m^3.



Note: 2 = two points of equivalent value.

Figure 31. RTIOR versus peak precipitation.
1 in. = 2.54 cm.

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358

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