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DEVELOPMENT OF A REGIONALIZED MATHEMATICAL MODEL FOR PREDICTING CHANGES IN STREAMFLOW QUANTITY AND QUALITY AS A FUNCTION OF LAND USE, SOIL TYPE AND RAINFALL CHARACTERISTICS.

10 Donald E. Overton, Roger A. Minear, Stephen P. Shelton

DEPT OF CIVIL ENGINEERING
THE UNIVERSITY OF TENNESSEE
KNOXVILLE, TN 37916

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This study was designed to regionalize a mathematical model stormwater runoff which can be used for predicting changes in streamflow quantity and quality as a function of land use, soil type, and rainfall characteristics. A water quality index was developed which responds as a function of land use and hydrologic characteristics. The index includes physical, chemical, and biological parameters.

The model was regionalized for immediate use in the Tennessee Valley on some watersheds up to approximately 10 square miles using hydrologic data from

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CONT

urban, agricultural, forested, and strip mined watersheds. However, in future studies the model can be regionalized to other parts of the United States by analysis of hydrologic air and water quality data in those regions utilizing the same scientific approach in this study in the Tennessee Valley. Results achieved from other regions can then be pooled with results achieved from the Tennessee Valley which would provide a more widely applicable model.

PREFACE

This report was prepared by Donald E. Overton and Roger A. Minear under contract F08635-76-C-0247, Job Order Number 19005W24, for Detachment 1 (CEEDO) HQ ADTC, Tyndall AFB FL.

This report summarizes work done between 1 January 76 and 1 February 77. Capt Stephen P. Shelton was the project officer.

This report has been reviewed by the Information Officer (IO) and is releasable to the National Technical Information Service (NTIS). At NTIS it will be available to the general public, including foreign nations.

This technical report has been reviewed and is approved for publication.

STEPHEN P. SHELTON, Capt USAF, BSC
Water Quality Research Engineer

EMIL C. FREIN, Maj, USAF
Chief, Environmental Engrg
and Energy Research Div

PETER A. CROWLEY, Maj, USAF, BSC
Director of Environics

JOSEPH S. PIZZUTO, Col, USAF, BSC
Commander

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TABLE OF CONTENTS

LIST OF FIGURES

LIST OF FIGURES (Concluded)

LIST OF TABLES

SECTION I
INTRODUCTION AND MODELING CONCEPTS

1. STORMWATER DEFINED

Stormwater is the direct response to rainfall. It is the runoff which enters a ditch, stream, or storm sewer which does not have a significant base-flow component. It is not assumed that all stormwater reaches an open channel by the overland flow route, although conceptually many of the models do not have an interflow or base-flow component. In urban areas, this should be a realistic approach because of the high degree of imperviousness; however, in some rural watersheds an overland flow component may be nonexistent and direct storm response may be only near the stream and occur as seepage through the banks (References 1 and 2).

As defined here, stormwater is associated with small upland or headwater watersheds where base flow is not a significant proportion of the total flow in the open channel during periods of rainfall. Hence, the attention here is directed principally at predicting watershed stormwater discharges as a function of land use and climate rather than predicting the water level along a river. The emphasis herein is upon the storm hydrograph rather than the stage hydrograph.

2. MATHEMATICAL MODELS

A mathematical model is a quantitative expression of a process or phenomenon one is observing, analyzing, or predicting. Since no process can be completely observed, any mathematical expression of a process will involve some element of stochasticism, i.e., uncertainty. Hence, any mathematical model formulated to represent a process or phenomenon will be conceptual to some extent and the reliability of the model will be based upon the extent to which it can be or has been verified. Model verification is a function of the data available to test scientifically the model and the resources available (time, manpower, and money) to perform the scientific tests. Since time, manpower, and money always have finite limits, decisions must be made by the modelers as to the degree of complexity the model is to have, and the extensiveness of the verification tests that are to be performed.

The initial task of the modeler then is to make decisions as to which to use or to build, how to verify it, and how to determine its statistical reliability in application, e.g., feasibility, planning, design, or management. This decision-making process is initiated by clearly formulating the objective of the modeling endeavor and placing it in the context of the available resources on the project for fulfilling the objective.

If the initial model form does not achieve the intended objective, then it becomes a matter of revising the model and repeating the experimental verifications until the project objective is met. Hence, mathematical modeling is by its nature heuristic and iterative. The choice of model revisions as well as the initial model structure will also be heavily affected by the range of choice of modeling concepts available to the modeler, and by the skill which the modeler has or can develop in applying them.

Figure 1 is a schematic representation of the modeling process. The modeling process is not new but is nothing more than a modern expression of the classical scientific thought processes involved in the design of an experiment. What is very new and which was not available to Darwin or Euler is that today a very large number of concepts can be evaluated efficiently in a very small amount of time at a relatively small expense. The mechanisms which permit these evaluations are the high speed digital computer and a body of analytical techniques called systems analysis. To be effective, the modeler must therefore be knowledgable and skilled in computer science and in the discipline of systems analysis. Calculus and differential equations are the basic requirements for developing as a systems analyst.

3. SYSTEMS TERMINOLOGY

There has been an evolution of systems-modeling jargon, and it is important to review its main parts before proceeding to modeling work.

A variable has no fixed value (e.g., daily rainfall) whereas a parameter is a constant whose value varies with the circumstances of its application (e.g., Manning n-value).

The distinction between linear and nonlinear systems is of paramount importance in understanding the mechanism of mathematical modeling. A linear system is defined mathematically by a linear differential equation, and principle of superposition applies and system response is only a function of the system. An example of a linear system representation is the unit hydrograph model. A nonlinear system is represented by a nonlinear differential equation and system response depends upon the system and the input intensity. An example of a nonlinear system representation is the equation of gradually varied open channel flow. It is well known that real world systems are very nonlinear, but linear representations have often been made (e.g., the Streeter-Phelps stream dissolved oxygen model) because of lack of knowledge of the system or because of the pressures exerted by the

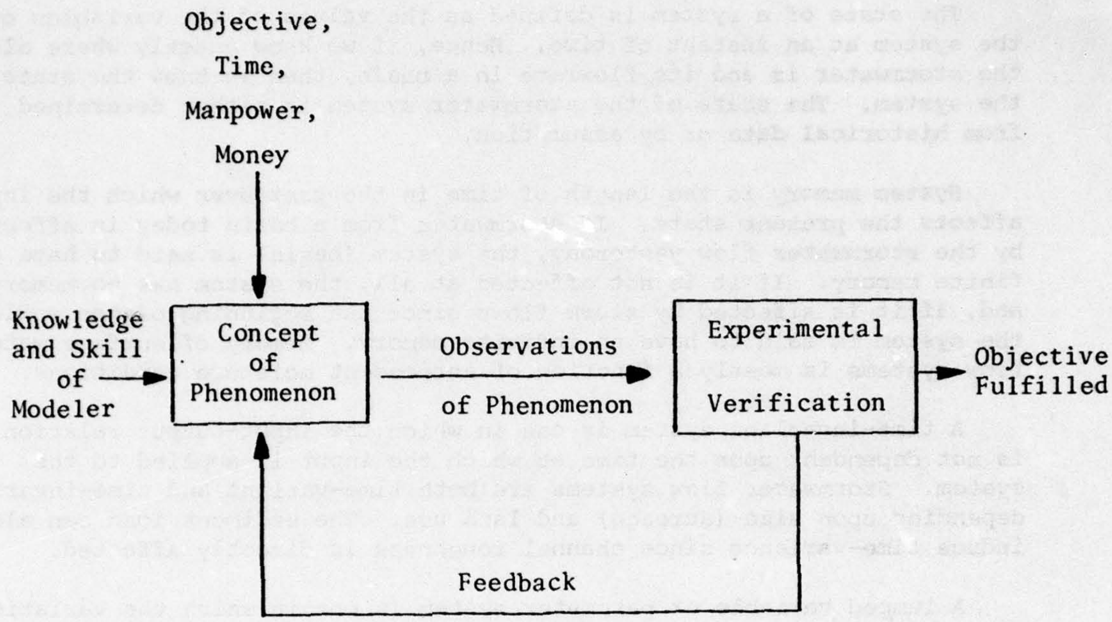

Figure 1. The Modeling Process (Reference 3)

resource constraint.

The state of a system is defined as the values of the variables of the system at an instant of time. Hence, if we know exactly where all of the stormwater is and its flowrate in a basin, then we know the state of the system. The state of the stormwater system is either determined from historical data or by assumption.

System memory is the length of time in the past over which the input affects the present state. If stormwater from a basin today is affected by the stormwater flow yesterday, the system (basin) is said to have a finite memory. If it is not affected at all, the system has no memory; and, if it is affected by storm flows since the beginning of the world, the system is said to have an infinite memory. Memory of surface water flow systems is mostly a function of antecedent moisture conditions.

A time-invariant system is one in which the input-output relation is not dependent upon the time at which the input is applied to the system. Stormwater flow systems are both time-variant and time-invariant depending upon size (acreage) and land use. The sediment load can also induce time-variance since channel roughness is directly affected.

A lumped variable or parameter system is one in which the variations in space either do not exist or have been ignored. The input is said to be lumped if rainfall into a system model is considered to be spatially uniform. Lumped systems are represented by ordinary differential equations and distributed systems are represented by partial differential equations.

A system is said to be stochastic if for a given input there is an element of chance or probability associated with obtaining a certain output. A deterministic system has no element of chance in it, hence for a given input a completely predictable output results for given initial and/or boundary values. A purely random process is a system with no deterministic component, and output is completely given to chance. A parametric or conceptual model does have an element of chance built into it since there will always be errors in verifying it on real data. It does therefore have a stochastic component. A "black box" model relates input to output by an arbitrary function, and therefore has no inherent physical significance.

4. THE MODELING APPROACH

Stormwater models are needed in land use planning if the consequences of development strategies on the water resource are to be evaluated. Even where actual data collected under land use conditions similar to that being proposed are available, differences in site characteristics will tend to invalidate results that are simply transferred. However, in the typical situation, very little if any data are available for directly assessing the consequences of alternate development strategies. As a result, mathematical models must be employed in the planning process. These models are needed to account for differences in site characteristics and to simulate the consequences of alternate development schemes.

There are two conceptual approaches that have been used in developing stormwater models. An approach often employed in urban planning has been termed deterministic modeling or system simulation. These models have a theoretical structure based upon physical laws and measures of initial and boundary conditions and input. When conditions are adequately described, the output from such a model should be known with a high degree of certainty. In reality, however, because of the complexity of the stormwater flow process, the number of physical measures required would make a complete model intractable. Simplifications and approximations must therefore be made. Since there are always a number of unknown model coefficients or parameters that cannot be directly or easily measured, it is required that the model be verified. This means that the results from usable deterministic models must be verified by being checked against real watershed data wherever such a model is to be applied.

The second conceptual stormwater approach has been termed parametric modeling. In this case, the models are somewhat less rigorously developed and generally simpler in approach. Model parameters are not necessarily defined as measurable physical entities although they are generally rational. Parameters for these models are determined by fitting the model to hydrologic data usually with an optimization technique.

The two modeling approaches thus appear to be similar and indeed for some subcomponent models, the differences are relatively minor. The real difference between the two approaches lies in the number of coefficients or parameters typically involved. The typical deterministic model has more processes included and thus more coefficients to be determined. Because of the inherent interactions among processes in nature, these coefficients become very difficult to determine in an optimum sense. Because of interactions within the model, a range of values for various coefficients may all yield similar results. Hence without rigorous model verification, the output from a deterministic model are suspect. The parameters in a parametric model on the other hand, are determined by optimization (objective best fitting).

Both modeling approaches require data before the model can be employed. The significant difference lies where the data must be located. For the deterministic model, the data should be available at the site of the application. For the parametric model, this latter requirement can be avoided by employing a two-step approach. The model can be fitted to data at locations where it is available in order to obtain optimum model parameters. These parameter values can then be correlated with the physical characteristics of the catchment or watershed. When this is done over a geographic area, the model is said to be regionalized. Once regional relationships between the site characteristics and model parameters are developed, it then becomes possible to measure the site characteristics at locations where water resource data are unavailable and to reliably predict the model parameters and hence, make scientifically based predictions.

There are advantages to both modeling approaches. If observed rainfall and storm hydrographs are available, then both approaches can be employed in the development of the stormwater system process models.

5. LINKAGE BETWEEN PARAMETRIC AND DETERMINISTIC MODELING

Parametric and deterministic stormwater models can be and should be complimentary. Parametric stormwater models such as the TVA model or the USGS model are lumped system representations, and they have the capability of assessing the gross effects of land use on runoff, but they do not have the capability of assessing the sensitivity of internal distributions of land use on runoff. Deterministic stormwater models such as the EPA model are distributed system representations and can simulate the transport mechanism at the source of runoff production to the basin outlet. The EPA model can also simulate stormwater flow through storm and combined sewers.

Hence, if the hydrologic and physical data are available for the study site, both modeling approaches can be effectively utilized. The parametric model can be utilized as the most scientific basis for predicting basin storm hydrographs on a regional basis, and the deterministic model can be utilized to investigate various land use scenarios on runoff and to simulate the transport mechanism including quality constituents. For the most reliable results, the two model types should be correlated. Further, as the deterministic approach provides information and improvements for the parametric approach, so will the parametric approach feedback information to indicate where further detailed specification is needed and where areas of the problem are most in need of further study.

6. CHOICE OF MODEL COMPLEXITY

The modeler must choose how complex a mathematical system representation should be made. As pointed out above, this choice is principally dictated by the project objectives, the knowledge and skill of the modeler, and resource constraints.

If a highly complex mathematical representation of the system under study is made, either parametric or deterministic, then the risk of not representing the system will be minimized but the difficulty of obtaining a solution will be maximized. Much data will be required, programming effort and computer time will be large, and the general complexity of the mathematical handling may even render the problem formulation intractable. Further, the resource constraints of time, money, and manpower may be exceeded. Hence, the modeler must determine the proper degree of complexity of the mathematical model such that the best problem solution will result and the effort will meet the project constraints. Conversely, if a greatly simplified mathematical model is selected or developed, the risk of not representing the system will be maximized but the difficulty in obtaining a solution will be minimized. The main point here is that the modeler must make a decision from the range of choice of models available or from the models which could be built. But, as pointed out in Figure 1, refinements in the model can be made by the modeler and indeed this is usually true.

Figure 2 is called the "trade-off diagram" (Reference 3) because it illustrates the consequences of the decision of how complex the model should be. If after preliminary verification, it is determined that the initially chosen model is either too complex or not complex enough, then the modeler may move along the abscissa scale in Figure 2 and experiment with another degree of complexity. This modeling effort should continue until the project objective is attained within the resource constraint.

7. MODEL OPTIMIZATION BY OBJECTIVE BEST FITTING

Since parametric models are conceptual, a set of unknown coefficients or parameters will appear in the mathematical formulation. The parameter values in the model are experimentally determined in the verification procedure. Intuitively, the proper coefficient values would produce the best fit or linkage between storm rainfall (input) and the stormwater hydrograph (output). There is an instinctive temptation, which has appeared in modeling literature, to derive model parameters from observed storms by trial-and-error "eyeballing" best fit procedures. There are certain distinct and far reaching disadvantages associated with this approach to model verification. They are:

1. If the model is of average complexity, about four or five parameters, then there are a very large number if not an infinite set of coefficients which will produce essentially the same fit.

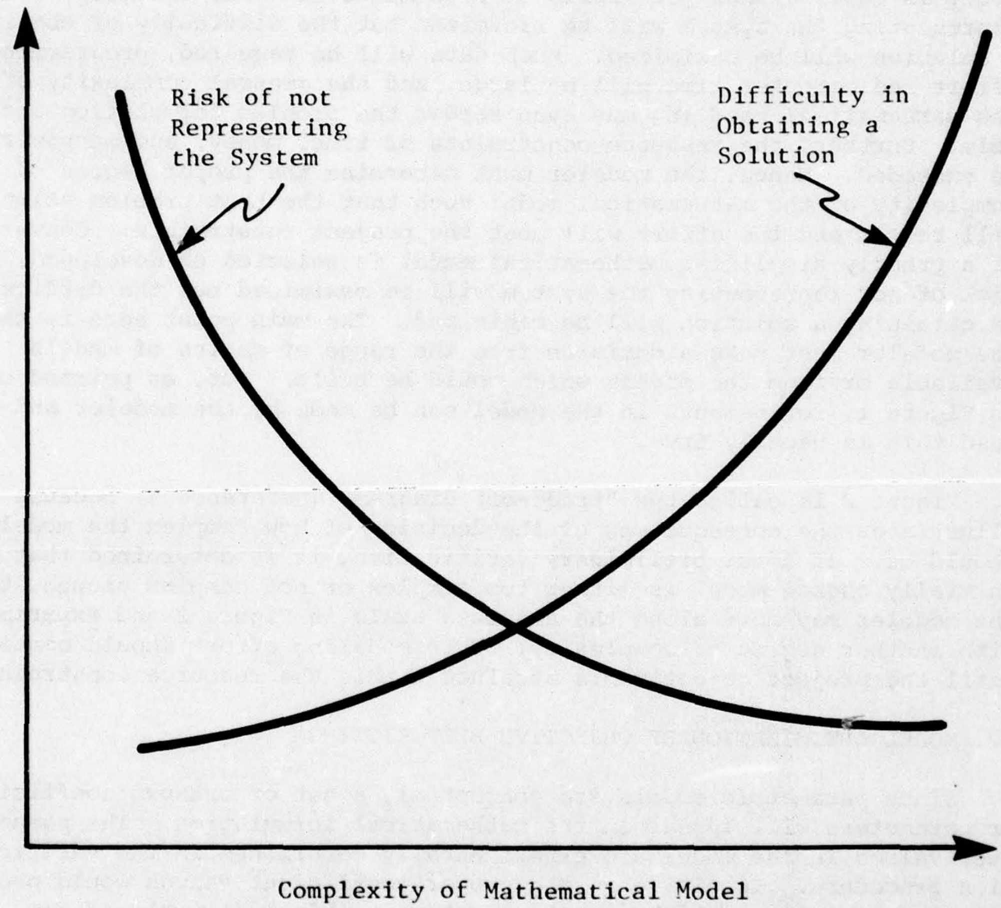

Figure 2. The Trade-Off Diagram (Reference 3)

Hence, a large operational bias is induced into the modeling process and attributing physical significance to and regionalizing the model parameters may be precluded.

2. If the goodness of fit between the model and the observed stormwater hydrograph is not quantified, the "eyeballing" technique itself induces another operational bias and the same negative effects as (1) above will result.

3. The trial-and-error process is very time consuming and inefficient. Time constraints will permit but a relatively few number of computer trials.

Parameter optimization in parametric stormwater models is achieved scientifically and economically by utilization of objective best fit criteria rather than by a trial-and-error "eyeballing" process. However, there is a limit to the size and complexity of a parametric model which can be optimized using an objective best fit criteria. The US Geological Survey model has nine parameters and has been successfully optimized. The Stanford model (developed at Stanford University Research Institute, Palo Alto, CA) of which there are many versions, has at least 20 parameters and optimization by objective best fitting has not as yet been accomplished and is seemingly intractable.

8. SENSITIVITY ANALYSIS

Model verification is not complete without a sensitivity analysis. Once the calibrated parameters are arrived at by a best fit procedure, sensitivity analysis proceeds by holding all parameters constant but one, and perturbating the last one such that variation of the objective function (measure of fit between the observed storm hydrograph and the fitted model) can be examined. If small perturbations of the parameter produce large changes in the objective function, the system is said to be sensitive to that parameter. This gives a measure of how accurate that parameter must be estimated if the model is to be used in prediction. If the objective function is not sensitive to the perturbated parameter, then the parameter need not be accurately estimated in prediction. If the system is extremely insensitive to the perturbated parameter, the parameter and its associated system component may be redundant or insignificant and could be deleted from the model.

9. REGIONALIZATION OF PARAMETERS

The effectiveness of parametric stormwater models will be measured, in the long run, by the confidence modelers have in their ability to estimate model parameters on basins which have no hydrologic data for calibrating the model being utilized. A high level of confidence could be achieved if enough bench mark watersheds with hydrologic data were available for analysis. Optimized model parameters for each basin could then be related to physiographic, land use, and climatic characteristics of the study basin. This would permit an interpolation and extrapolation of the results to ungauged basins within the study region at some specified confidence level. There have been very few reported attempts at parameter regionalization in the open literature. Primarily, this has been the result of a general lack of hydrologic data.

SECTION II

TECHNICAL DISCUSSION OF APPROACH

1. CALIBRATION OF TVA STORMWATER MODEL

a. Description of Model

Although thousands of stream gauges have been in operation in this country for decades, there is often little streamflow data available for small watershed planning applications. The reason is that water resource planning has focused largely upon basins hundreds of square miles in size. Today a considerable amount of planning is involved in smaller drainage basins such as in urban development and the 208 sections of PL 92-500, and in flood plain zoning. In smaller drainage areas the effect of land use, soils, and physiographic characteristics upon stormwater is profound. Further, there are a multitude of smaller basins to be considered, and the probability of finding a gauge on the stream involved or one similar to it is very small.

The paucity of data and lack of modeling effort has stagnated our limited knowledge of the hydrology of smaller basins and has left us with the inability to transfer stormwater information from one basin to another. Such a transfer cannot be done until stormwater response at gauged basins is related to the characteristics of each watershed. Once this is done, each stream gauge becomes a part of the statistical sample of hydrologic responses for the watersheds in the sample. Each additional watershed added to the sample provides more information about the relationships involved. The regionalization effort is seen to be a continuing effort and when it is substantially complete, it becomes possible to draw an inference of stormwater response on areas where data are not available and simulations can be reliably made.

Ardis (Reference 4) developed a model to compute storm hydrographs at gauged or ungauged sites in the Tennessee Valley from rainfall data and watershed characteristics. The model uses a unit response function to represent the response of a watershed to a given storm. The unit response function is a quadrilateral that can be formed by adding together two triangles. It is referred to herein as the TVA stormwater model. The shape of the response function is very flexible and allows the model to meet the response shape characteristics of most of the storms analyzed.

Four parameters are needed to define the TVA stormwater model. The parameters have significant variation both within and among the watersheds studied. This response variation is found to be nonlinear and significantly related to storm and watershed characteristics. The model parameters were regionalized and these relations are incorporated in the regionalization scheme presented in this study. Ardis' data base consisted of 11 small watersheds; it can be used as a planning tool. To this end, the TVA stormwater model can be used on a regional basis for evaluating the effects of land use changes on stormwater response on small watersheds.

Ardis recognized that the choice of a model depends upon the objectives it intends to fulfill while living with the limitations of data reliability. The model should be only as complicated as needed to solve the problem and satisfy the objectives. An overly complex model needlessly complicates the problem thus adding to the analysis load and required input. This would inhibit its prospective use, while too little detail in the model may not yield satisfactory results. This is another way of explaining the "trade-off diagram" shown earlier (Figure 2). Furthermore, a large number of model parameters will result in a higher risk that a unique solution will not be obtained in parameter optimization.

In the data set available for calibrating the TVA stormwater model, only infrequently could one rain gauge per 20 square miles be found. Hence, rainfall was assumed uniform over the basins studied and the system was considered to be lumped rather than distributed.

It was also found that the watershed systems studied were characterized by long term rather than short term time variance. This means that the response function could be considered to be time invariant throughout a storm but may be time variant from storm to storm.

It is well known from surface water hydraulics and experimental hydrology that stormwater response varies during a storm and is nonlinear. Ardis attributes this effect mostly to the partial area runoff concept rather than to the dependence of the response function on input intensity. However, in regionalizing the stormwater model parameters, Ardis did recognize a significant nonlinear system effect due to rain excess intensity. The TVA stormwater model was conceptualized to account for a quick and a delayed response which characterized the partial area contribution effect.

Wanting to keep the shape of a triangle for stormwater response and to incorporate a quick and delayed response, Ardis developed a double-triangle unit response function. This was based upon the concept that the heaviest runoff into the stream is derived first from the riparian wet areas and that other areas contribute later as their soils become saturated. At the same time, the riparian areas grow in size. This concept results in an initial response and a delayed response that together form a unit response function ("urf") for a given storm and basin system.

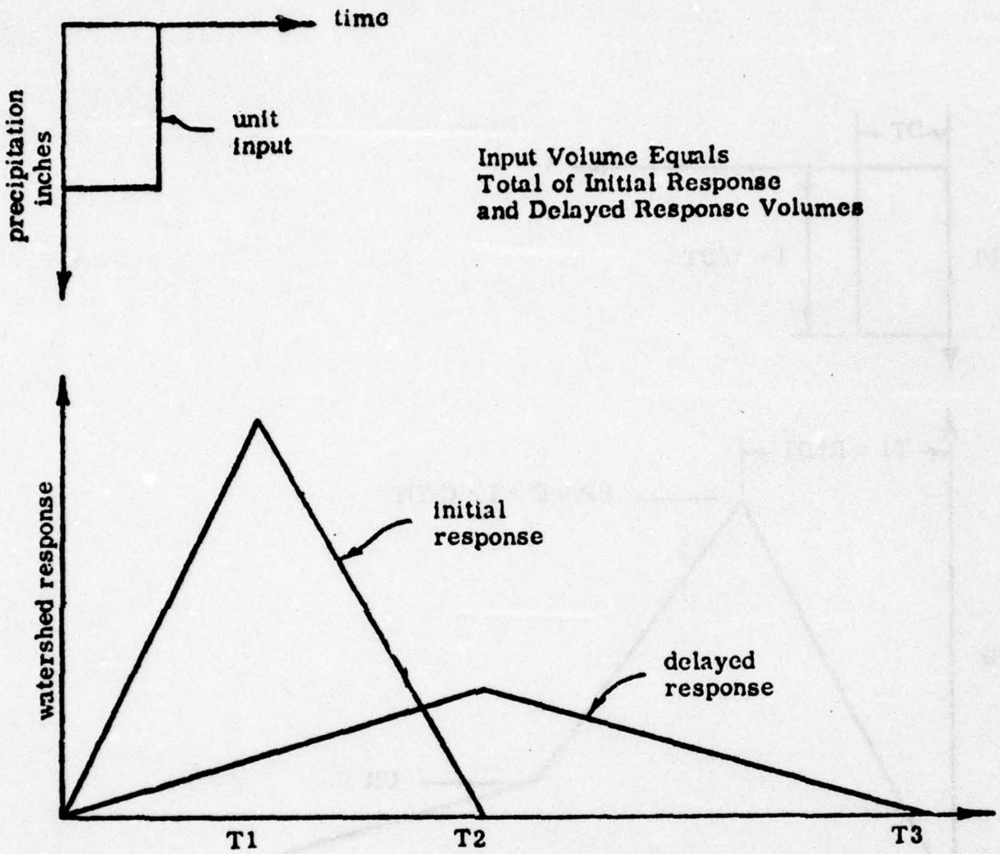

Figure 3. Partial Area Runoff Concept Represented by Initial and Delayed Response (Reference 4)

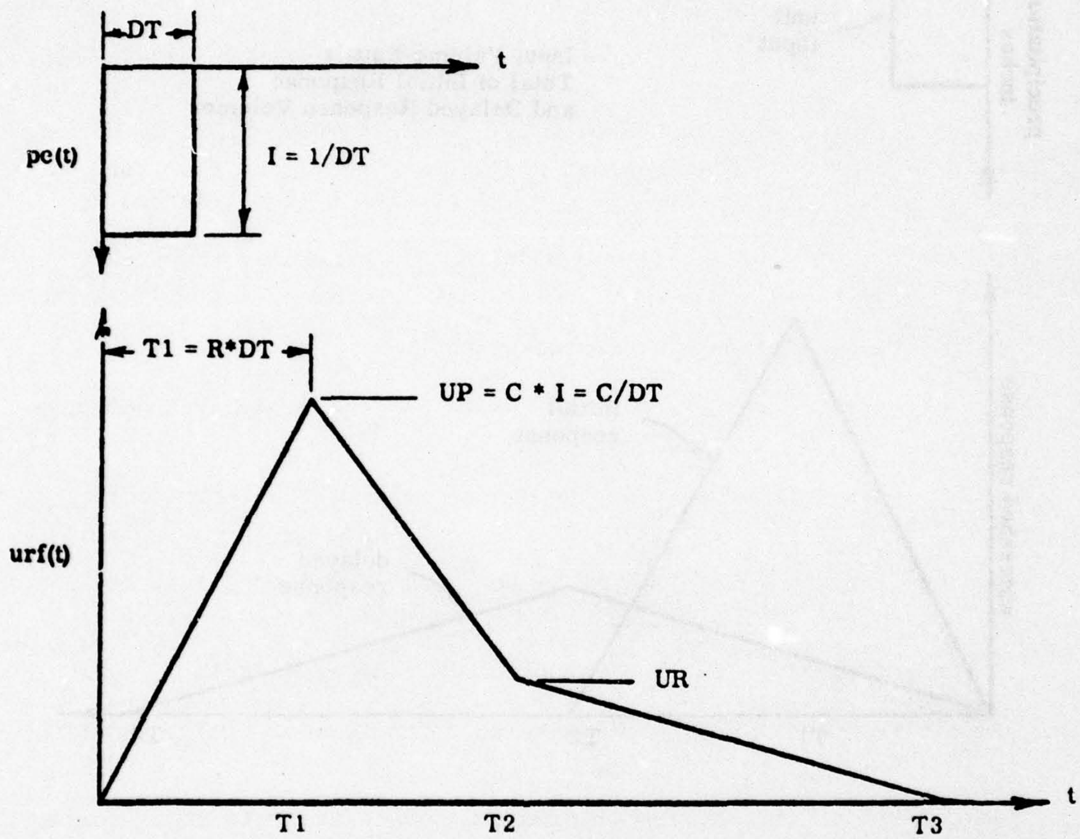

Figure 4. Four-Parameter "urf" of TVA Stormwater Model (Reference 4)

This double-triangle response is represented in Figure 3. It was assumed that the delayed response peaks where the initial response ends and that both responses begin at the same time. The resulting four-parameter "urf" is found by superposition and is shown in Figure 4.

The symbols used in Figures 3 and 4 are:

- I Precipitation excess intensity (in/hr). Since the volume of input is 1 basin-in, I = 1/DR, where
- DT Time interval (hr) used for abstracting rainfall and discharge records.
- C Dimensionless multiplier of I related to UP, the ordinate of the double-triangle model at T1. It was chosen as such for its similarity to the C in Q = CIA of the "rational method."
- UP Ordinate of double-triangle model at T1, generally the peak (in/hr).
- T1 Time to peak of initial response (hr).
- T2 Time base of initial response = time to peak of delayed response (hr).
- T3 Time base of delayed response = time base of double-triangle model (hr).
- R Dimensionless multiplier of DT to equal the time of peak of the initial response, T1.
- pe(t) Precipitation excess as a function of time t (in/hr).
- urf(t) unit response function ordinate as a function of time t (in/hr).

The basic four parameters defining the double-triangle model for a unit response function are UP, T1, T2, and T3. Each parameter measures a specific attribute of the model. To maintain unit volume, the variable UR is determined from the four basic parameters:

$$UR = \frac{2 - (UP*T2)}{T3 - T1} \qquad (1)$$

(* used throughout as multiplication symbol)

Since DT was selected to equal one hour in Ardis' study, C is equal to UP and R is equal to T1. Also, all unit response functions described hereafter are equivalent to one-hour unit hydrographs.

Figure 5 is an example of the double triangle's flexibility for a constant time base T3. It can assume most conceivable shapes and can be fitted to them to approximate response behavior.

Base-flow separation was employed to differentiate between fast and slow response and to eliminate the slow response. These are not the same as the initial and delayed response described earlier. Fast response corresponds with the rapid stormflow associated directly with the storm rainfall as opposed to the attenuated recessional flow from saturated soils that typically occurs several days following the storm rainfall.

Ardis evaluated 12 different base-flow separation technique shapes. He found that methods other than a single straight line between point of rise and end of fast response or two straight-line segments to remove an antecedent recession showed little advantage. Although Hewlett and Hibbert (Reference 5) found that variations in separation criteria had little effect on response characteristics, their work indicated that any technique used must be reasonable and consistent since it was found that selection of the point of rise and end of fast response has a very sensitive effect on the resulting stormwater hydrograph.

Fast response ends where contributions to the total hydrograph normally considered as direct surface runoff are no longer represented at the watershed outlet. On the TVA study watersheds, this point was selected where the rate of change in total discharge became essentially constant. Selection of this point varied as much as 12 hours for the size of watersheds in Ardis' study. However, corresponding volume estimates remained below a maximum difference of 10 percent. A typical base-flow separation technique is shown in Figure 6. Except for two of the study watersheds, an average value of T3 (time base of double-triangle model) was then selected for each watershed since individual differences were found to be small. Nonlinear behavior was observed during smaller floods, hence, a constant T3 would not be valid. An average value of T3 having been selected, B was redetermined so that NOBS could be determined consistent with

$$\underline{NOBS} = \underline{NPE} - \underline{DT} + \underline{T3}/\underline{DT} \qquad (2)$$

where NPE is the number of periods, in multiples of DT, of precipitation excess estimated from the rainfall hyetograph, NOBS the number of storm-

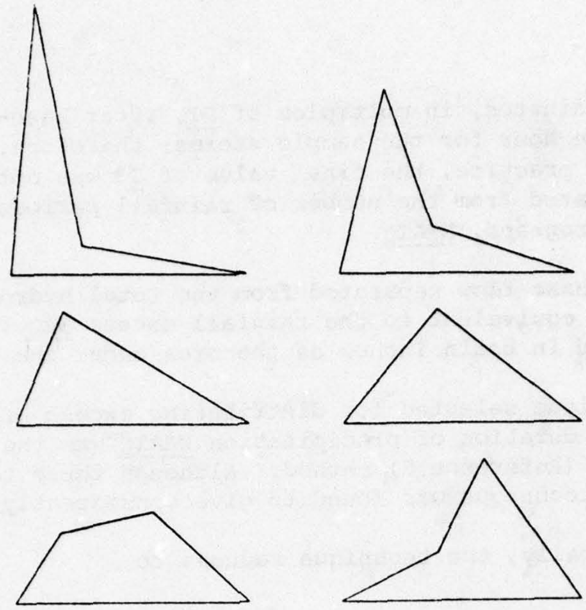

Figure 5. Flexibility of the double-triangle model used as a unit response function. Time base T3 is constant. (Reference 4)

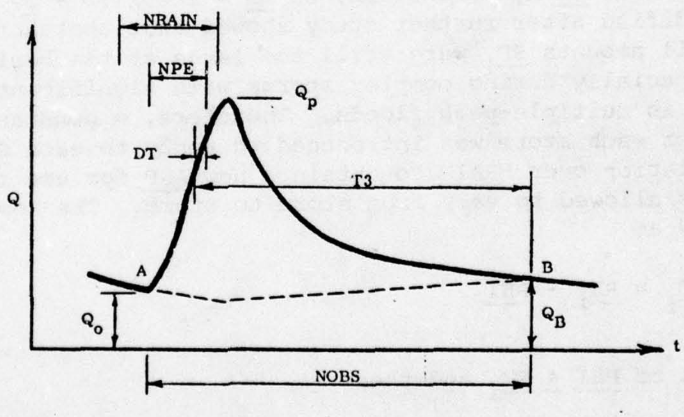

Figure 6. Typical base-flow separation procedure. (Reference 4)

hydrograph ordinates, in multiples of DT, after base-flow separation, and DT the one hour for the sample storms; therefore, NOBS and NPE are in hours. In practice, the final value of T3 was not constant since NPE was estimated from the number of rainfall periods associated with the storm hydrograph, NRAIN.

With the base flow separated from the total hydrograph, the remaining volume is equivalent to the rainfall excess SRO. The volume SRO was calculated in basin inches as the area under the storm hydrograph.

The technique selected for distributing excess precipitation in time over the duration of precipitation NRAIN was the Soil Conservation Service (SCS) (Reference 6) method. Although three techniques were tested, this technique was found to give consistently good results.

Mathematically, the technique reduces to

$$\underline{SRO} = \frac{(\underline{ARF} - \underline{IA})^2}{\underline{ARF} - \underline{IA} + \underline{S}} \tag{3}$$

where ARF is the accumulated rainfall, IA an initial abstraction from ARF, S the maximum potential retention which is related to the SCS curve number CN by definition, as CN = 1000/(10 + S). The SCS technique was modified after further study showed that abstractions from hourly rainfall amounts $\underline{RF}_i$ were still too large at the beginning of storms and especially during complex storms with significant lulls which resulted in multiple-peak floods. Therefore, a constant loss parameter, PHI, for each storm was introduced to apply to each $\underline{RF}_i$ prior to accumulation over NRAIN to obtain a new ARF for use in Equation 3. PHI was allowed to vary from storm to storm. The new $\underline{RF}_i$, $\underline{NRF}_i$, is then defined as

$$\underline{NRF}_i = \underline{RF}_i - \underline{PHI} \tag{4}$$

subject to $\underline{PHI} \leq \underline{RF}_i$ and then

$$\underline{ARF}_i = \sum_{o}^{i} \underline{NRF}_i \tag{5}$$

Once the S or CN for a particular storm was found, it was held constant and a vector of $\underline{SRO}_i$ values was determined from the $\underline{ARF}_i$ vector as suggested by the SCS. The time-incremental values of precipitation excess $\underline{PE}_i$ were determined as in Equation 6.

$$\underline{PE}_i = SRO_i - SRO_{i-1} \tag{6}$$

Analysis of each storm hydrograph involves determining five parameters: C, R, T2, T3, and PHI. With DT = 1, these are equivalent to UP, T1, T2, T3, and PHI respectively, as previously described and shown in Figure 4. T3 was determined by a modification of Equation 2 and is an integer.

$$\underline{T3} = (\underline{NOBS} - \underline{NRAIN} + 1) * \underline{DT} \tag{7}$$

where the number of observations NOBS is redefined to begin coincident with NRAIN. To assure that in convolution all of the double triangle is used, T1 and T2 are also required to be integers. Figure 7 shows how portions of the double triangle may not be used when nonintegers are used. For very peaked double triangles, the shape can be drastically modified. Since the double triangle can reduce to a single triangle to meet such a need, the following restriction was also imposed.

$$\underline{DT} = 1 \leq \underline{T1} < \underline{T2} \leq \underline{T3} \tag{8}$$

Storm hydrographs with distinct peaks, caused by lulls in long-duration rainfall storms, were separated during analysis and treated as separate bursts. Such complex hudrographs were separated based upon an exponential decay from an existing portion of a falling or recession limb just prior to an increase caused by the next rainfall burst. Each burst of rainfall was used with its associated portion of the complex hydrograph to evaluate the time distribution of precipitation excess for that burst. For each burst, S in Equation 3 was held constant. Although S could vary among bursts model parameters and PHI were required to be constant for all bursts in a given storm.

Initial estimates of the double-triangle parameters define an "urf" which, along with PHI, made up a set of parameters that were improved by optimization. Since linearity was assumed for each storm, the convolution of calculated precipitation excess with the double-triangle, unit response function model resulted in a predicted storm hydrograph which was compared with the observed storm hydrograph during optimization. Optimization was performed using PATSEAR, the pattern search routine of Green (Reference 7), and the best fit criterion used was the minimization of the weighted sums of squares of errors SSE. All errors where the observed storm hydrograph ordinates were greater than 0.1 times the maximum observed discharge were assigned a weight of 1.0; all others were assigned a weight of 0.5.

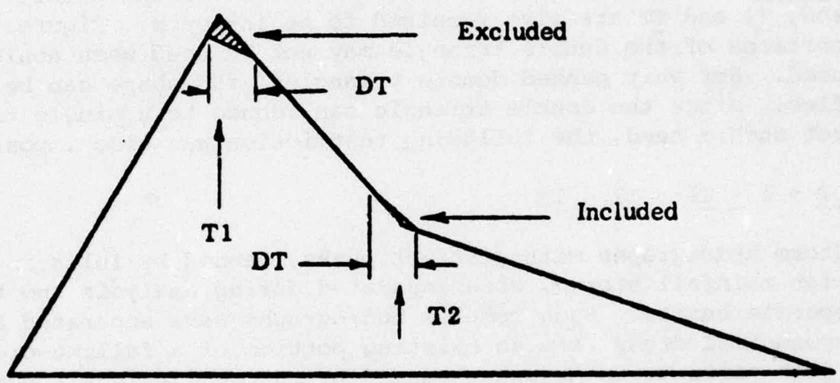

Figure 7. Potential Shape Modification If T1 and T2 Are Not Restricted to Integers (Reference 4)

b. Data Base

The model has been calibrated using data from 24 watersheds. Table 1 lists the 24 watersheds modeled by Ardis (Reference 4) and Betson (Reference 8). Two of the watersheds in Table 1 are outside of the Tennessee Valley: Boneyard Creek and Browns Creek drain urban watersheds and both are US Geological Survey gauges. Calibration was extended in this study to six watersheds in a strip mining basin plus an additional 1½ years of data from the Knoxville urban watersheds which were not included in the regionalization scheme to be presented subsequently.

c. Results of Calibration

A total of 354 storms were used in the calibrations for the watersheds in Table 1. Of the 354 storms, 100 were from the urban watersheds. The average coefficient of variation for all 354 calibrations was 85 percent, and an example of an average fit of the model to a storm is shown in Figure 8. The model storm hydrograph is synthesized by convoluting the derived response function with the associated rainfall excess hydrograph.

Multiple-peaked storms were included in the set of storm hydrographs used to calibrate the model. To handle these storms the storm hydrograph analysis program contains a rainfall burst analysis feature. With this feature, when the beginning and end points of individual bursts are identified (to a total of three), the analysis program will separate the burst hydrographs and determine an individual rainfall and runoff volume for each. This feature is important because it allows large storms, which are typically complex, to be included in the analysis and it permits associating the runoff volume and precipitation excess duration, SRO and NPE respectively, with the major storm peak. When the model is used to simulate storm hydrographs the process is reversed. First, PHI is applied to the rainfall to determine any constant loss. The rainfall is next converted to precipitation excess and then the precipitation excess is screened to detect any lulls that may exist. A lull is defined as a period of time in excess of a present limit with all precipitation excess less than a limit, generally 0.01 inches or less. For each separate burst of rainfall detected, individual SRO and NPE values are determined. The model therefore can vary the unit hydrographs used during the simulation of complex storms.

d. Effects of Watershed and Rainfall Variables

A regionalization scheme for the Stormwater Model was presented by Betson (Reference 8). The optimized four model parameters were related

TABLE 1. CALIBRATION WATERSHEDS FOR TVA STORM HYDROGRAPH MODEL

Watershed	Drainage Area km^2	Drainage Area (mi^2)	Physiographic Province*	Percent Impervious	Percent Forest	Annual Rainfall cm	Annual Rainfall (in)
Crab Cr. - Penrose, NC	28.2	(10.9)	BR	0	83	183	(72)
Boylston Br. - Horseshoe, NC	38.3	(14.8)	BR	0	64	145	(57)
S. F. Mills R. - Pink Beds, NC	25.9	(9.99)	BR	0	100	175	(69)
Allen Cr. - Hazlewood, NC	37.3	(14.4)	BR	0	99	145	(57)
N. Indian Cr. - Unicoi, TN	41.2	(15.9)	BR	0	79	135	(53)
Noland Cr. - Bryson City, NC	35.7	(13.8)	BR	0	100	173	(68)
N. F. Citico Cr. - Tellico Plains, TN	18.2	(7.04)	BR	0	100	196	(77)
L. Chestuee Cr. - Wilson Sta., TN	21.3	(8.24)	VR	0	49	135	(53)
Cane Cr. - Sandy Hill, TN	43.5	(16.8)	MA	0	23	124	(49)
Upper Bear Cr. AL (SA2)	18.6	(7.18)	HR	0	42	142	(56)
Upper Bear Cr. AL (SF2)	36.0	(13.9)	HR	0	85	147	(58)
Upper Bear Cr. AL (SF1)	6.42	(2.48)	HR	0	100	142	(56)
Upper Bear Cr. AL (S2)	93.8	(36.2)	HR	0	62	142	(56)
Upper Bear Cr. AL (SC1)	2.25	(0.87)	HR	0	25	140	(55)
Sewee Cr. - Decatur, TN	303	(117)	VR	0	46	132	(52)
White Cr. - Sharps Chapel, TN	6.94	(2.68)	VR	0	100	119	(47)
Parker Br. - Leicester, NC	3.91	(1.51)	BR	0	15	97	(38)
Chestuee Cr. - Englewood, TN	38.3	(14.8)	VR	0	18	132	(52)
Boneyard Cr. - Urbana, IL	9.27	(3.58)	CL	44	10	91	(36)
Browns Cr. - Nashville, TN	30.6	(11.8)	NB	33	24	117	(46)
Fourth Cr. - Knoxville, TN	2.12	(0.82)	VR	45	10	117	(46)
Third Cr. - Knoxville, TN	4.14	(1.60)	VR	28	26	117	(46)
First Cr. - Knoxville, TN	1.29	(0.50)	VR	16	45	117	(46)
Plantation Hills - Knoxville, TN	0.62	(0.24)	VR	23	37	117	(46)

*BR = Blue Ridge
VR = Valley and Ridge
MA = Mississippi Alluvium
HR = Highland Rim
CL = Central Lowlands
NB = Nashville Basin

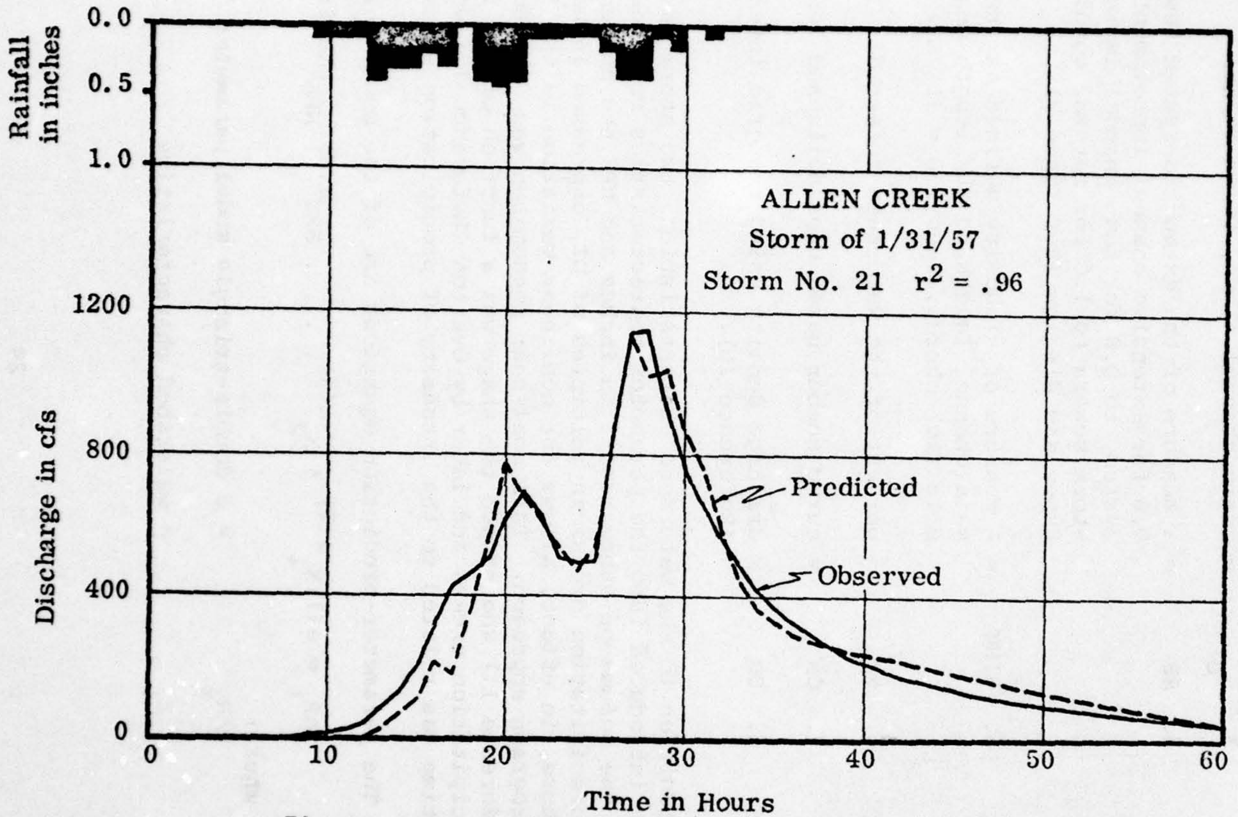

Figure 8. Example of Average Adjustment of Model (Reference 4)

to the following watershed characteristics:

1. AREA = drainage area, mi^2 (km^2)

2. SHAPE = dimensionless measure of shape defined as the squared length of the main channel divided by the area = l_c^2/AREA

3. S_c = main channel slope, ft/mi (m/km)

4. SS = a measure of the extend to system sewered from 0.6 for extensive channel improvement and sewer system, to 0.8 for some channel improvement and storm sewers to 1.0 for natural conditions. After Espey and Winslow (Reference 9).

5. SINU = A measure of sinuosity defined as the ratio of the main channel length to the length measured by one-mile (km) chords, less 1.0 = $(L_c/L_1) - 1.0$

6. PF = percent of the watershed forested

7. CN = curve number based upon soils and land use

8. DD = drainage density using the grid intersection method (Reference 10).

In addition to the watershed characteristics, two storm variables have been introduced into the parameter characteristics relationships, SRO the volume of storm runoff in inches and NPE the number of periods of precipitation excess in multiples of DT, expressed in hours. This feature, in effect, allows for nonlinear variation in the linear unit-hydrograph approach. This nonlinear phenomenon was documented by Minshall (Reference 11) who showed the shape was a function of the intensity of precipitation excess and later by Overton (Reference 12) who showed that lagtime was related to the intensity of precipitation excess.

The parameter-prediction equations are of the power form:

$$PAR_i = a(1)X_1^{a(2)} * X_2^{a(3)} \cdots SRO^{a(n)}\ NPE^{a(n+1)} \qquad (9)$$

where:

PAR_i = a double-triangle model parameter

X_i = watershed characteristics

a(1)-a(n+1) = coefficients

Stepwise linear regression was used to identify significant variables for the prediction of each parameter. Therefore, not all variables were used in predicting each parameter. Table 2 shows the final parameter prediction equations based upon the analysis of 354 storms. Also shown are the multiple correlation coefficients obtained using log transformed data.

A linear equation was used for the prediction of the constant loss parameter PHI. Two additional measures were used to describe the precipitation including: LOSS which is the storm rainfall minus the runoff volume, and RFINT which is the rainfall intensity expressed as storm rainfall divided by the number of rainfall intervals in multiples of DT expressed in hours. In addition, another variable was found to be necessary to explain the high PHI values encountered at watersheds where high water losses to the carbonate rock system occur. The variable used is the value obtained for the transmission loss parameter when the continuous streamflow model is adjusted to the watershed streamflow data. Although this approach necessitates use of the two models at those watersheds where transmission losses occur, this phenomenon is not easily quantified in any other manner. Fortunately transmission losses can usually be ignored since they occur only under urban conditions and then only where the overburden and carbonate rock system is very permeable. The equation for PHI is:

$$\begin{aligned} PHI = & -1.285 + 1.259 * SS + 0.79 * IMP + 0.37 * TLP \\ & + 0.061 * LOSS + 0.186 * RFINT - 0.021 * RF \quad (r = 0.76) \end{aligned} \tag{10}$$

and $PHI \geq .0$

where:

SS = measure of sewer system (previously defined)

IMP = impervious fraction of watershed

TLP = transmission loss parameter, in (cm)

LOSS = rainfall minus runoff, in (cm)

RFINT = average storm rainfall intensity in/hr (cm/hr)

RF = storm rainfall, in (cm)

TABLE 2. TVA STORM HYDROGRAPH MODEL PARAMETER PREDICTION EQUATION COEFFICIENTS

Model Parameter	Constant	Independent Variable Exponent										
		AREA	SHAPE	Sc	DD	SINU	PF	CN	SS	SRO	NPE	R^2
UP	0.000434	-0.207	0.128	0.210	0.426	0.0	-0.289	1.455	-1.406	0.0765	-0.303	0.25
T1	10.09	0.228	0.0	-0.204	-0.688	0.215	0.175	0.0	2.535	0.0	0.276	0.34
T2	794.2	0.309	-0.206	-0.232	0.0	0.0	0.175	-1.272	1.511	0.0	0.310	0.84
T3	21,608.0	0.313	-0.438	-0.154	0.0	-0.133	0.136	-1.752	1.750	0.0973	0.234	0.74

2. SCREENING WATER QUALITY VARIABLES

a. Principal Components Analysis

A very powerful technique has been utilized for coping with the problems, in both linear and non-linear least squares, associated with statistical relations amongst the independent variables. The techniques, principal component analysis, transforms the independent variates, for both linear and non-linear models, into new variates called "components" which are linear sums of the original variates. A search technique is employed to locate the components such that they are not statistically correlated. This provides us with new variates to correlate with stormwater Q which are truly independent, i.e., statistically unrelated. When the correlation with Q is completed, the components can be transformed back to the original variates.

Let us work with the linear model

$$\hat{Q} = c_1 x_1 + \ldots + c_p x_p \qquad (11)$$

and transform it by components analysis to

$$\hat{Q} - \beta 1 \zeta^1 + \ldots + \beta_p \zeta_p \qquad (12a)$$

where ζ_i is a component or eigenvector. The components are new variates which are linear functions of x_i (the original variates) and they are statistically independent. Statistical independence is defined as having a co-variance (cov) equal to zero.

$$\text{cov}\{\zeta\alpha\zeta_\beta\} = \sum_{j=1}^{n} \zeta_{\alpha j}\zeta_\beta = 0 \qquad (12b)$$

Our problem is made simpler by removing the scale effects of the original variates. Hence, the normalized original variates are defined as

$$\psi_i = (x_i - \bar{x})/s_i \qquad (13)$$

letting the mean be zero and the standard deviation s be unity. The first two moments of the normalized variates are

$$\sum_{j=1}^{n} \psi_{ij} = 0 \text{ (mean)} \qquad (14)$$

and

$$\frac{1}{n}\sum_{j=1}^{n} \psi_{ij}^2 = 1 \text{ (standard deviation)} \qquad (15)$$

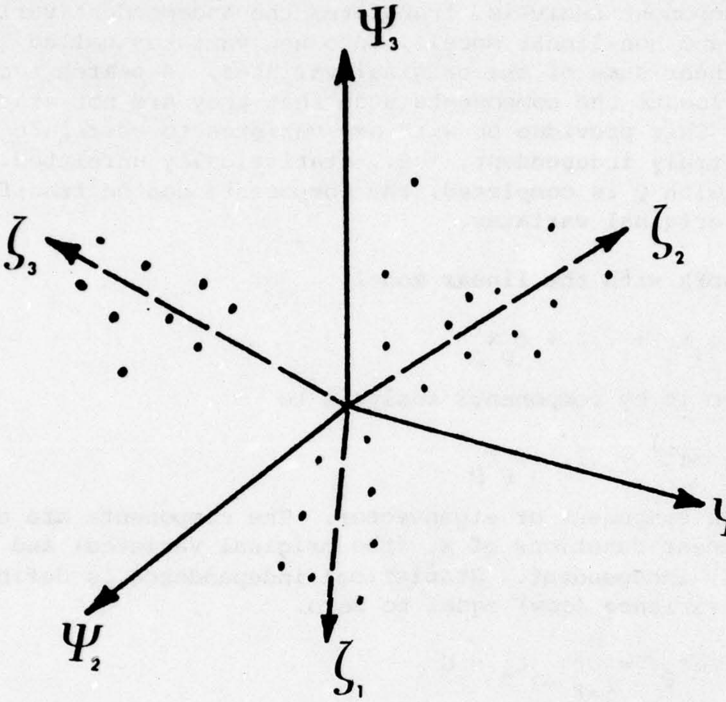

Figure 9. Location of Components in Three Dimensions

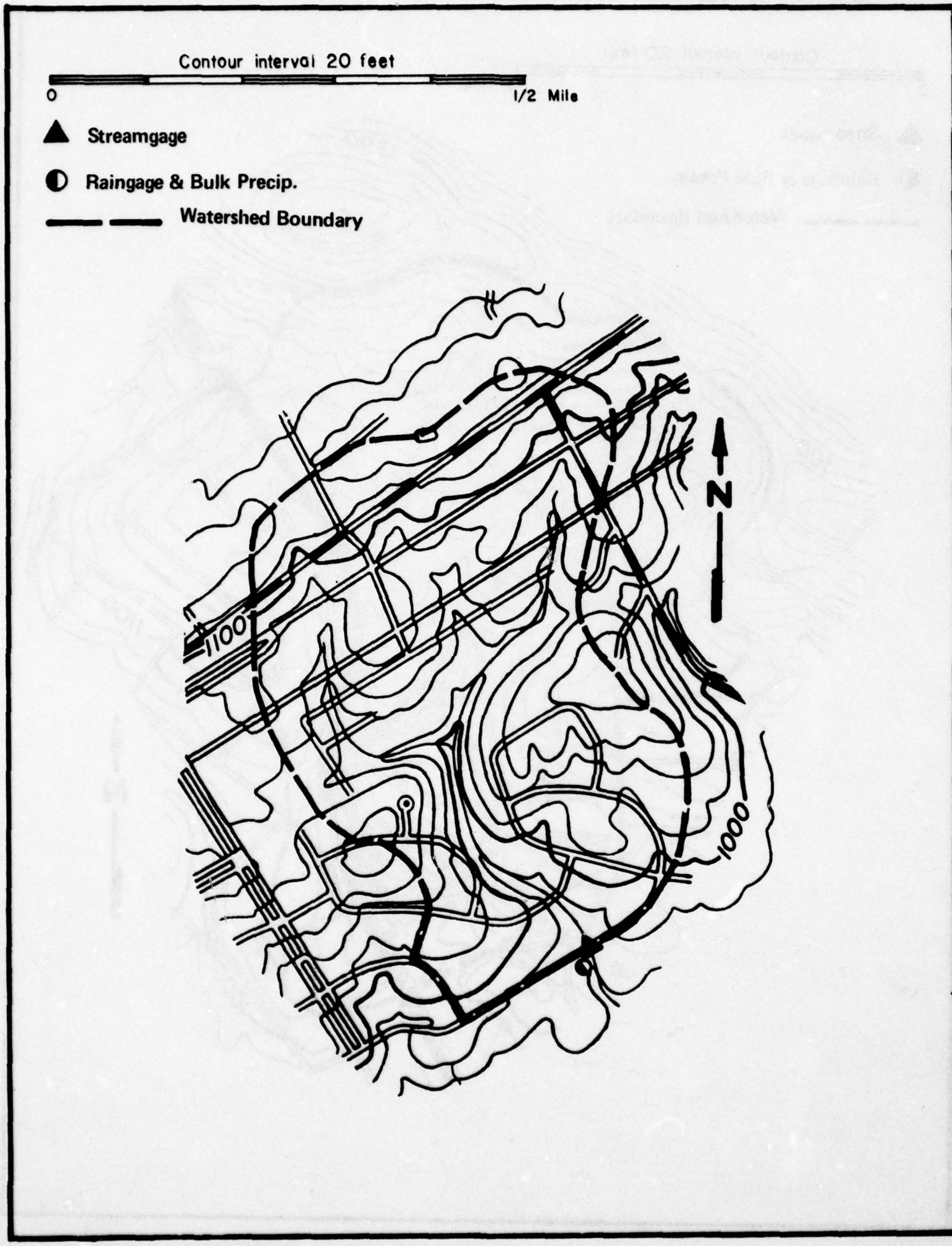

Figure 10. Map of Plantation Hills Watershed

(Reference 1)

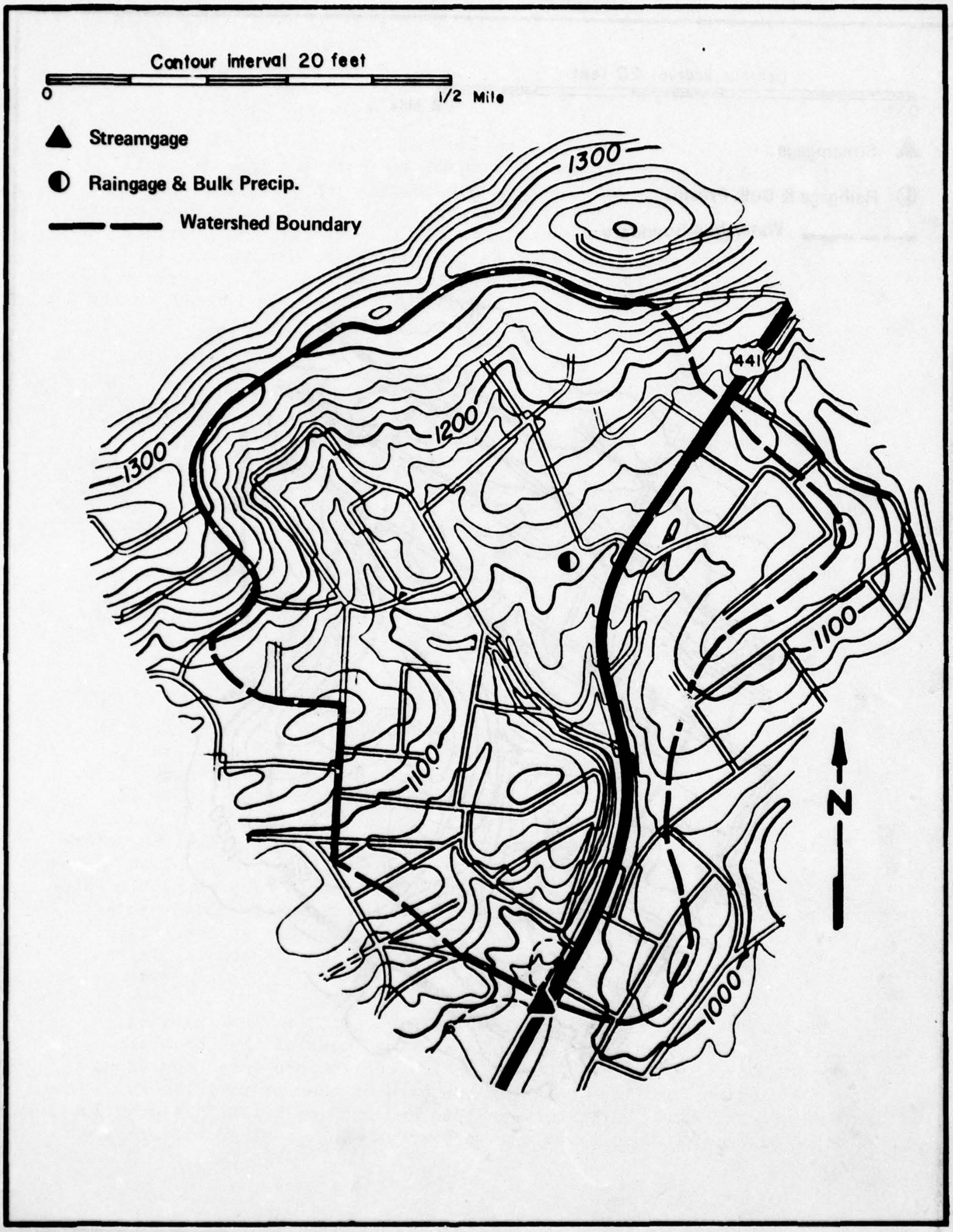

Figure 11. Map of First Creek Watershed

(Reference 1)

The solution to our problem begins by plotting all p of the original variates in p-dimensional space and rotating the axes until the orthogonal system of components are found. An attempt to demonstrate this for three dimensions is shown in Figure 9. The data points are plotted as referenced to the axes of the three original variates and then the axes are rotated until the components are orthogonal, i.e., statistically independent. Statistically, this feat is achieved by minimizing the variance or spread around the components subject to the constraint that orthogonality must be achieved.

b. Data Base

Four watersheds within the city of Knoxville, Tennessee, having different land use and geologic characteristics, but possessing relative stability of land use patterns and lack of major point sources of pollution, were originally chosen for study by the Tennessee Valley Authority. Rainfall, stormwater runoff and quality, and atmospheric fallout data were collected in these watersheds from 1972 to 1975.

The watersheds are characterized by carbonate rock formations overlain by permeable soils of varying depth. A few older homes in the First Creek watershed have septic tanks, but all other residences and establishments in the study area are served by sanitary sewers. Streets are cleaned in each of the watershed areas at least once per month by flushing followed by manual sweeping into a "vac-all" unit.

The Plantation Hills watershed, shown in Figure 10, is located in a subdivision of the same name in East Knoxville, which was developed between 1950 and 1965. It is located near an interstate highway, I-75. Lot sizes are 1/2 to 3/4 acre, and the population density (1970) was 3.5 per acre. The entire watershed is underlain by dolomite limestone, which is quite soluble and is associated with a karst terrain characterized by sinkholes, blind drainage, caves, and springs. Much of the runoff generated in this watershed disappears into a very permeable section of the stream channel upstream from the stormwater runoff gauge. Often the only source of stormwater runoff is from the street located immediately above the stream gauge. Stormwater runoff usually lasts only a short time in this watershed.

The First Creek watershed, shown in Figure 11, is in a primarily residential neighborhood of North Knoxville. Most of the homes are about 50 years old and are located on 1/4 to 1/2 acre lots, but along the ridge which forms the northern boundary of the watershed there are some newer homes built between 1950 and 1965 on 1/2 to 3/4 acre lots. A major urban artery, Highway 441, traverses the watershed. There are

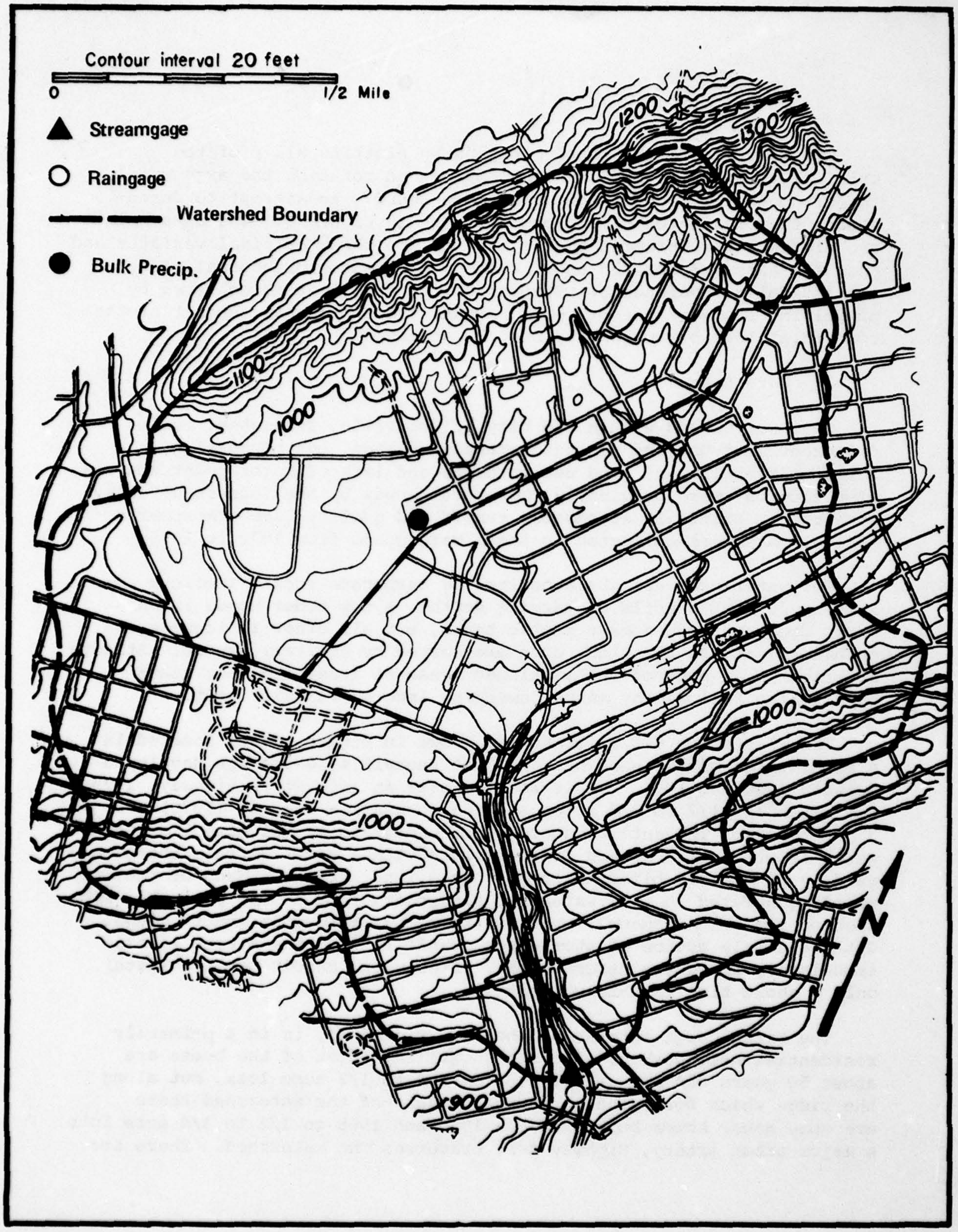

Figure 12 Map of Third Creek Watershed
(Reference 1)

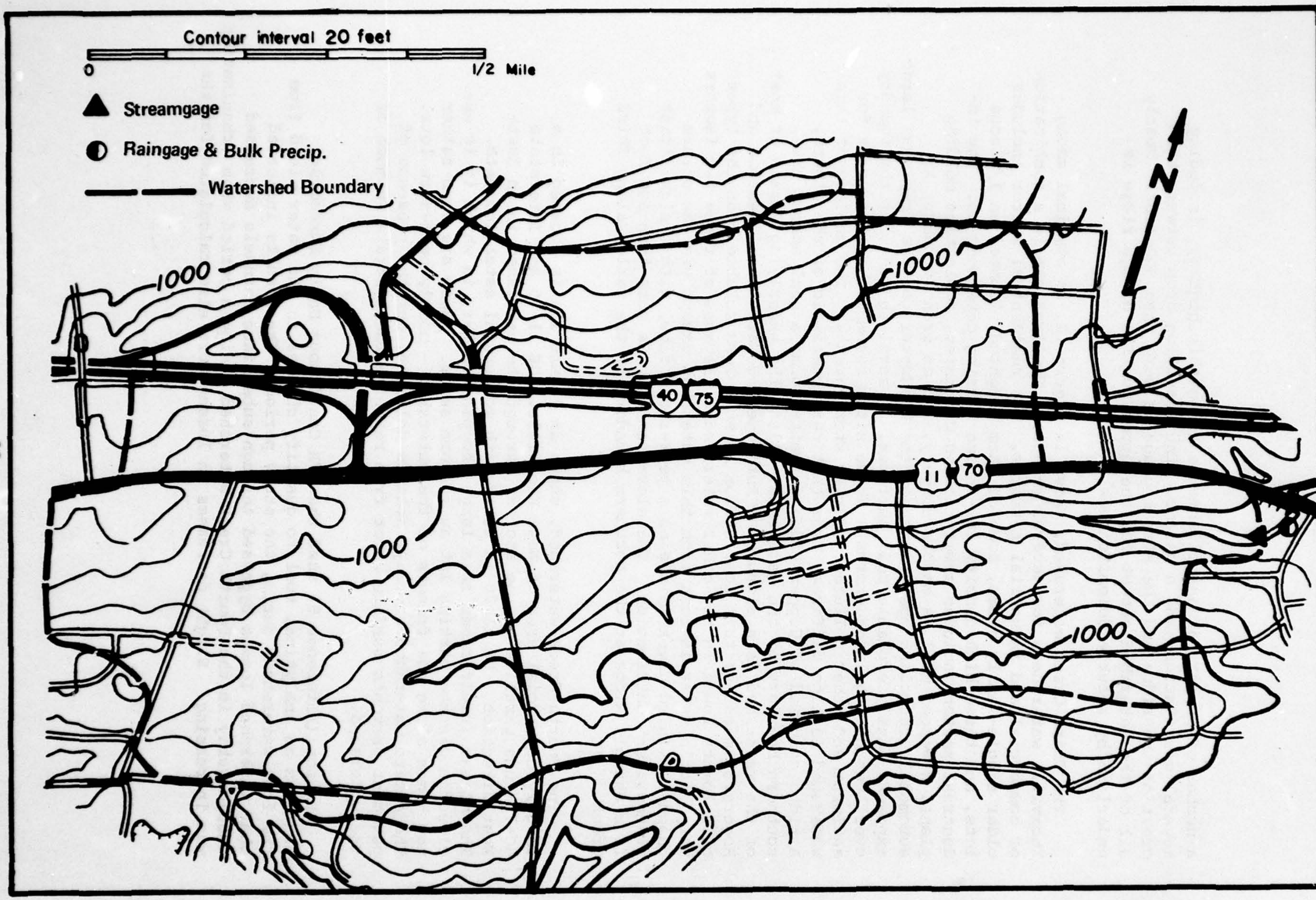

Figure 13. Map of Fourth Creek Watershed (Reference 1)

a number of commercial establishments along this highway, including several gas stations and a garage. The elevation drop between the crest of the ridge and the stream gauge is more than 300 feet. Nearly all of the watershed except for the upper portion of the ridge is underlain by soluble dolomite rock.

The Third Creek watershed, shown in Figure 12, is atypical among Knoxville watersheds for several reasons. The land use is a combination of commercial and industrial patterns. The residential sector includes older single-family homes, some of them blighted, located on 1/4 acre lots, and three high-density low-income housing developments. The industrial sector includes some trucking companies, a plastics molding plant, a veneer mill, a marble company, and an iron foundry. An automobile recycling operation near the headwaters of the creek has large exposed, easily erodable soil surfaces. A settling pond for the foundry overflows during large storms. These sites resemble point sources for sediment and other pollutants during storm events. The geology of this watershed is also different from that characteristic of the others. A ridge composed of Copper Ridge dolomite forms a seal across the southern boundary of the watershed. Also, the underlying strata of most of the watershed is a mixture of Rutledge and Maryville limestone and Rogersville shale; this combination is much less soluble than the types of dolomite found in the other watersheds. Because of these two factors very little water is lost from this watershed except via the stream channel. Third Creek maintains a year-round flow, with a significant groundwater contribution. The elevation drop is more than 300 feet from Sharp's Ridge at the northern boundary to the valley along Third Creek.

The Fourth Creek watershed, shown in Figure 13, is located in a largely commercial area of West Knoxville. US 11-70 and Interstate 40-75 runs parallel to one another through the watershed in an East-West direction. US 11-70 is lined with commercial establishments. The stream itself rises in a large shopping center, in which it is enclosed under the parking lot as a storm sewer. There are some rather new homes along the fringes of the watershed - mostly on 1-acre lots. The entire watershed overlays soluble dolomite, but the presence of pavement prevents surface water from reaching these strata in much of the watershed.

Betson (Reference 8) used the TVA Continuous Daily Streamflow Model as an analytical tool to quantify changes in the water yield from the four watersheds during the study period. The results indicated that watershed losses bypassed through subsurface channels decreased considerably in the Fourth Creek watershed during a period when urbanization was increasing. Slight decreases in losses were also calculated for the

Plantation Hills and First Creek watersheds. Average losses during the study period were 50, 81, and 89 percent of the expected runoff for Fourth Creek, First Creek, and Plantation Hills, respectively. Losses were negligible in the Third Creek watershed.

Recording streamflow gauges were installed at the lower boundary of each watershed during 1971. In the First Creek watershed a rain gauge and an atmospheric fallout sampler were placed more than 1/2 mile from the stream gauge, but in the other watersheds these instruments were located near the stream gauge.

The atmospheric fallout samplers have a height of 8.1 inches and an effective top diameter of 7.7 inches. They are equipped with a bird ring. The sampling period for atmospheric fallout was generally about one month.

Two water quality sampling devices were used. One was the ISCO automatic water sample collector, model 780, (manufactured by the Instrument Specialties Company, Lincoln, Nebraska).[1] The sampler was modified so that it would be activated whenever the stage of flow reached a certain height. The sampler was mounted in a trailer and was moved from First Creek to Fourth Creek. Sampling was done from May, 1972, to October, 1973, on Fourth Creek, and from November, 1973, to February, 1975 on First Creek.

The other sampler was a PS69 water sampler designed at the Federal Inter-Agency Sedimentation Project, St. Anthony Falls Hydraulic Laboratory, Minneapolis, Minnesota. It was planned that this unit would be moved from Third Creek to Plantation Hills, but the unit was so large that there was no room for it at the Plantation Hills site. Therefore, it remained at Third Creek; it was used from May, 1972 to December 1974. This sampler utilizes a variable sampling interval adjusted to the stream gauge rating curve, so that the sampling interval is proportional to the discharge.

Sampling at Plantation Hills took place between August 1974 and March 1975. All sampling was done manually.

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[1] Trade names and company names are used in this publication solely to provide specific information. Mention of a trade name does not imply any endorsement of the product listed by the US Air Force.

At First, Third, and Fourth Creeks the stream gauge charts were usually marked at the times when sampling was done. At Plantation Hills this was not done. This was a serious problem; the only knowledge available concerning time of sampling was the date, which was indicated in the water quality data. It was necessary to assume that the sampling for the date indicated took place during hydrograph peaks. There is no assurrance that this assumption is valid, because in some of the other watersheds, samples were taken when there was no storm event. These samples were discarded for study purposes. As much as possible, events were limited to single storms during one day, but this guideline could not be entirely followed because there was not enough useable data. No reliability can be claimed for the results from Plantation Hills. The study of this watershed was useful for developing and refining computational procedures and the results are useful for comparison with those from the other watersheds.

This study concentrated primarily upon the Plantation Hills, First Creek, and Fourth Creek watersheds. The collection of dustfall and water quality data in these watersheds often was not synchronized. In addition, some of the sampling units malfunctioned at times, resulting in incomplete observations. A larger data base would have been very beneficial, because it would have allowed more screening of events according to guidelines reported by Barkdoll (Reference 13). In each watershed, only eight or nine storm events were found to have all of the necessary data available. It must be mentioned, however, that the requirements for this study were not foreseen by the researchers who organized the data collection project.

From the beginning of the project in 1972 until May, 1973, all water quality samples were shipped by bus to the TVA Water Quality Laboratory in Chattanooga, Tennessee, shortly after they were taken. The analytical procedures used conformed to either the Environmental Protection Agency Methods (EPA, 1971) or to Standard Methods (APHA, 1971).

Due to manpower limitations, the TVA laboratory was unable to analyze the samples after mid-1973. For the remainder of the project the samples were analyzed at the Mark C. Whitaker Water Treatment Plant operated by the Knoxville Utilities Board. Sample analysis conformed to Standard Methods (APHA, 1971) except for pH (potentiometric method); ammonia (specific iron); and silica and calcium (atomic adsorption method). Four constituents (organic-N, Hg, As, and COD) were not analyzed during this latter phase of the project.

Barkdoll (Reference 13) noticed that for many constituents the reported concentration values for Third Creek did not appear to be

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consistent between the two phases of analysis. He performed a split-data test on those constituents which were analyzed by both laboratories; he found that the data from the two phases represented statistically different populations at a significance level of at least 0.05 for the constituents Fe, K, NH_3, TDS, pH, Color, and SS. For this reason he dropped these constituents from his matrix for principal components analysis screening. In this study, the only significant overlap between the two laboratories' work occurred in the Fourth Creek data. This data did not present any unusual difficulties in our experience.

c. Results

For Plantation Hills watershed, 25 water quality variates were reduced to seven; seven components represented by these seven variates explained 81.9 percent of the total variance of the original matrix. These variates were SiO_2, PO_{4ORG}, CA, NH_3-N, Pb, Color, and K.

For First Creek, 25 original variates were reduced to eight; these were Fe, Ca, Mg, TDS, $CaCO_3$, Mn, F, and Pb. Eight components explained 81 percent of the original variance.

For Fourth Creek, twenty original variates were reduced to eight; these were pH, PO_{4ORG}, NH_3-N, ORG-N, NO_2-NO_3, TDS, Hg, and As. Eight components explained 84.6 percent of the variance.

An admitted weakness in this screening effort is that several variates were almost equally weighted across the first component for each watershed. This was also Barkdoll's experience in attempting to screen the Third Creek Data (Reference 13). In this study that constituent having a large data base, a relatively large weighting, and a relatively high correlation with average discharge was chosen to represent the first component. The choice was usually obvious in components of lower rank.

Parametric linear regression models were developed for discharge as a function of the screened water quality variates from each watershed.

The model for Plantation Hills is

$$\text{DISCHARGE} = 0.1254 * \text{SIO}_2 + 0.2591 * \text{PO}_{4ORG} + 2.147 * \text{NH}_3\text{-N} + 1.733 * \text{PB} + 0.0037 * \text{COLOR} + 0.0754 * \text{K} \quad (16)$$

The coefficient of determination is 0.3798.

For First Creek the model is

$$DISCHARGE = 0.0060 * FE + 2.798 * CA + 4.296 * MG + 0.9377 * TDS + 0.0068 * CACO_3 - 0.0297 * MN + 8.591 * F + 0.1024 * PB \quad (17)$$

The coefficient of determination is 0.0286.

The model developed for Fourth Creek is

$$DISCHARGE = 0.4384 * pH + 12.24 * PO_{4ORG} - 11.37 * NH_3\text{-}N - 237.2 * ORG\text{-}N + 6.216 * NO_2\text{-}NO_3 + 0.3683 * TDS - 0.0447 * HG + 5.638 * AS. \quad (18)$$

The coefficient of determination is 0.5343.

The fact that some of the same constituents are retained after screening in more than one watershed indicates that principle components analysis is a promising tool for reducing the complexity of the problem of stormwater quality analysis in this type of study. Some of the difficulties encountered in screening the data by principle components analysis could have been caused by discrepancies and gaps in the data itself.

The coefficients of determination suggest that the regression model for Fourth Creek and possibly the Plantation Hills model as well could be used for some purposes with a limited degree of confidence.

The deterministic model generally produced low coefficients of determination for predicting constituent concentrations. This could be due to the fact that the model structure does not reflect the nonlinear nature of surface water transport mechanisms or the first-flush effect in contaminant removal. Results for some constituents are sufficiently impressive, however, to indicate that this model could be used in some cases in which rough estimates are needed. However, the relatively low correlations of pollutant concentration with discharge indicates that averaging over storms damps out significant variations and that pollutographs are needed to accurately document these variations.

3. RELATION BETWEEN DUSTFALL AND STORMWATER QUALITY ACCUMULATION

a. Description of Model

Barkdoll (Reference 13) developed a deterministic model for use in analyzing the relationship between dustfall, stormwater runoff, and water quality. His equation is,

$$C = \frac{D * N}{Vol} \tag{19}$$

where

C	= average constituent concentration during a storm event,
D	= dustfall rate (weight/unit time),
N	= antecedent dry period, and
Vol	= runoff volume.

The model is based on the following assumptions:

(1) Removal of contaminants by runoff is 100 percent efficient

(2) There is no carry-over from storm to storm; i.e., each storm is an isolated event.

(3) All dustfall becomes a water contaminant.

(4) Dustfall is the only source of water quality impairment.

(5) Dustfall jar measurements are true values for dustfall.

Admittedly the model is "---very simplistic and has severe limitations." Barkdoll thought that it would be helpful in assessing the magnitude of the effects of dustfall on qater quality. Predicted versus measurered stream concentrations were compared, and "K" values (predicted divided by observed concentration) and coefficients of determination were reported for various constituents in the Third Creek watershed (See Table 3).

b. Results

The values of K for the urban basins are shown in Table 4, (Reference 14). For most contaminants the model indicated that dustfall contributions were more than adequate to account for stream concentrations. These findings should be used with caution, chiefly because they are based upon a single dustfall jar.

TABLE 3. RESULTS OF DUSTFALL MODEL

Watershed		Plantation Hills		First Creek		Third Creek		Fourth Creek	
Group	Constituent	K	Coeff. of Det.	K	Coeff. of Det.	K	Coeff. of Det.	K	Coeff. of Det.
Limestone Associated Minerals	CA	10.33	0.0016	1.644	0.0560	---	------	1.437	0.0145
	MG	5.90	0.1305	2.839	0.0034	0.39	0.0729	1.979	0.0002
	$CACO_3$	6.43	0.2401	6.362	0.1439	0.75	0.0961	2.154	0.4627
	ALK	9.18	0.0262	1.045	0.0296	---	------	-----	------
Other Minerals	SIO_2	3.18	0.0089	1.532	0.0249	1.73	0.0100	1.039	0.0874
	NA	15.00	0.0418	12.40	0.0128	0.52	0.0256	2.267	0.2842
	K	14.4	0.1104	10.67	0.0802	1.52	0.2601	5.535	0.1604
	SO_4	40.1	0.0020	1.902	0.4661	2.23	0.0016	3.558	0.0008
	CL	20.88	0.0097	21.91	0.0383	4.63	0.1024	7.254	0.1525
	F	290.4	0.0250	14.43	0.0529	4.01	0.0196	5.230	0.7149
Solids	SS	----	-----	2.356	0.0358	0.27	0.5776	0.5291	0.0885
	TDS	1.375	0.4173	2.243	0.1373	2.19	0.3271	6.050	0.0102

TABLE 3. RESULTS OF DUSTFALL MODEL (CONCLUDED)

Watershed		Plantation Hills		First Creek		Third Creek		Fourth Creek	
Group	Constituent	K	Coeff. of Det.	K	Coeff. of Det.	K	Coeff. of Det.	K	Coeff. of Det.
	FE	9.20	0.1671	2.106	0.0282	0.12	0.3249	0.1870	0.2495
	MN	3.48	0.1310	9.521	0.0218	0.27	0.0729	0.4506	0.2028
Metals	PB	----	-----	----	-----	0.83	0.2809	7.844	0.3173
	HG	----	----	----	-----	5.84	0.0169	6.119	0.2638
	AS	----	-----	----	-----	1.14	0.2304	1.144	0.0141
	ORGN	----	-----	----	-----	----	-----	1.154	0.0620
Nutrients	NH_3N	42.0	0.0192	24.49	0.0709	152.11	0.2304	243.3	0.0274
	$NO_2.3$-N	9.52	0.9968	8.528	0.2944	18.21	0.0001	30.80	0.1028
	PO_{4T}	2.10	0.0074	1.898	0.0[illegible]08	-----	------	0.6951	0.0025
	PO_{4ORG}	1.13	0.0813	1.616	0.3058	----	------	5.998	0.7932
	COD	----	-----	----	------	2.88	0.1521	1.659	0.0038

TABLE 4. ADJUSTED RESULTS FOR SELECTED CONSTITUENTS
FIRST CREEK

CONSTITUENT	$CACO_3$	K	TDS	FE	PO_{4T}
K Value	1.20	2.07	.418	.400	.307

The constituents have been divided into groups so that behavioral trends for various classes of pollutants might be identified, if they exist.

A large proportion of the potential runoff generated in the Plantation Hills, First Creek, and Fourth Creek watersheds is lost into groundwater channels; therefore it was realized that the model does not represent the system if only that fraction of the runoff which reaches the stream is considered to be a dilution factor for the accumulated dustfall. In the First Creek watershed Betson (Reference 8) calculated that 81 percent of the potential runoff is bypassed; therefore the runoff measured at the stream gauge is only 19 percent of the potential runoff volume. This total volume of dilution water was calculated, and average concentrations were recalculated for certain constituents (See Table 4). First Creek was used for this purpose because this watershed has a high percentage of losses and the data is reasonably reliable. One constituent from each group which had a relatively large number of measurements and a relatively good coefficient of determination in the previous trial was used. The model assumes that the quality of the water transported from the watershed by whatever mechanism, whether by runoff into the stream or by infiltration into the ground, is uniform over the entire watershed. This is admittedly a simplistic assumption; however, it was hoped that this approach could shed light on the problem of whether the large overpredictions resulting from this model are due to the dustfall rates or to a lack of accounting for all of the dilution water.

Betson (Reference 8) performed a mass balance on the Knoxville watersheds, calculating input from the atmospheric fallout samples and output from the stormwater quality samples and streamflow measurements. He emphasizes the fact that measured atmospheric fallout values are highly dependent on the location of the sampler, and suggests that average values of atmospheric input across the four watersheds should be examined. Atmospheric sources were found to approximately account for loadings of silica, fluoride, lead, mercury, arsenic, and COD. However, they could not account for all of the iron, calcium, magnesium, potassium, total dissolved solids, hardness, manganese, alkalinity, or suspended solids in any of the watersheds. These results are consistent with those obtained in the present study.

Barkdoll (Reference 13) and Betson (Reference 8) have theorized that a weather modification effect may be occurring in the Knoxville area. Betson points to the fact that a number of improbable floods occurred during the three-year study period. He attributes this effect to the production of aerosols at two large coal-burning power plants, one located 35 miles and the other 14 miles west of Knoxville. Surface heating in the city could produce convection currents which could transport

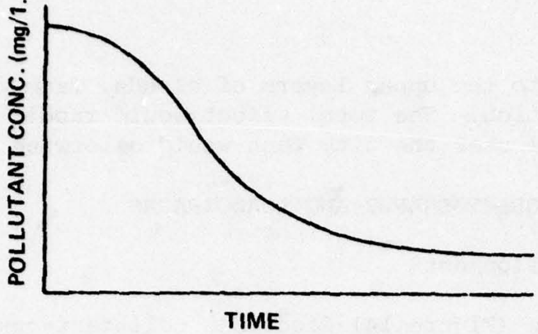

Figure 14. A Pollutograph

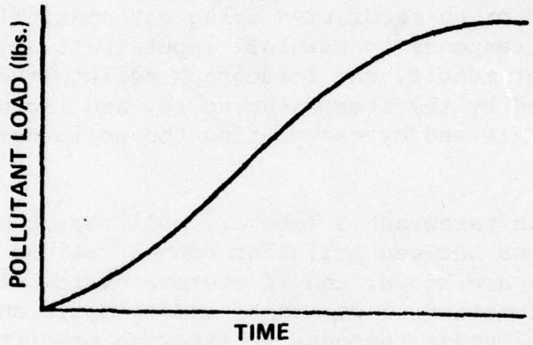

Figure 15. A Loadograph

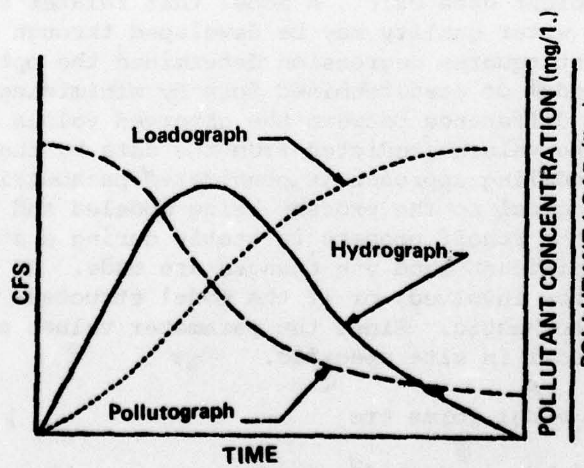

Figure 16. A Quantity/Quality Hydrograph Illustrating the Corresponding Hydrograph, Pollutograph, and Loadograph (Reference 3)

these aerosols into the upper layers of clouds, from which they would fall as precipitation. The total effect would result in a larger amount of rainfall near the city than would otherwise occur.

4. ANALYSIS OF POLLUTOGRAPHS AND LOADOGRAPHS

a. Model Development

A pollutograph (Figure 14) describes pollutant concentration as a function of time, and a loadograph (Figure 15) describes the cumulative pollutant load as a function of time. The correspondence of the pollutograph and loadograph to the storm runoff hydrograph is shown in Figure 16. The hydrograph and pollutograph are established through actual measurement or by prediction using mathematical models that relate the system response to rainfall inputs. At any point in time following incipient runoff, the loadograph depicts the total pollutant load delivered by the stormwater up to, and including, that time. The loadograph is derived by convoluting the pollutograph with the hydrograph.

As mentioned in paragraph 3 (above), pollutographs are needed to assess the relations between pollutant concentrations with discharge. If those relations are known, and if stormwater response is reliably predictable as a function of watershed and climatic characteristics, it follows that pollutant response is likewise predictable. However, the added variable which affects pollutant response is activity, e.g., lead of automobile traffic. Therefore, any attempt at regionalization of stormwater quality must take this factor inso consideration.

When sufficient data exist, a model that relates storm characteristics to runoff water quality may be developed through regression analysis. Least squares regression determined the optimal parameter values for a model of predetermined form by minimizing the sum of the squares of the difference between the observed values of the dependent variable and the values predicted from the data by the regression model. This modeling approach is considered parametric if the model structure is logical to the process being modeled and the process is stable; e.g., the runoff process is stable during a study period as long as no significant land use changes are made. If there is a strong element of chance involved, or if the model structure is "black box", the model is stochastic. Since the parameter values are data dependent a regression model is site specific.

To popular model forms are

$$Y = a + bX_1 + cX_2 + dX_3 + eX_4 \tag{20}$$

$$Y = aX_1{}^bX_2{}^cX_3{}^dX_4{}^e \qquad (21)$$

where Y is the dependent variable, X_1, X_2, X_3, X_4 are independent variables, and a, b, c, d, e are parameters to be determined based on the data by a least squares analysis.

These equations were written in terms of four independent variables only for illustrative purposes. The number of independent variables included in a model should be enough to adequately describe the process, but not so many as to render the equation meaningless and untenable. Equation (21) is a linear model, whose parameters are determined by linear least squares. The second equation is a nonlinear model. The parameter values for this equation are determined by nonlinear least squares or by linear least squares following the transformation

$$\log Y = \log a + b \log X_1 + c \log X_2 + d \log X_3 + e \log X_4 \qquad (22a)$$

which expresses the nonlinear equation in linear form.

An application of least squares regression to stormwater quality modeling was demonstrated by Colston in his study of urban storm runoff at Durham, North Carolina (Reference 14). As part of the study, regression equations were developed that described within-storm variations for 19 quality variables in terms of storm characteristics. A nonlinear model of the form of Equation (22a) was used. The independent variables were rate of runoff (CFS), time from the start of a storm (TFSS) in hours, time from last storm (TFLS) in hours, and time from last peak (TFLP) in hours. A stepwise regression using the data from 36 storm events found that the rate of discharge (CFS) and the time from the storm start (TFSS) were the two most significant variables. Only a modest gain in the coefficient of determination r^2 was observed when including the other two time variables. For this reason, Colston decided to limit regression equations to CFS and TFSS for regression simplicity. The equations were determined for the 19 quality variables. These equations are specific to the Third Fork Creek in Durham, North Carolina.

It is significant that the final model form relates the stormwater pollutant load to a product of the rate of discharge and the time sine the start of the storm, and did not include the other two time variables. This agrees with the statement in conjunction with the derivation of the pollutant removal model in the next section that the "removal of pollutants is a direct function of the total volume of runoff." The time since the last storm was not important for two reasons: (1) the frequency of storm events, and (2) the fact that a major portion of the pollutants present at the outset of a storm have accumulated on the basin in the first one or two days following the last storm. Since the time between most storms exceeds one or two days, an equivalent amount of pollutants will have accumulated prior to most runoff events. The time from the last peak was not significant since, again, it is the volume of runoff that is important in the removal of pollutants by stormwater.

Utilizing these concepts, it can be shown how pollutant response can be analytically associated with stormwater hydrograph response.

Colston's relation between pollutant concentration and the storm hydrograph is

$$C + a(CFS)^b(TFSS)^c \tag{22b}$$

For convenience, let Q = CFS and t = TFSS. Then the total load, L, for a storm can be found by integration.

$$L = \int_T C dt = \int_T aQ^b t^c dt \tag{23}$$

Hence, the load associated with the instantaneous unit hydrograph or response function, U, is:

$$C = a\, U^b t^c \tag{24}$$

Then the total load associated with the response function, or the unit load, is:

$$L[\upsilon] = \int_0^\infty a\, U^b t^c dt \tag{25}$$

Therefore, once the response function is reliably predictable via regionalization, likewise the corresponding unit load will be predictable in terms of the same watershed and climatic characteristics. This scheme will be explained in further detail in paragraph 8.

b. Data Base

Measurement of pollutographs is by far one of the greatest challenges in hydrology. Because of the flashiness of the urban basins and the occurrence of significant storms at extremely inopportune hours, there was not enough measurements of instantaneous concentrations to form a statistically representative sample. However, approximately 50 instantaneous measurements were made on three of the small watersheds in the New River basin--two have been strip mined for coal and one is undisturbed. These measurements were taken during three separate storms.

Although 50 measurements were taken, the degrees of freedom would be significantly less than 50 because there is a built-in serial correlation since there were only three storms included in the analysis. Hence, no reliable conclusions as to the effects of coal strip mining on pollutant response may be drawn. The results do, however, reflect the visual observation that considerably higher concentrations of heavy metals and suspended solids are being generated on the stripped watersheds.

c. Results

The extent of pollutant response as related to stormwater response is explained thus far in terms of total loads and total storm runoff volume. But a model has been developed which can be calibrated with data collected at a specific Air Force Base. This will be further explained.

5. RELATION BETWEEN STORMWATER RUNOFF AND QUALITY-REMOVAL

a. Description of Model

Transportation of contaminants from a land surface area results from resuspension by wind, sanitary practices such as street cleaning and urban runoff. Contaminants may be absorbed onto soil particles; these may remain in the area or they may be removed by one of the vectors mentioned above.

Some knowledge of the potentially polluting aspects of construction activity is also available. Sediment resulting from soil disturbance and exposure is the chief pollutant associated with construction; however, other construction activities can also release a host of wasted or spilled chemicals and building materials. These pollutants can be reduced by site planning for the control of runoff volume and velocity, and by various strategies for entrapping the sediment itself.

A high percentage of impervious, paved areas is usually associated with an urban environment. Paved areas and other impervious surfaces, such as buildings prevent the infiltration of stormwater into the soil. Moreover, paved areas are usually smooth and offer less resistance to flow than do vegetated areas. The combination of these two effects increases both the volume and the velocity of the stormwater runoff. The result is the "flashy" hydrography usually associated with urban watersheds, characterized by a short time to-peak and a high peak flow relative to base flow. This type of watershed response can result in erosion of the stream banks and flooding and sediment deposition downstream, The effect of land use on pollution loadings is better understood relative to the street surface sector of the urban environment than to the pervious areas. The reason for this is that EPA has, until now, concentrated its studies on the street surfaces. Street surfaces contribute not only the expected pavement decomposition and automobile-associated by-products but also a variety of metals, pesticides, and nutrients, which are thought to be the result of atmospheric fallout. Studies conducted by the EPA have shown that the heaviest pollutant loadings are found on streets in industrial areas. Residential streets are less heavily loaded, and commercial streets have the lightest loadings.

Sartor and Boyd (Reference 15) found that the pollutant accumulation rate on street surfaces was constant for one to two days after a street cleaning or storm event; then it decreased asympotically. Amy and Pitt (Reference 16) reported a study of street debris samples from Chicago; it was determined that the very fine, silt-like portion of the debris (less than 43 u) which was less than 5.9 percent of the total weight of the debris, contained approximately one-fourth of the total oxygen demand, more than half of the heavy metals, and nearly three-fourths of the total pesticides. This is an important discovery because particle size affects the design of street sweeping equipment and stormwater treatment facilities, and can also affect transportation and accumulation of pollutants in receiving waters.

The Environmental Protection Agency studied removal rates of contaminants from paved surfaces by artificially produced runoff (Reference 16). Runoff rates were varied from 0.1 to 1.0 times per hour, and runoff durations were varied from 0.25 to 6.0 hours. It was found that removal percentage was a function of total runoff volume, but that it could not be precisely related to runoff intensity. Ninety percent removal resulted from 0.5 inches of total runoff. They developed the equation,

$$\% \text{ Removal} = (1 = e^{-P/.217}) * 100 \qquad (26)$$

where

P - total runoff in inches.

The coefficient 0.217 represents the easiest removal case; that is, for impervious surfaces. This coefficient would be expected to be larger for semi-pervious and pervious areas.

Using stormwater quality measurements from actual storm events, Barkdoll (Reference 13) determined generally larger runoff coefficients for Knoxville's Third Creek watershed. His runoff coefficients varied from contaminant to contaminant, whereas the EPA street surface coefficient was essentially constant for all contaminants.

In Bakersfield, California, EPA researchers artificially flushed streets and tested the quality of the runoff. They found that initially the runoff was quite dirty, but that the water quality improved as flushing continued, and Wilber and Hunter (Reference 17) in New Jersey and Colston (Reference 14) in North Carolina reported similar results for a variety of contaminants in an urban watershed. The graphical representation of pollutant concentration versus time during a storm event became known as a pollutograph. Most pollutographs exhibit what is referred to as the first-flush effect; that is, concentrations decrease as the storm progresses. Stormwater quality response may also be described by a loadograph, which is a plot of cumulative stream loads versus time during a storm event. Colston and Wilber, have shown that loading usually increases rather linearly with time at first, and then asymptotically approaches a limiting value.

Wilber found that high settleable solids concentrations are associated with the rising limb of the hydrograph, and high non-settleable solids concentrations are associated with the falling limb. He postulated that this effect is due to the scouring action of high-kinetic energy high volume flows which occur early in the storm, which can remove the larger particles. Since kinetic energy is velocity-related this hypothesis challenges the conclusion of other researchers that contaminant removal is not a function of velocity. Betson (Reference 8) believes that the large sediment loadings observed in Knoxville's Fourth Creek are caused at least in part by the high-velocity runoff from paved areas, which erodes the stream channel.

Barkdoll (Reference 13) used a parametric model similar to the EPA equation for contaminant removal from street surfaces. His equation (modified for use in a computer program) is

$$C^{*}RO = M_T(1 - e^{-RO/RO_C}) \quad (27)$$

where

C = average concentration during a storm event,

M_T = the total mass of contaminant material within a watershed which could be removed by runoff,

RO = runoff volume, and

RO_c = that amount of runoff which removes 63.2 percent of accumulated contaminants.

The variate C*RO is obviously an estimate of the total cumulative stream loading during the storm event. A nonlinear least squares program (DIFCOR) (Reference 18) was used to evaluate streamflow and water quality data in order to develop optimized values of the parameters M_T and RO_C is of course, the runoff coefficient.

b. Results

The results for the study watersheds, are shown in Tables 5 and 6. Barkdoll's results from Third Creek are also presented.

The RO_C values were found to be higher than the street surfaces for all contaminants. This is an indication that pervious areas contributed significantly to stream loadings. Total mass M_T was divided by the dustfall rate in order to determine the time period necessary for the accumulation of this mass. In nearly all cases M_T/D was longer than the average antecedent dry period for Third Creek (4.4 days), indicating that there were contributing sources other than dustfall.

It was realized that this model, also, cannot represent the system unless the watershed losses are considered. The loadings removed by a small portion of the potential runoff in the study watersheds cannot be expected to compare with those removed by the total potential runoff in the Third Creek watershed. Therefore, the parameters were recalculated, based on estimated total loads removed by estimated total runoff, in an approach similar to that used with the dustfall model. In order to do this, it was necessary to assume that the removal of contaminants into groundwater sinks follows a mechanism approximately the same as that by which surface runoff conveys contaminants into the stream. This would usually be a weak assumption because the laws of groundwater hydrology are somewhat different from those for surface water hydrology. However, in this case the assumption may have some validity because a large portion of the losses are thought to occur through discreet sinks or rapid-infiltration areas rather than through pervious soil surfaces in a uniform manner. Surface water and its associated loadings are conveyed to these sinks, of course, by the runoff process.

To allow comparison of the results between watersheds the M_T values were also normalized to a per-acre basis so that the effect of the varying sizes of the watersheds would be eliminated.

Table 7 gives normalized M_T values based on runoff volumes measured at the stream guage. Tables 8 and 9 show normalized M_T values and RO_C values based on the total volume of water available for runoff.

TABLE 5. VARIATION IN M_T AMONG WATERSHEDS

Group	Watershed / Constituent	Plantation Hills	First Creek	Third Creek	Fourth Creek
			M_T in Pounds		
Limestone Associated Minerals	CA	3,607	200.2	143,695	646.8
	MG	0.6045	22.10	1,808	187.6
	$CACO_3$	0.1492	656.6	44,084	2807
	ALK	17.50	501.7	-----	1480
Other Minerals	SIO_2	1.368	43.73	2,336	131.1
	NA	0.5238	33.86	2,761	90.00
	K	0.6288	16.00	----	759.7
	SO_4	7.100	141.2	22,299	697.9
	CL	1.505	39.11	2,347	199.2
	F	0.1013	2.306	76.34	2.004
Solids	SS	-----	343.3	1,145,804	85271
	TDS	100.0	1354	----------	3403
Metals	FE	0.3009	13.11	----------	1505
	MN	0.0125	0.4503	402.6	49.12
	PB	0.0050	1.046	95.04	8.022
	HG	-----	------	0.1100	0.0515
	AS	----	------	2.816	3.412

TABLE 5. VARIATION IN M_T AMONG WATERSHEDS (CONCLUDED)

Group	Constituent	Plantation Hills	First Creek	Third Creek	Fourth Creek
			M_T in Pounds		
Nutrients	ORGN	———	———	———	56.22
	NH_3N	0.0545	0.7169	———	6.019
	$NC_{2}.3$-N	0.2746	3.993	1074	13.22
	PO_{4T}	0.7101	11.00	———	120.0
	PO_4	0.6575	8.812	———	3.592
	COD	———	———	25,689	3749

TABLE 6. VARIATION IN RO_c VALUES AMONG WATERSHEDS

Watershed Group	Constituent	Plantation Hills	First Creek	Third Creek	Fourth Creek
			RO_c, in		
Limestone Associated Minerals	CA	0.0103	0.0700	21.0	0.1533
	MG	0.0104	0.0665	1.41	0.1794
	$CACO_3$	0.0042	0.0904	1.50	0.2111
	ALK	0.0102	0.0947	----	0.1670
Other Minerals	SiO_2	0.0105	0.0820	1.64	0.2378
	NA	0.0086	0.0923	1.92	0.2120
	K	0.0108	0.0578	___	0.2091
	SO_4	0.0113	0.1001	5.71	0.1857
	CL	0.0110	0.0957	0.99	0.1422
	F	0.0130	0.0853	2.16	0.0816
Solids	SS	___	0.0959	15.0	0.3178
	TDS	0.0367	0.0875	___	0.1897
Metals	FE	0.0099	0.0801	___	0.1521
	MN	0.0088	0.0897	2.78	0.2808
	PB	0.0129	0.0980	1.30	0.2040
	HG	___	___	0.50	0.1712
	AS	___	___	1.46	0.3744

TABLE 6. VARIATION IN RO_C VALUES AMONG WATERSHEDS (CONCLUDED)

Watershed		Plantation Hills	First Creek	Third Creek	Fourth Creek
Group	Constituent	RO_c, in			
	ORG-N	—	—	—	0.2256
	NH_3-N	0.0043	0.0753	—	0.2886
Nutrients	NO_2-+3-N	0.0109	0.0893	10.0	0.2777
	PO_{4T}	0.0117	0.0897	—	0.2074
	PO_4	0.0106	0.0490	—	0.1086
	COD	—	—	0.74	0.2590

TABLE 7. NORMALIZED M_T VALUES FOR STUDY WATERSHEDS

Group	Constituent	M_T/DA in lb/acre Watershed			
		Plantation Hills	First Creek	Third Creek	Fourth Creek
Limestone Associated Minerals	CA	0.0234	0.6256	140.3	1.232
	MG	0.0039	0.0690	1.766	0.3575
	$CACO_3$	0.0010	2.052	43.05	5.349
	ALK	0.1139	1.568	------	2.820
Other Minerals	SIO_2	0.0087	0.1366	2.281	0.2498
	NA	0.0034	0.1058	2.696	0.1715
	K	0.0041	0.0500	-----	1.448
	SO_4	0.0462	0.4412	21.78	1.158
	CL	0.0098	0.1222	2.292	0.3796
	F	0.0006	0.0072	0.0746	0.0038
Solids	SS	-----	1.073	1119	162.5
	TDS	0.6510	4.231	---------	6.484
Metals	FE	0.0020	0.0410	---------	2.867
	MN	0.0001	0.0014	0.3932	0.0094
	PB	0.00003	0.0033	0.0928	0.0153
	HG	-----	------	0.0001	0.0001
	AS	-----	------	0.0028	0.0065

TABLE 7. NORMALIZED M_T VALUES FOR STUDY WATERSHEDS (CONCLUDED)

Group	Constituent	M_T/DA in lb/acre Watershed			
		Plantation Hills	First Creek	Third Creek	Fourth Creek
	ORG-N	------	------	------	0.1071
	NH_3-N	0.0004	0.0022	------	0.0115
Nutrients	NO_2-+3-N	0.0018	0.0125	1.049	0.0252
	PO_{4T}	0.0046	0.0344	-----	0.2286
	PO_4	0.0043	0.0275	-----	0.0068
	COD	-----	------	25.09	7.144

TABLE 8. NORMALIZED M_T VALUES BASED ON PROJECTED RUNOFF VOLUMES

Watershed		Plantation Hills	First Creek	Third Creek	Fourth Creek
Group	Constituent				
		M_T, lb/acre			
Limestone Associated Minerals	CA	0.2127	3.293	140.3	2.464
	MG	0.0354	0.3632	1.766	0.7150
	$CACO_3$	0.0091	10.80	43.05	10.70
	ALK	1.035	8.252	-----	5.640
Other Minerals	SIO_2	0.0791	0.7189	2.281	0.4996
	NA	0.0309	0.5568	2.696	0.3430
	K	0.0372	0.2632	-----	2.898
	SO_4	0.4200	2.322	21.78	2.316
	CL	0.0891	0.6432	2.292	0.7472
	F	0.0054	0.0379	0.0746	0.0076
Solids	SS	------	5.647	1119	325.0
	TDS	5.918	22.26	------	12.97
Metals	FE	0.0182	0.2157	------	5.734
	MN	0.0009	0.0073	0.3932	0.0188
	PB	0.0003	0.0174	0.0928	0.0306
	HG	-----	------	0.0001	0.0002
	AS	------	------	0.0028	0.0130
Nutrients	ORG-N	------	------	------	0.2142
	NH_3-N	0.0036	0.0116	------	0.0230
	NO_2+3-N	0.0164	0.0658	1.049	0.0504
	PO_{4T}	0.0418	0.1810	------	0.4572
	PO_4	0.0391	0.1447	------	0.0136
	COD	------	-------	5.09	14.29

TABLE 9. RO_c VALUES BASED ON PROJECTED RUNOFF VOLUMES

Group	Constituent	Watershed			
		Plantation Hills	First Creek	Third Creek	Fourth Creek
Limestone Associated Minerals	CA	0.0936	0.3684	21.0	0.3066
	MG	0.0945	0.3500	1.41	0.3598
	$CACO_3$	0.0382	0.4758	1.50	0.4222
	ALK	0.0927	0.4984	-----	0.3340
Other Minerals	SIO_2	0.0954	0.4316	1.64	0.4756
	NA	0.0782	0.4858	1.92	0.4240
	K	0.0982	0.3042	----	0.4182
	SO_4	0.1027	0.5268	5.71	0.3714
	CL	0.1000	0.5037	0.99	0.2844
	F	0.1182	0.4489	2.16	0.1632
Solida	SS	------	0.5047	15.0	0.6356
	TDS	0.3336	0.4605	------	0.3794
Metals	FE	0.0900	0.4216	------	0.3042
	MN	0.0800	0.4721	2.78	0.5616
	PB	0.1173	0.5158	1.30	0.4080
	HG	------	------	0.50	0.3424
	AS	-------	-------	1.46	0.7488
Nutrients	ORG-N	------	-------	------	0.4512
	NH_3-N	0.0391	0.3963	------	0.5772
	NO_2-+3-N	0.0991	0.4700	10.0	0.5554
	PO_{4T}	0.1064	0.4721	-----	0.4148
	PO_4	0.0964	0.2579	-----	0.2172
	COD	------	-------	0.74	0.5180

Table 10 presents coefficients of determination calculated from observed and predicted loadings for the study watersheds. Coefficients of determination for Barkdoll's work are also shown.

The M_T values were divided by dustfall rates so that accumulation times for the various constituents in the study watersheds could be compared with those computed by Barkdoll for Third Creek. These ratios expressed in days, are shown in Table 11. The average antecedent dry period for the watersheds is about four days.

One constituent having a relatively complete representation across the study watersheds and a relatively high coefficient of determination was chosen from each group. For these constituents the M_T/D ratios were recalculated from the adjusted M_T values given in Table 8. It was hoped that this approach would provide a clearer understanding of the actual contribution of dustfall to pollutant loading in these watersheds. Adjusted M_T/D ratios are given in Table 12.

c. Statistical Reliability

Overton and Meadows (Reference 3) assert that if a model produces different predictions or simulations given different scenarios for a particular problem, these variations need to be placed relative to the errors associated in fitting the model to the data. In order to gain understanding of the significance of the variations in the data for this study, the Wilcoxson test was used to determine the significance of between-watershed variations in normalized measured loadings. It was hoped that a context would be provided for assessing the significance of predicted M_T and RO_C values.

It was hoped that one constituent for which analysis was complete in all watersheds and which produced a high coefficient of determination could be chosen to represent each group. However, no constituent of the non-limestone-associated metals met these criteria. Lead was the only constituent meeting the needs of this test for which data from Third Creek was readily available. The Plantation Hills watershed was not included in this analysis because of the uncertainties concerning the data which were previously mentioned. It was felt that First Creek could be considered to be representative of a residential watershed, and Third Creek and Fourth Creek could be considered to be representative of industrial and commercial watersheds, respectively. Results obtained through the Wilcoxson two-sample test are shown in Table 13.

Note that the null hypothesis that the population means for normalized stormwater loadings in two different watersheds are equal is

TABLE 10. AVERAGE COEFFICIENT OF DETERMINATION FOR VARIOUS WATER QUALITY CONSTITUENTS - PARAMETRIC MODEL

Group	Constituent	Avg Coeff of Det	Number of cases (Watersheds)
Limestone	$CACO_3$	0.86	4
Associated	ALK	0.81	3
Minerals	CA	0.81	4
	MG	0.78	4
	NA	0.74	4
	K	0.72	3
Other	CL	0.67	4
Minerals	F	0.60	4
	SO_4	0.59	4
	SIO_2	0.55	4
Solids	TDS	0.67	3
	SS	0.35	2
	AS	0.81	2
	FE	0.73	3
Metals	HG	0.71	2
	PB	0.50	4
	MN	0.47	4
	ORG-N	0.92	1
	PO_{4T}	0.88	3
Nutrients	COD	0.80	2
	NO_2-+3-N	0.77	4
	PO_4	0.76	3
	NH_3-N	0.50	3

TABLE 11. COMPARISON OF M_T/D RATIOS AMONG WATERSHEDS

Group	Constituent	Watershed			
		Plantation Hills	First Creek	Third Creek	Fourth Creek
		M_T/D, days			
Limestone Associated Minerals	CA	0.315	27.38	1070	11.85
	MG	0.305	4.28	118	12.29
	$CACO_3$	0.0020	4.07	80	9.66
	ALK	0.398	8.88	-----	11.16
Other Minerals	SIO_2	0.702	4.66	41	21.20
	NA	0.113	2.76	90	7.63
	K	0.215	3.74	----	22.07
	SO_4	0.214	3.20	----	4.18
	CL	0.179	1.37	14	1.90
	F	0.106	1.51	25	1.43
Solids	SS	-----	16.68	320	289.8
	TDS	2.57	14.48	---	3.30
Metals	FE	1.82	20.46	---	348.6
	MN	0.326	4.26	366	39.36
	PB	-----	4.32	61	2.74
	HG	-----	----	6	4.29
	AS	-----	----	20	27.34
Nutrients	ORG-N	----	----	---	1.38
	NH_3-N	0.099	0.713	---	0.44
	NO_{2+3}-N	0.421	1.67	40	1.26
	PO_{4T}	2.21	3.61	--	3.00
	PO_4	4.49	4.41	--	----
	COD	----	----	14	----

TABLE 12. SELECTED M_T/D RATIOS ADJUSTED FOR PROJECTED RUNOFF VOLUMES

Constituent	Watershed			
	Plantation Hills	First Creek	Third Creek	Fourth Creek
	M_T/D, days			
$CACO_3$	0.018	21.4	80	19.3
K	1.95	19.7	--	44.1
FE	16.5	107.7	--	697.2
TDS	23.4	76.2	--	6.6
PO_4	40.8	23.2	--	---

TABLE 13. WILCOXSON TWO-SAMPLE TEST

Ho: $\mu_1 = \mu_2$, $\alpha = 0.05$ Significance Level

Watersheds Compared	Constituent	Critical Region	μ (Probability)
First Creek	$CACO_3$	$\mu \leq 12$	26
versus	TDS	$\mu \leq 17$	34
Fourth Creek	PB	Pr(_22.51/Ho true) _.05	Pr(_22.5/Ho true) = .178
	NO_2-NO_3	$\mu \leq 12$	26
First Creek			
versus	PB	29	25
Third Creek			
Third Creek			
versus	PB	29	33

accepted at the 0.05 level of significance in every case except for the comparison of lead between First Creek and Third Creek.

The Wilcoxson Test for paired observations was used to determine whether there were significant differences between RO_c values for different pollutant groups. The calculated RO_c values given in Table 9 representing two watersheds for a given constituent were considered to be a pair. $CaCo_3$, Pb, and NO_2-NO_3 represented the limestone associated minerals, heavy metals, and nutrients, respectively. In no case were the differences statistically significant at the $\alpha = 0.05$ significance level.

d. Conclusions on Pollutant Accumulation and Removal

Of course the dustfall model predicts constituent concentrations during storm events and the mass balance compares annual input (dustfall) and output (stormwater) loadings. Actually, the model structures are similar, and since both approaches address the question of the degree to which dustfall is responsible for stormwater pollution it was hoped that the results could be compared in order that our understanding of the magnitude of the dustfall contribution for various constituents could be better understood. In all cases where there were discrepancies between the two approaches the dustfall model overpredicted while the mass balance showed that inputs were actually smaller than outputs. It has previously suggested that concentrations predicted by the deterministic model may have been too high because not all of the dilution water was taken into account. Betson's input loadings could also have been too small. They were based on precipitation volumes taken from the atmospheric samplers; since these samplers were only serviced at approximately monthly intervals, evaporation losses could have been considerable.

The removal model predicted large differences in ultimate pollutant loading M_T between watersheds if runoff volumes measured at the stream gauge were used. The prediction that the loadings are heaviest in the industrial Third Creek watershed is consistent with Sartor and Boyd's work; the prediction that loadings in the commercial Fourth Creek watershed are generally about double those of residential First Creek is not. This could be a real occurrence, but this effect could also be due to the difference between bypassed losses for the two watersheds. Table 8 shows that if projected available runoff volumes were used to determine the parameters the differences in M_T values between the two watersheds became slight. This response is more consistent with the results of other work in the field (Reference 15). Notice that lead and suspended solids were still relatively high for the shopping centers and on the highways in this watershed. The suspended solids loadings could be elevated because of the erosion of the streambed by high-volume high-velocity stormwater runoff.

RO_c values computed using measured runoff volumes were less than the EPA street surface runoff coefficient in all of the watersheds except for Third Creek. This indicates that most contaminants which are actually washed into the streams originate from impervious surfaces. If the projected total available runoff volumes were used, the RO_c values became larger than the street surface coefficient for most constituents in the First and Fourth Creek watersheds, indicating that pervious surfaces may also be involved in the total contaminant removal process. It appears that contaminants which find their way into First and Fourth Creeks are mostly originating in the paved areas of these watersheds, and that most contaminants associated with the pervious areas are lost into the groundwater channels. If this conclusion is valid, it would certainly have implications for stormwater management strategies in these watersheds.

There does not appear to be a pervious area contribution to even the total pollutant removal process in the Plantation Hills watershed, as the revised RO_c values are smaller than the street surface coefficient for all constituents except for TDS.

The large RO_c values for Third Creek indicate contaminant removal from pervious areas. Removal only by direct runoff mechanisms is not necessarily implied. A portion of the contaminant loadings could come from the foundry's settling pond, which only overflows into the creek when surface runoff feeding the pond reaches a certain magnitude. In addition, it should be remembered that subsurface leaching could be a source of pollution, since groundwater flows feed the stream channel in this watershed. Groundwater flow is indirectly linked with surface runoff, since runoff volume affects infiltration rates. Therefore, the large RO_c values could be reflecting this process as well as surface runoff per se.

If M_T/D ratios are determined from measured runoff volumes, the conclusion would be that dustfall could account for all contaminant accumulations in Plantation Hills; for all minerals except for Ca; for all metals except for Fe; and for all nutrients in First Creek; and for the minerals SO_4, Cl, and F; dissolved soils; the metals Pb and As; and all nutrients in the Fourth Creek watershed. Only in the case of Hg does the M_T/D ratio approach the typical antecedent dry period for Third Creek.

It should be realized that the M_T values used in this determination for the Plantation Hills, First Creek, and Fourth Creek watersheds represent only the fraction of the total mass which contributes to stream loadings, while D represents the atmospheric input to the entire watershed. Therefore these figures could be misleading. If M_T values representing the total mass contributing to all vectors of contaminant' removal from these watersheds is used, the M_T/D ratios are more similar to those developed by Barkdoll for Third Creek, indicating that dustfall should not be considered to be the only contributor to pollutant loading in the study watersheds. (See Table 11).

The Wilcoxson Test does not indicate statistically significant differences between loadings during storm events for any constituents in any two watersheds except in the case of lead, between First Creek and Third Creek. Differences between the results in the study watersheds for the various constituents should be interpreted relative to this fact as well as to the size and quality of the data base.

No inference could be drawn concerning the behavior of the various pollutant groups for different land used from this study. Land use appeared to have an affect, but this effect was different for each pollutant.

6. REGIONALIZATION SCHEMES - STORMWATER DISCHARGE

a. Linear Programming Model

Because of the very low correlations of Betson's first generation regionalization model (stepwise regression) in paragraph 1, the reliability of the model is low. Hence, another approach to optimizing the same model form of Equation 9 was taken. Linear programming was used whereby the objective function was to minimize the sum of the differences between the observed and the model (rather than squares) and the constraints were 354 linear equations which related the watershed and storm characteristics, for each storm, to the associated optimized unit response function parameters, UP, T1, T2, and T3.

The data base was split between urban and rural because of the significant scale effect between the two data bases. The results are shown in Table 14. The correlations have been improved, but not to a level which permits simulations with a high degree of statistical confidence. Further, the optimized model coefficients are very inconsistent, and hence do not permit the drawing of an inference as to drainage process.

An alternative approach to regionalization scheme, therefore, is needed and is presented next.

TABLE 14. REGIONALIZATION OF TVA STORMWATER MODEL PARAMETERS BY LINEAR PROGRAMMING

	MODEL PARAMETER	CONSTANT	AREA	SHAPE	SC	DD	SINU	PF	CN	SS	SRO	NFF	R^2
RURAL	UP	$e^{10.22466}$	-0.24286	-0.27402	0.16972	0.34778	0.05535	-1.01075	-1.99886	-4.00839	0.10117	-0.21996	0.85
	T1	$e^{-0.21757}$	0.16052	0.01712	-0.28252	-0.18869	0.19953	0.23333	0.38219	3.62603	-0.02216	0.22835	0.58
	T2	$e^{-3.74790}$	0.18462	-0.04532	-0.17176	-0.06159	0.11196	0.44352	1.06753	1.59401	-0.00648	0.19932	0.59
	T3	$e^{-2.92311}$	0.16047	-0.41439	-0.05747	-0.17167	0.16743	0.44228	1.29012	3.47289	0.03862	0.15507	0.41
URBAN	UP	$e^0 = 1$	-0.76167	0.0	0.0	0.0	-0.66280	0.09401	0.0	-1.68125	-0.17789	-0.20933	0.72
	T1	$e^0 = 1$	1.03123	0.0	0.0	0.0	0.14521	-0.20801	0.0	0.14718	0.0	0.0	0.46
	T2	$e^0 = 1$	0.71828	0.0	0.0	0.0	-0.10619	0.33802	0.0	1.87289	0.27625	0.8861	0.77
	T3	$e^0 = 1$	0.53174	0.02402	0.0	0.76171	0.0	0.0	0.0	2.03459	0.18510	0.19797	0.55

b. Lag Modulus Model

The lag modulus approach attempts to filter out the effect of the nonlinearity (rain excess intensity) from the right-hand side of the regionalization model. This is done by derivation of a lag modulus which relates the generating rain excess intensity to the associated lag time or T1 for each storm.

(1) Linear and Nonlinear Systems

Linear and nonlinear systems are defined by linear and nonlinear differential equations, respectively. In hydrologic terms this means that the response from a catchment watershed will remain constant for a prescribed set of boundary conditions. The response of a watershed is represented as a unit hydrograph; hence, for a specified duration of rainfall D the unit hydrograph U(D,t) is found by dividing the storm hydrograph ordinates by the associated volume of rainfall excess P_e. This forms a new hydrograph, i.e. the unit hydrograph, with 1 inch of runoff volume beneath it.

The volume under the storm hydrograph (see Figure 17) is

$$P_e = \int_0^\infty Q(t)\,dt = i_e * D \tag{28}$$

where Q is stormwater discharge. The unit hydrograph then is

$$U(D,t) = Q(t)/P_e \tag{29}$$

since the volume beneath it is seen to be 1 inch, from Equation 28

$$1\text{ inch} = \int_0^\infty \frac{Q(t)}{P_e}\,dt \tag{30}$$

The unit hydrograph concept says that for a given land use, initial moisture content, and rainfall excess duration, the unit hydrograph will be the same for each storm. Much evidence has been reported which has shown that the unit hydrograph is also a function of rainfall excess intensity. This simply means that the system is nonlinear. The example of Minshall (Reference 11), shown in Figure 18, illustrates this variation on a small agricultural watershed. These five storms have nearly the same duration of rainfall excess but have widely varying rainfall excess intensities and this resulted in the wide variation in unit hydrographs as shown in Figure 18. The variation of lag time with rain excess intensity was shown by Overton (Reference 12)

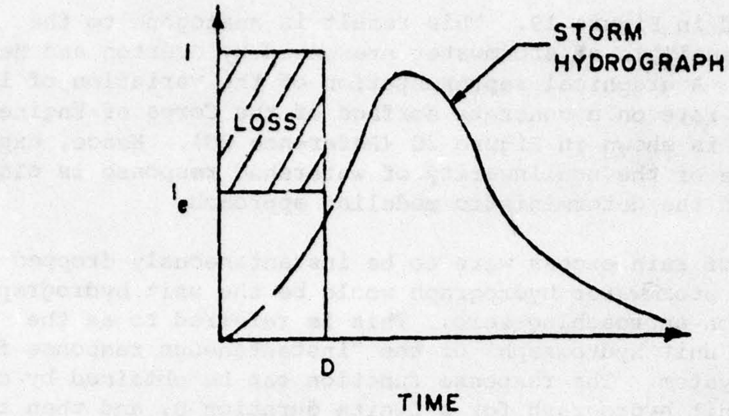

Figure 17. The Unit Hydrograph Concept.
(Reference 3)

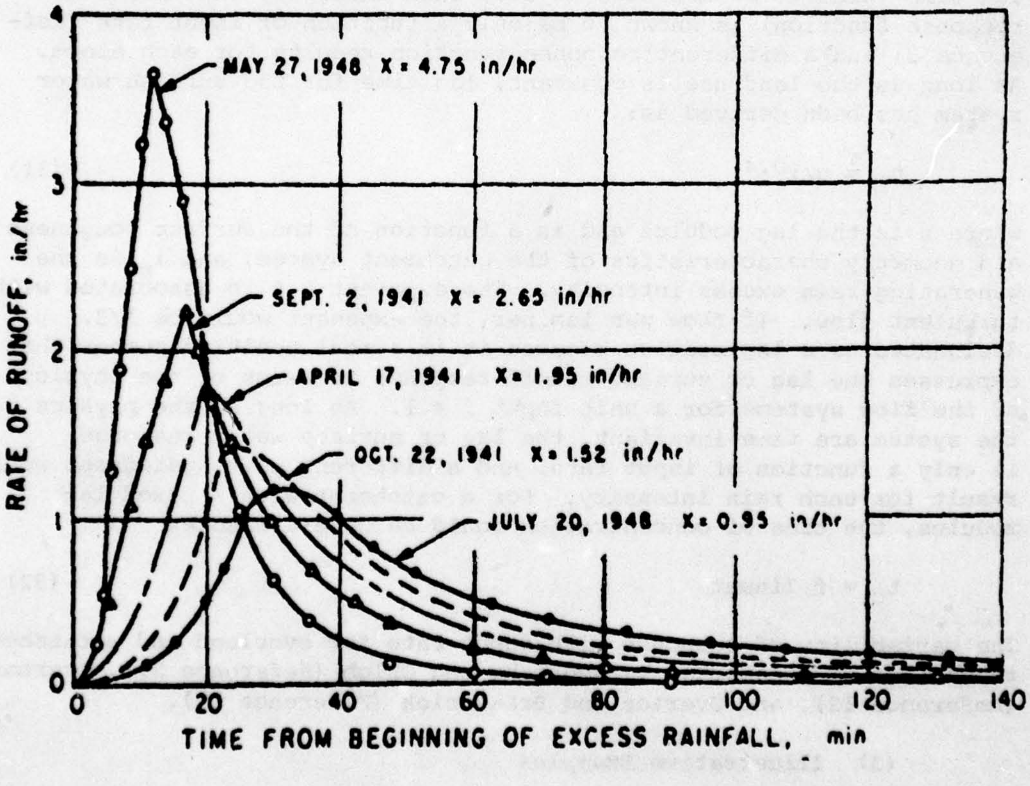

Figure 18. The Example of Minshall
(Reference 11)

and is repeated in Figure 19. This result is analogous to the deterministic analysis of stormwater presented by Overton and Meadows (Reference 3). A graphical representation of the variation of lag time with rain rate on a concrete surface of the Corps of Engineers (Reference 19) is shown in Figure 20 (Reference 20). Hence, experimental evidence of the nonlinearity of watershed response is closely correlated with the deterministic modeling approach.

If 1 inch of rain excess were to be instantaneously dropped on a watershed, the stormwater hydrograph would be the unit hydrograph for a storm duration approaching zero. This is referred to as the "instantaneous unit hydrograph" or the "instantaneous response function" (IRF) of the system. The response function can be obtained by convoluting the unit hydrograph for a finite duration D, and then taking the first derivative with respect to time.

(2) Lag Modulus

From deterministic stormwater modeling of surface water systems, lag time (time to occurrences of 0.50 inch under the instantaneous response function) is known to be only a function of input rate (Reference 3) and a different response function results for each storm. As long as the land use is constant, lag time for the surface water system has been derived as:

$$t_L = \mu / i_e^{0.4} \qquad (31)$$

where μ is the lag modulus and is a function of the surface roughness and geometry characteristics of the catchment system, and i_e is the generating rain excess intensity. The exponent 0.4 is associated with turbulent flow. If flow was laminar, the exponent would be 1/3. μ designated as a lag modulus because it is a real positive number that expresses the lag or surface runoff response in terms of the physics of the flow systems for a unit input $i = 1$. As long as the physics of the system are time-invariant, the lag or surface water response is only a function of input rate, and a different unit hydrograph would result for each rain intensity. For a catchment with a fixed lag modulus, the time of concentration could be generalized as

$$t_c = f\ (\text{input}) \qquad (32)$$

The variability of response with input rate for overland and watershed runoff has been reported by Amorocho and Orlob (Reference 21), Overton (Reference 12), and Overton and Brakensiek (Reference 22).

(3) Illustrative Examples

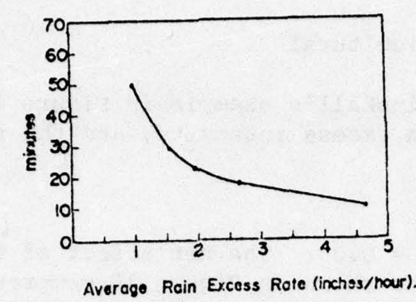

Figure 19. Variation of Lagtime of Unit Hydrographs of Minshall Example. Lagtime Versus Supply Rate. (Reference 12)

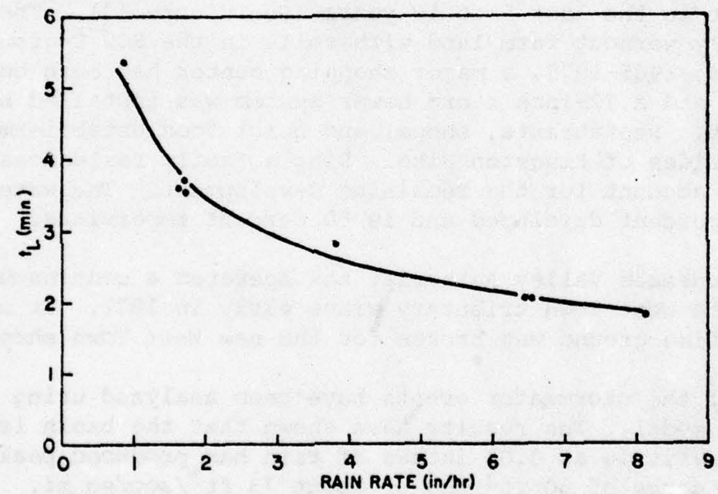

Figure 20. Lagtime Versus Rain Rate. Concrete, ½ Percent Slope, 84 Feet.

Example 1: Agricultural

Lag time from Minshall's example in Figure 18 was correlated with the associated rain excess intensity, and the results were:

$$t_L = 44/i_e \qquad (32)$$

where $R^2 = 0.98$ and SEE = 0.08. The net effect of this exercise is that the five unit hydrographs shown in Figure 18 compress to a single variable unit hydrograph shown in Figure 21. The coefficient of variation around this variable unit hydrograph is 10 percent.

Example 2: Urban

The West Town tributary to Fourth Creek, Knoxville, Tennessee, is a 0.82 square mile watershed which has undergone substantial development in the last 5 to 10 years (See Figure 22). The watershed is basically wornout farm land with soils in the SCS C-group. In the past decade, 1965-1975, a major shopping center has been built in the headwaters and a 72-inch storm sewer system was installed beneath the parking lot. Restaurants, shops, and quick food establishments now line both sides of Kingston Pike. Single family residences and garden apartments account for the remaining development. The watershed is about 100 percent developed and is 60 percent impervious.

The Tennessee Valley Authority has operated a continuous stream gauge on the West Town tributary since early in 1971. It was installed about the time ground was broken for the new West Town shopping center.

Some of the stormwater events have been analyzed using the TVA stormwater model. The results have shown that the basin is very flashy. As little as 0.05 inches of rain has produced peak flows at the stream gauge of 60 ft^3/sec or about 73 ft^3/sec/sq mi.

High quality data for eight storms were analyzed for the period of spring and summer of 1972. This was completed after the West Town Mall was completed and the watershed was essentially 100 percent developed. The storms were all short duration with fairly intense rainfall. A DT of 5 minutes was used for all storms and the eight unit response functions derived from the stormwater hydrographs are shown in Figure 23. At a glance the results seem to widely vary. The optimized SCS CN lie between 85 and 100 indicating in some instances that a 100 percent runoff condition occurs.

An explanation of the variation of the "urf" from storm to storm will be attempted. Time to peak of the "urf" is plotted versus the

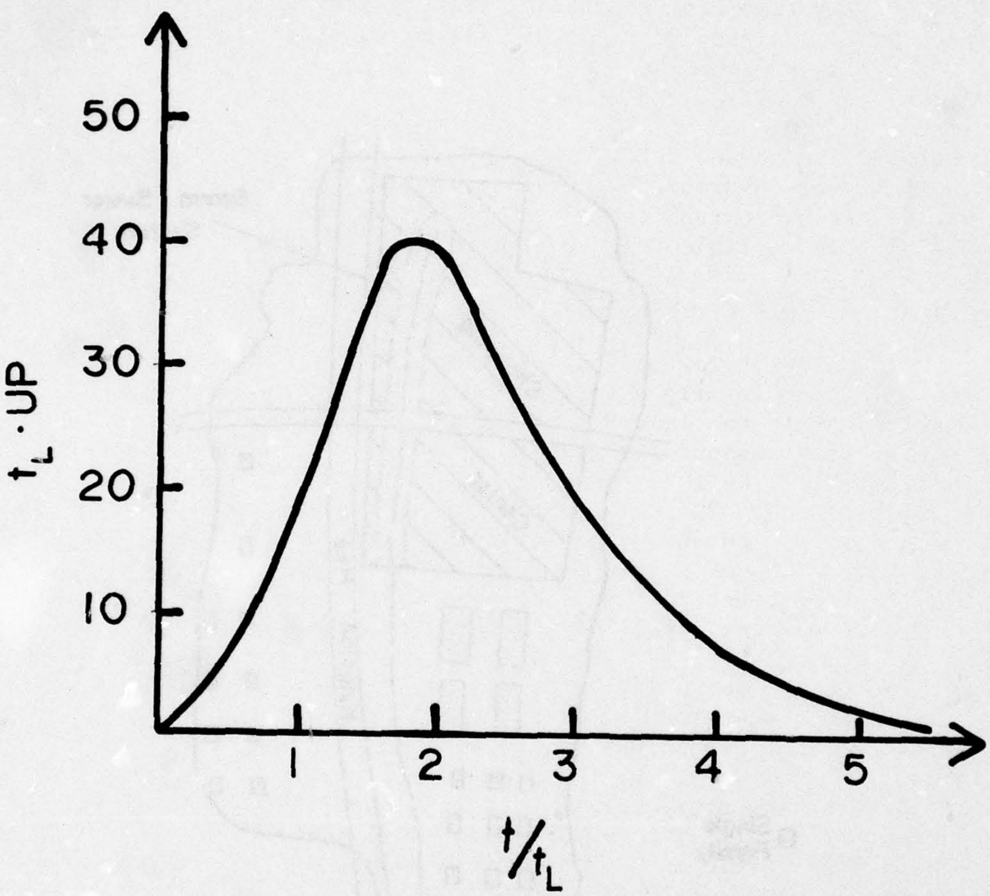

Figure 21. Variable Unit Hydrograph for the Minshall (Reference 11) Watershed

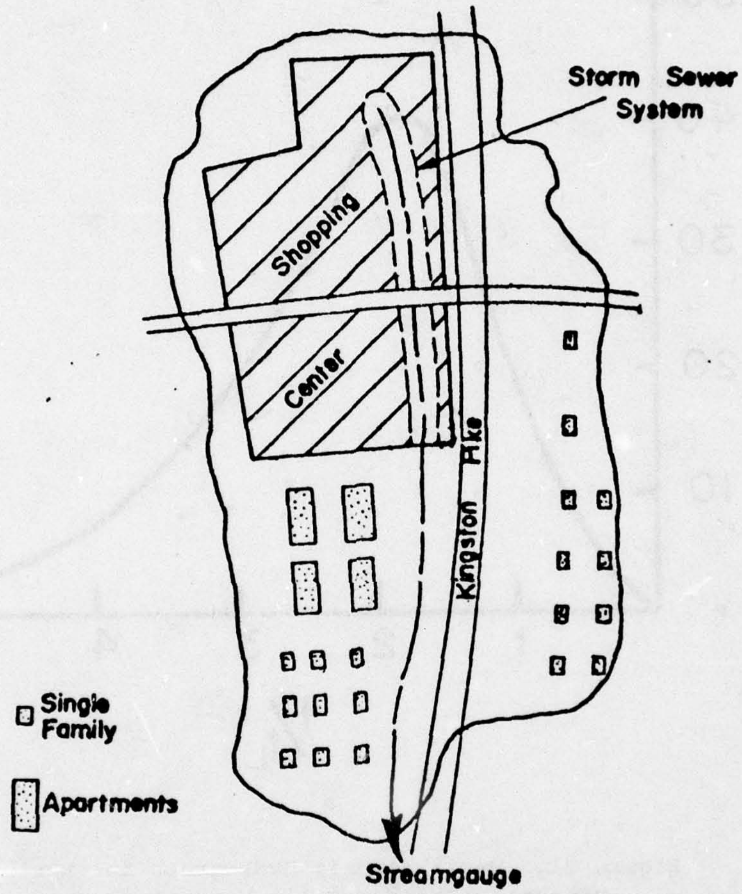

Figure 22. West Town Tributary to Fourth Creek.
Knoxville, Tennessee (Reference 3)

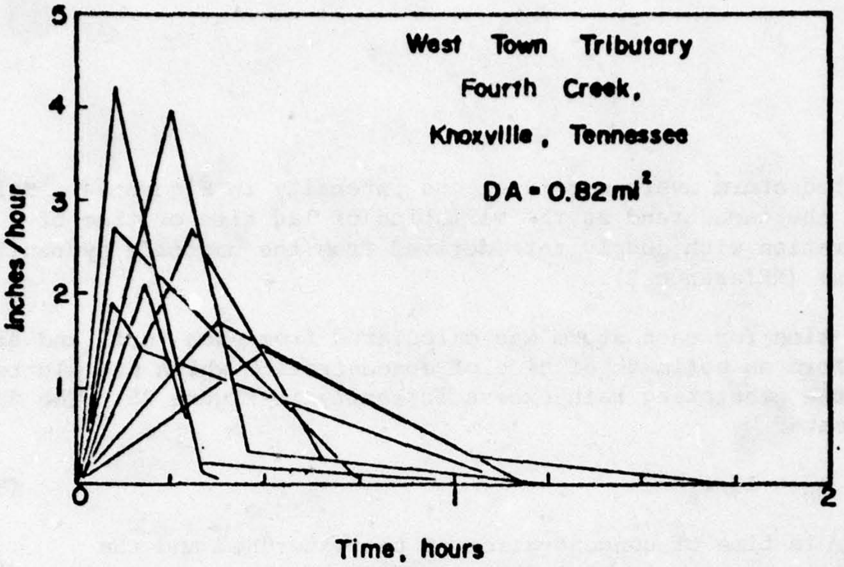

Figure 23. Five-Minute Response Functions, April Through September 1972 (Reference 3)

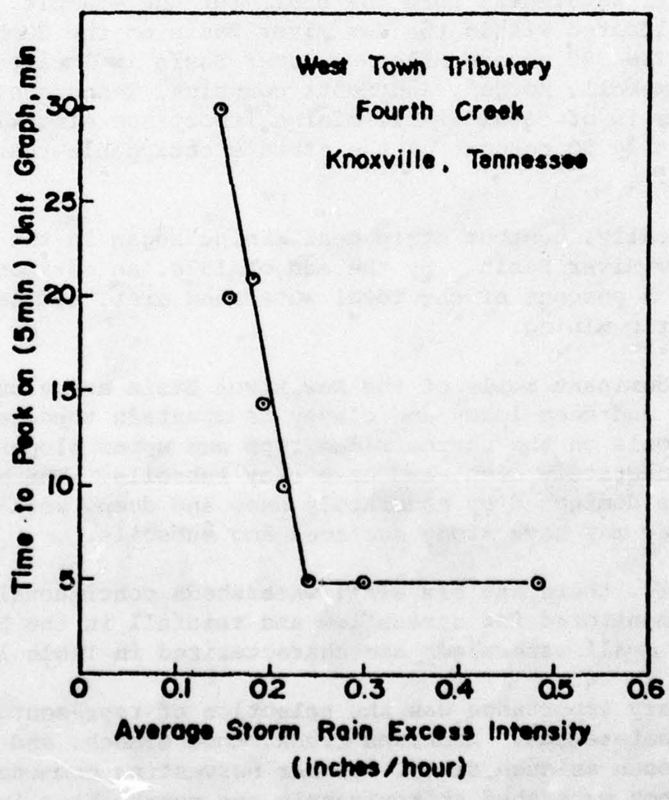

Figure 24. Time to Peak as a Function of Rain Excess Rate for Summer Storms of 1972 (Reference 3)

associated storm average rain excess intensity in Figure 24. This follows the same trend as the variation of lag time or time of concentration with supply rate derived from the unsteady hydraulic equations (REference 3).

Lag time for each storm was calculated from each "urf" and divided by 0.6 to form an estimate of time of concentration which was plotted versus the generating rain excess intensity in Figure 25. The data are approximated by

$$t_c = 13/i_e^{0.4} \tag{33}$$

where t_c is time of concentration of the watershed and the value 13 is an experimental determination of the lag modulus.

Example 3: Coal Strip Mining

Six small watersheds form the basis for the example. These watersheds are located within the New River Basin on the Cumberland Plateau. The 382 square mile New River Basin is located in parts of Anderson, Campbell, Morgan, and Scott counties, Tennessee. This four-county area is of considerable mining importance since it contains approximately 50 percent of the state's strippable reserves of bituminous coal.

Historically, contour strip coal mining began in the early 1940s in the New River Basin. By the end of 1974, an estimated 12,000 acres, or about 5 percent of the total watershed area, had been disturbed by strip mining.

The predominant soils of the New River Basin are classified as moderately deep and deep loamy and clayey on mountain topography. Commonly, the soils on the narrow ridge tops and upper slopes are well drained, moderately deep, and have clay subsoils. The middle and lower slopes are dominated by moderately deep and deep, well-drained loamy soils; they may have stony surfaces and subsoils.

Currently, there are six small watersheds continuously and simultaneously monitored for streamflow and rainfall in the New River Basin. The six small watersheds are characterized in Table 15.

Of primary importance was the selection of representative unmined (virgin) watersheds. Anderson Creek, Lowe Branch, and Bowling Branch, were chosen as such sites. Timber harvesting commenced in the Bowling Branch watershed approximately one month after installa-

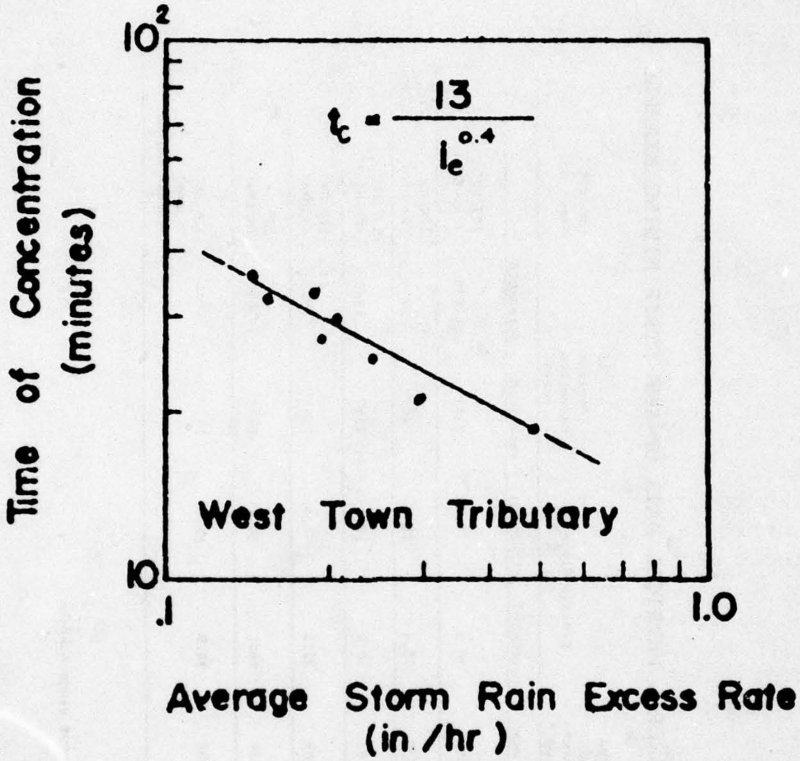

Figure 25. Time of Concentration as a Function of Rain Excess Rate for Summer Storms of 1972. (reference 3)

TABLE 15. GENERAL PHYSICAL DATA OF THE STRIP MINING EXAMPLE

Watershed	Area of Watershed [a]	Average Slope (%)		Average Elevation)Feet)		USG/TVA Sheet No.
	Acres	Watershed	Streambed	Watershed	Streambed	
Indian (A) Fork	2765	38.1	8.4	2341	1830	129 NW (Fk. Mtn)
Bill's (D) Branch	429	38.4	26.0	2407	2130	129 NW (Fk Mtn)
Green (F) Branch	883	36.8	17.6	2287	1845	129 NW (Fk Mtn)
Bowling (E) Branch	1888	32.5	7.4	1967	1580	128 SW (Norma)
Anderson (B) Creek	518	34.1	18.4	1890	1630	128 SW (Norma)
Lowe (C) Branch	589	31.9	12.6	1820	1540	128 SW (Norma) (Nearest Norma)

a Above continuous gauge station

tion of the gauges, thus removing it from consideration as a purely virgin basin in this particular phase of the overall study. Continuous rainfall/streamflow recorders were installed on Anderson Creek in November 1975. In July 1976, a coal company began construction of haul roads in the Anderson Creek watershed opening that area to strip mining. This will offer an invaluable opportunity for the collection and analysis of data during and following mining for comparison with virgin conditions as represented in this study. To date, Lowe Branch remains in virgin condition with no apparent impending mining activity.

The remaining three watersheds have been subjected to strip mining to varying extents. All of the mining activity in Indian Fork occurred prior to enactment and enforcement of surface mining regulations by the State of Tennessee. Recognizing that unregulated mining will not be allowed in the future, stormwater analysis efforts were concentrated on Bill's Branch and Green Branch. The differing hydrologic effects of preregulation mining of the Jellico seam in Green Branch is considered to be negligible because less than one percent of the watershed is affected.

A wide range of storm levels were recorded on the study watersheds during the monitoring period. The highest magnitude storm occurred on July 5, 1976, on the Green Branch watershed. This storm, which was approximately equal to the one hour, two year frequency storm for the area, produced a peak flow of 167 cfs. Rain excess intensities were determined by summation of incremental runoff intensities weighted by their contribution to total runoff. Values ranged from 0.097 in/hr to 0.865 in/hr for the various storms analyzed.

The coefficients of determination, calculated over the entire storm, ranged from a low of 0.84 to a high of 0.99 with the average being 0.94 for all four basins. Poorer overall fits were produced for multiple peak storms because the "urf" optimized on the largest burst was used in convoluting the entire hydrograph.

Figures 26, 27, 28, and 29 graphically represent the results of storms analyzed on the study watersheds. Time intervals of 5, 10, 15, 20, 30, and 60 minutes were used in the initial phase of the study. It was determined that a DT of 15 minutes offered a favorable balance between hydrograph definition and computational efficiency.

Since all storms were analyzed at a common DT, classical unit hydrograph theory would require that a single unit response function would represent the response of a given watershed. Clearly, the

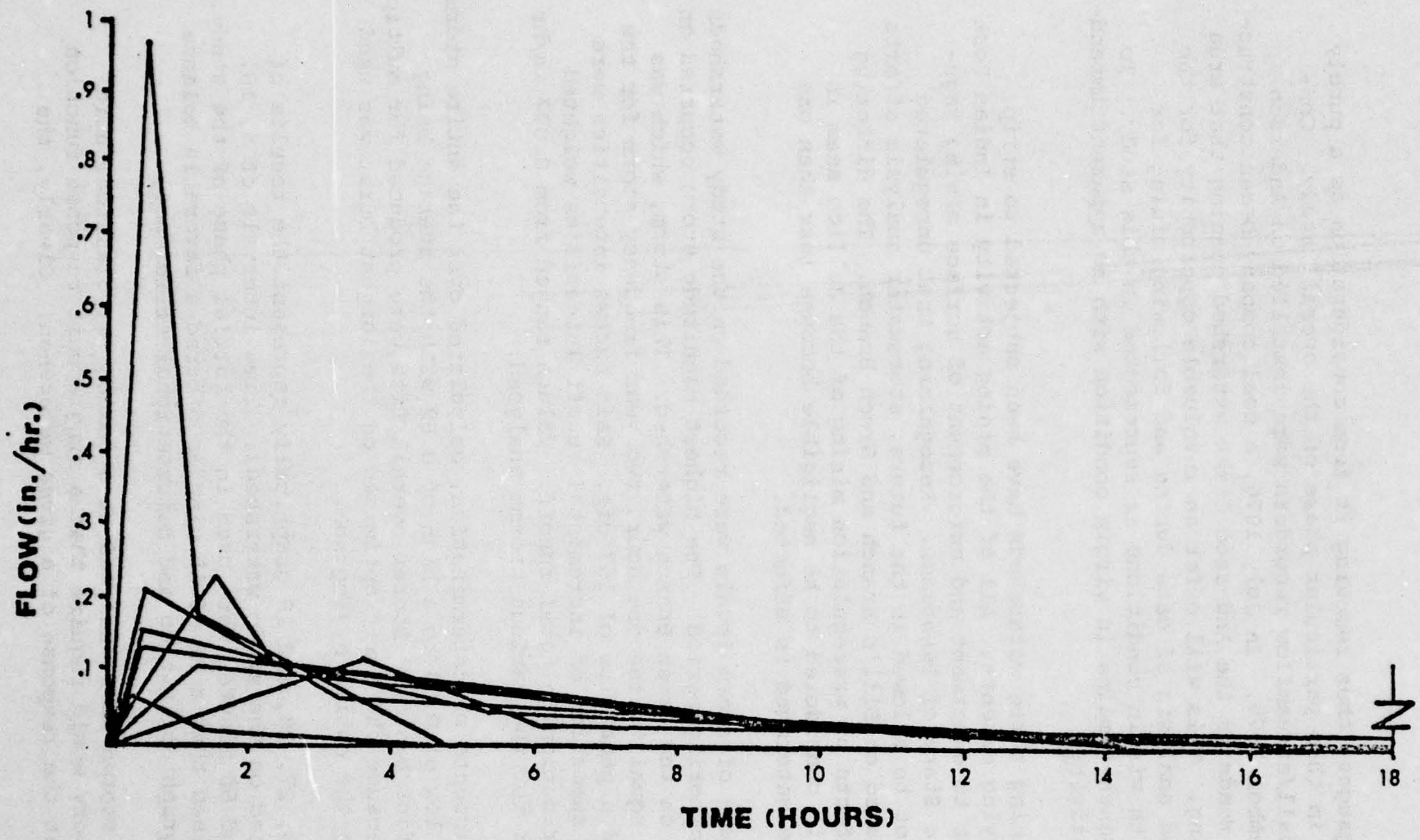

Figure 26. Optimized Unit Response Functions for Bill's Branch Storms (DT = 0.25 hr)

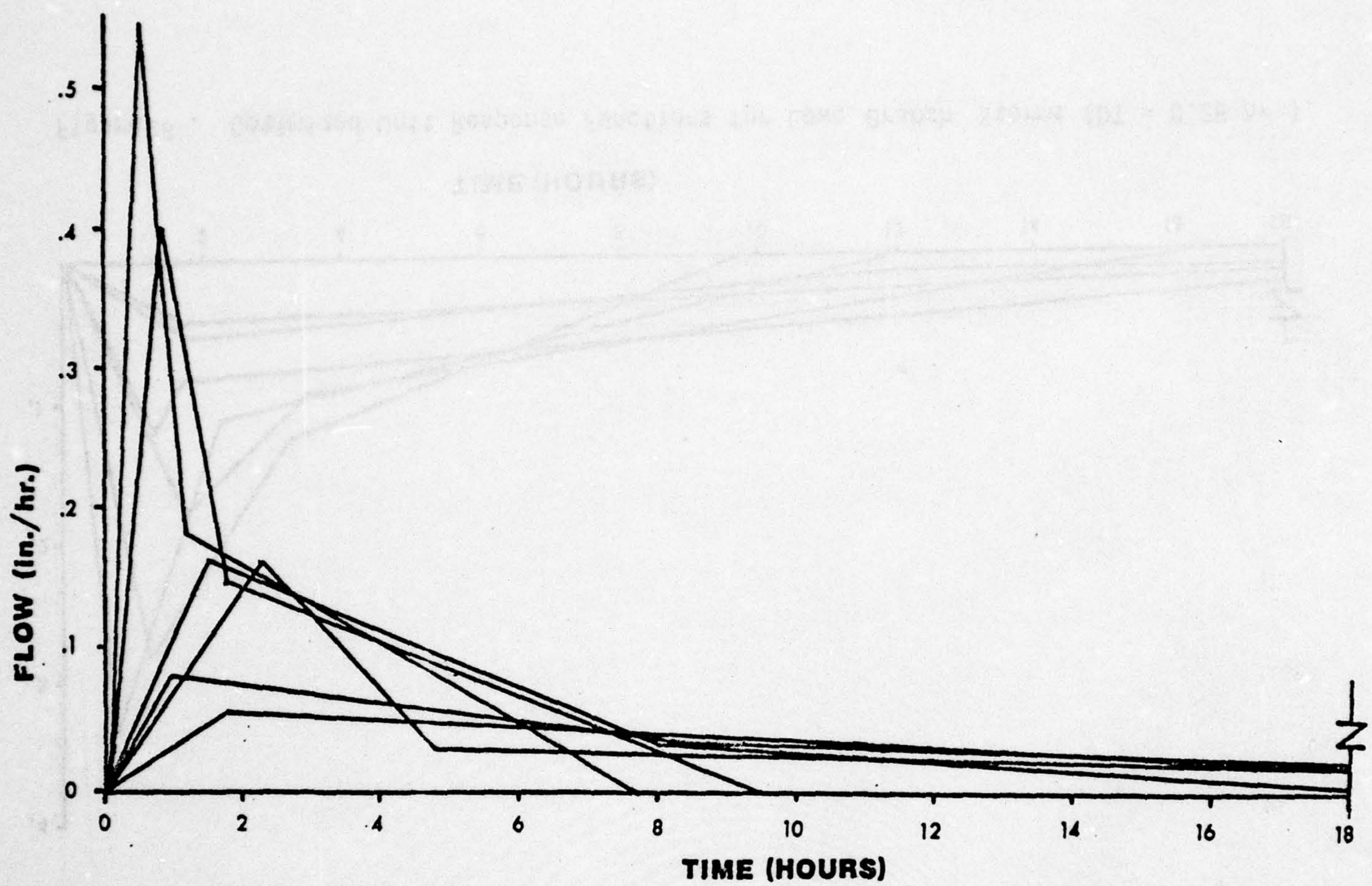

Figure 27. Optimized Unit Response Functions for Green Branch Storms (DT = 0.25 hr).

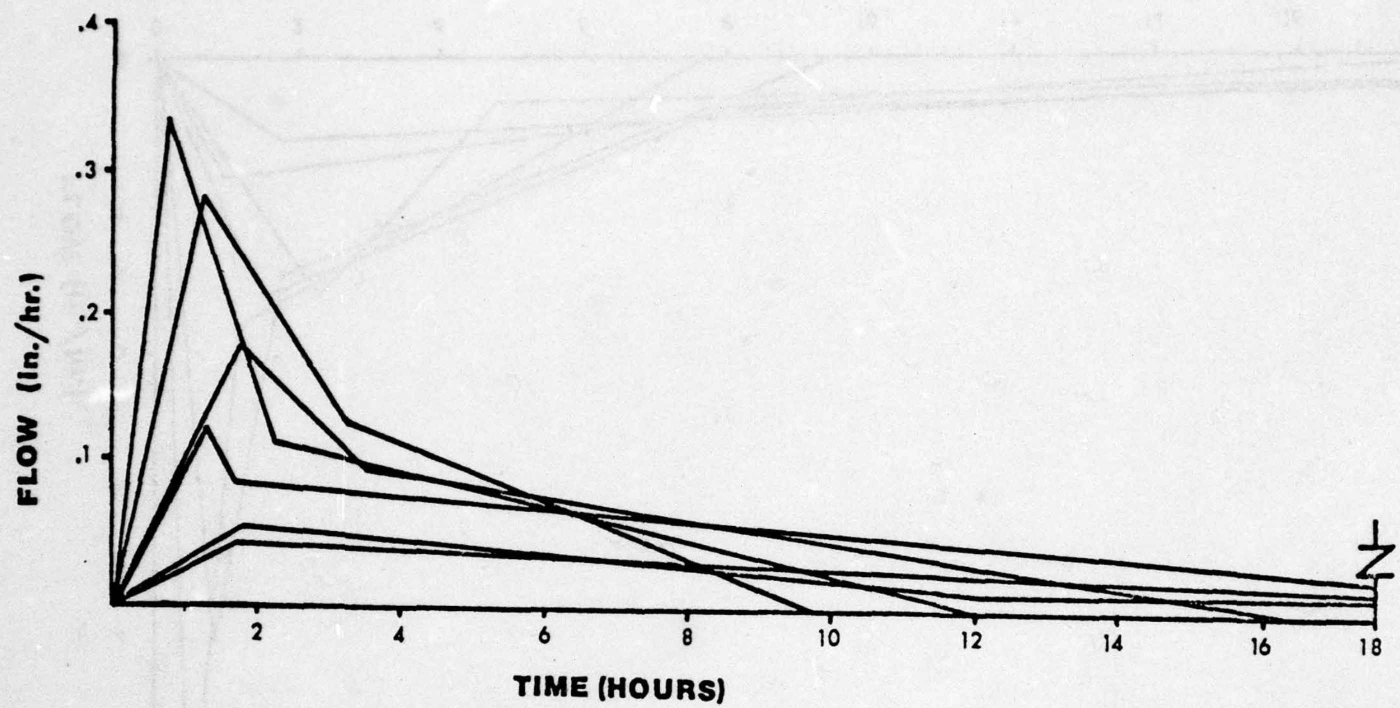

Figure 28 . Optimized Unit Response Functions for Lowe Branch Storms (DT = 0.25 hr).

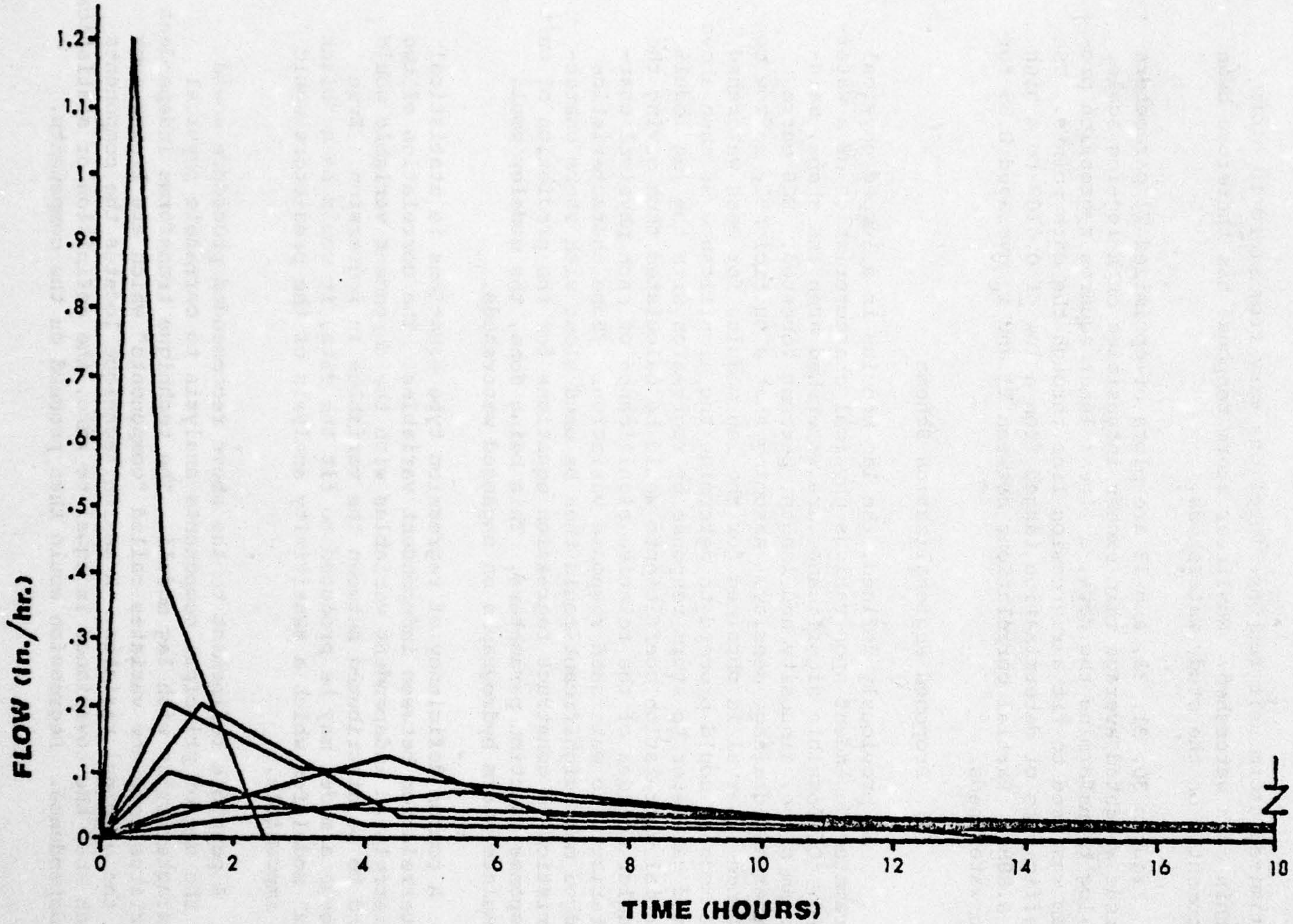

Figure 29. Optimized Unit Response Functions for Anderson Branch Storms (DT = 0.25 hr)

optimized storm unit response functions vary from storm to storm within each watershed. Nonlinear storm response has therefore been documented on the study watersheds.

Figures 30, 31, 32, and 33 are plots of optimized T1 parameters versus weighted average rain excess intensities on a log-log scale. By log transforming the data, a linear least squares regression program was used to fit a regression line through the data points. The coefficients of determination ranged from a low of 0.6109 to a high of 0.8881. Partial correlations between T1 and $\bar{i}_e$ averaged 0.85 for the watersheds.

(4) Proposed Regionalization Scheme

As previously defined, the lag modulus is a lumped physical parameter dependent upon various physical characteristics of a watershed. Of possible significance are watershed area and shape, mainstream slope, sinuosity and length, percent forested, SCS curve number and drainage density. Assuming that a sufficiently narrow confidence interval is obtained for the lag modulus for each watershed, the modeler could proceed to determine the significance of each above named parameter to storm response by regression with the lag moduli. Partial correlation coefficients would be calculated thus giving the modeler an idea of the relative significance of each physical characteristic to watershed response variation. Those characteristics judged most significant could then be used along with storm characteristics to construct regression equations for the prediction of unit response function parameters. This being done, the modeler could simulate storm hydrographs on ungauged watersheds.

A common deficiency of regression type equations is statistical interrelation between independent variables. The correlation of two interrelated independent variables with the dependent variable would tend to be distributed between the variables in regression. Even though a model may be produced to fit the data, it would be a "black box" model for which a sensitivity analysis of the predictors would be impossible.

A possible refinement to the above recommended procedure would be the use of principal components analysis to correlate physical characteristics with lag moduli. The technique transforms independent variates into new variates called "components" which are linear sums of the original variates. A search technique locates the components such that the covariance is equal to zero, the definition of statistical independence. Regression would then proceed on the components.

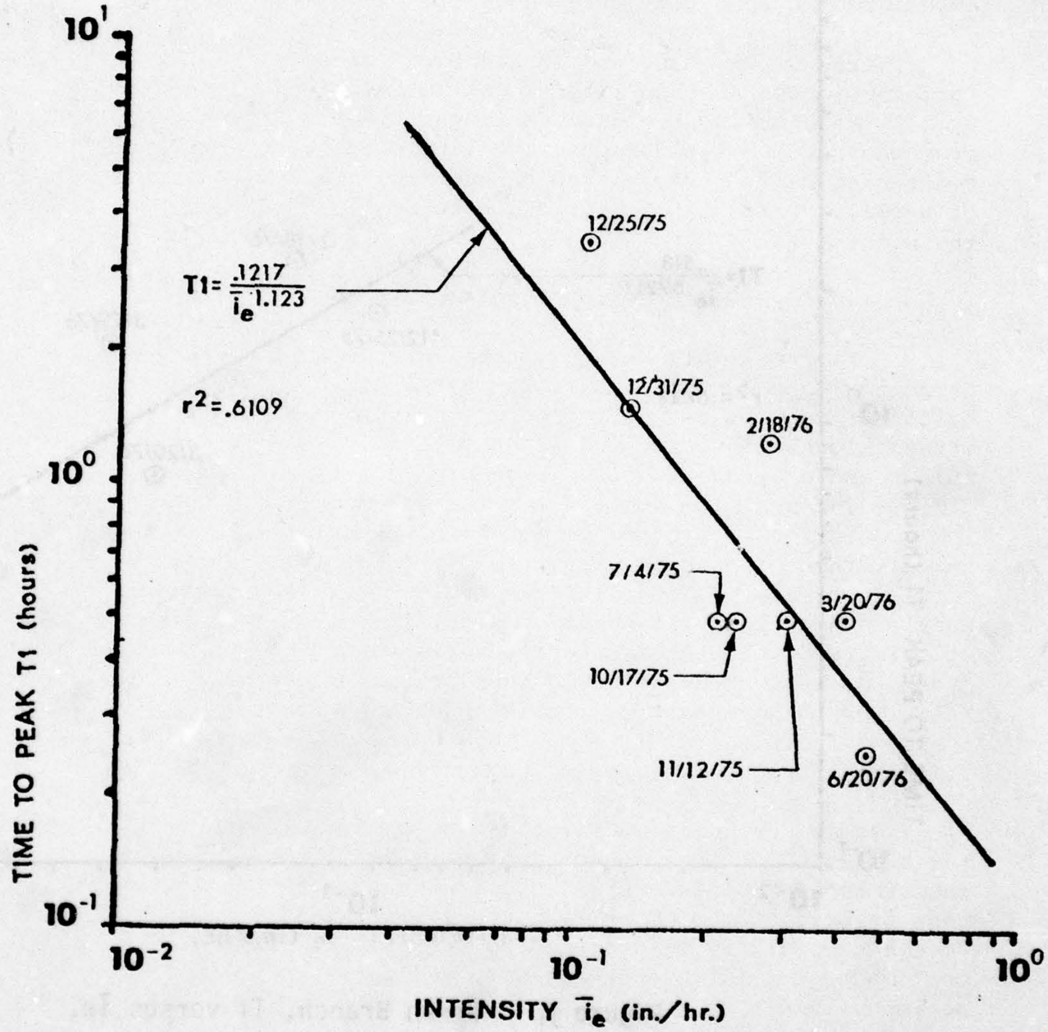

Figure 30. Bill's Branch, T1 versus $\bar{i}e$.

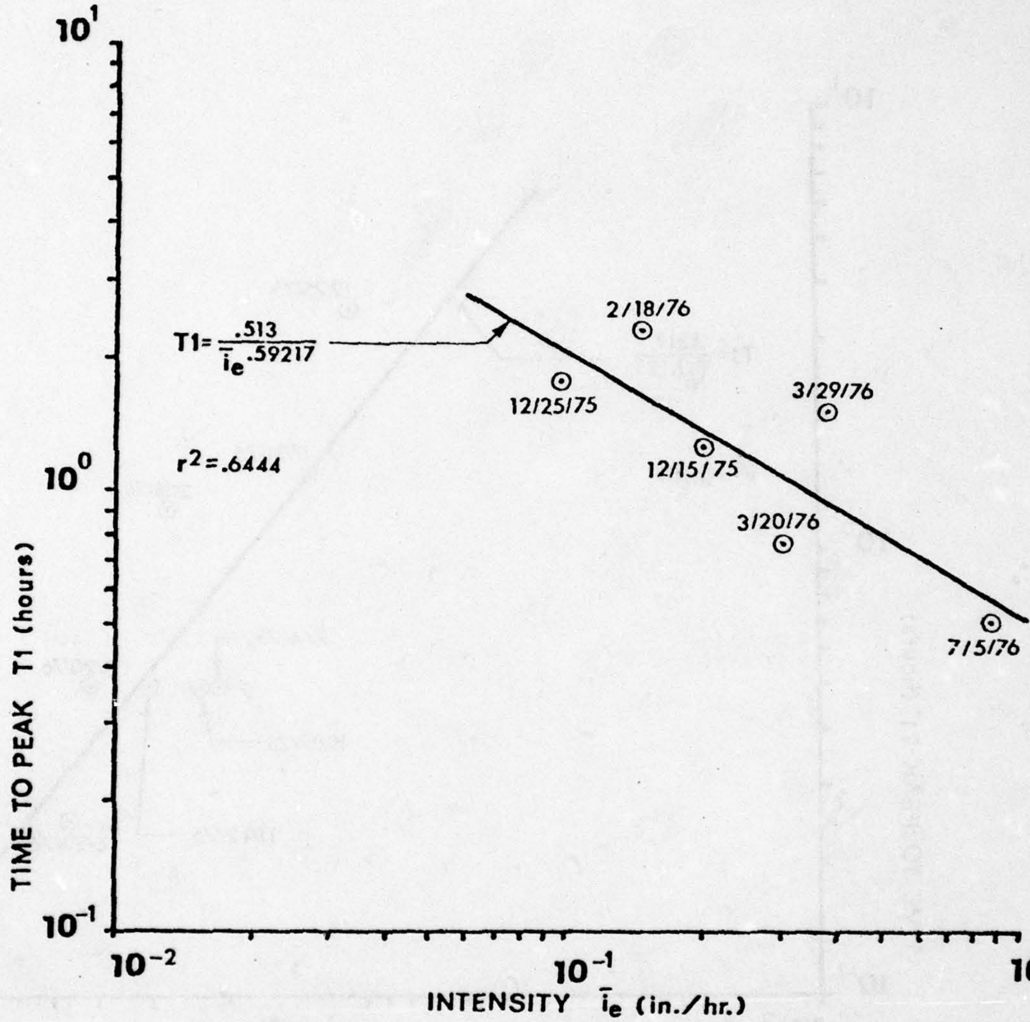

Figure 31. Green Branch, T1 versus $\bar{i}e$.

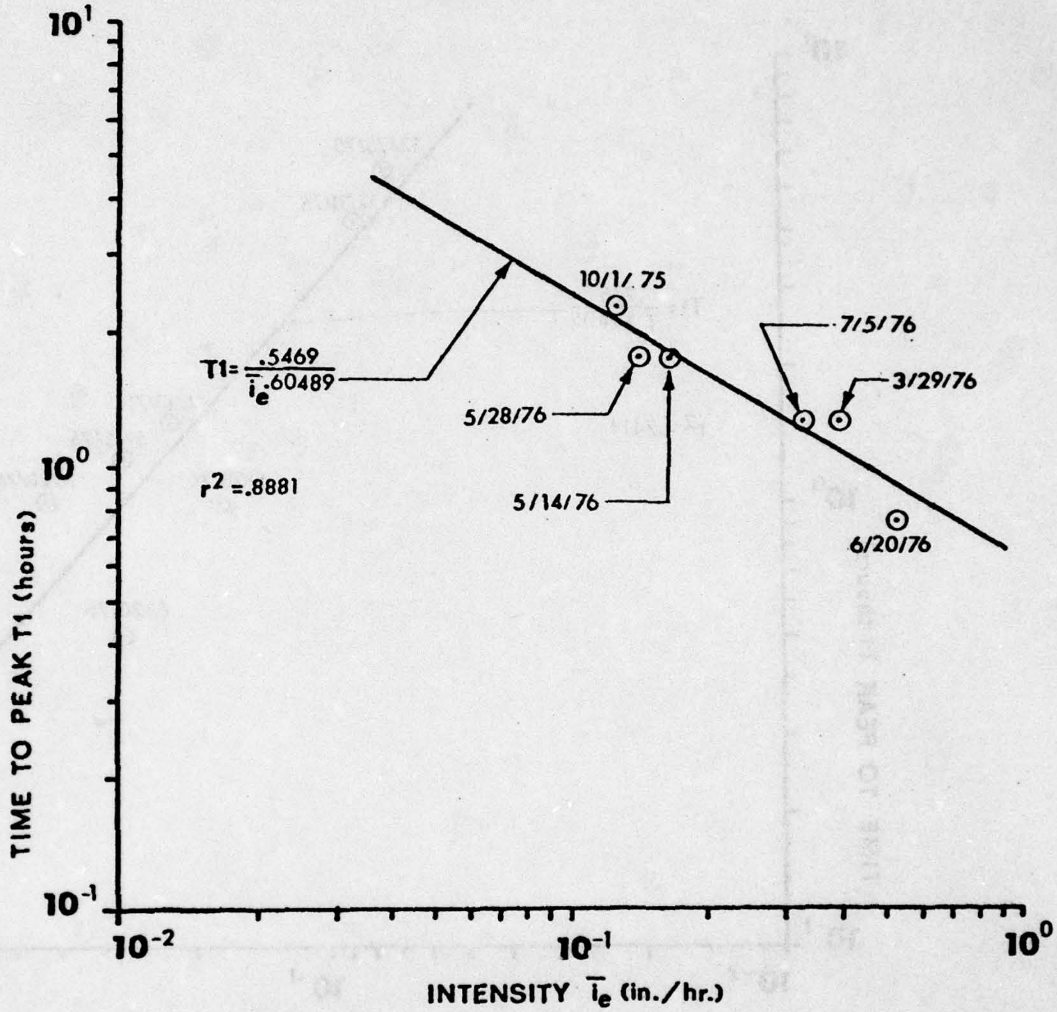

Figure 32 Lowe Branch, T1 versus $\bar{i}$e.

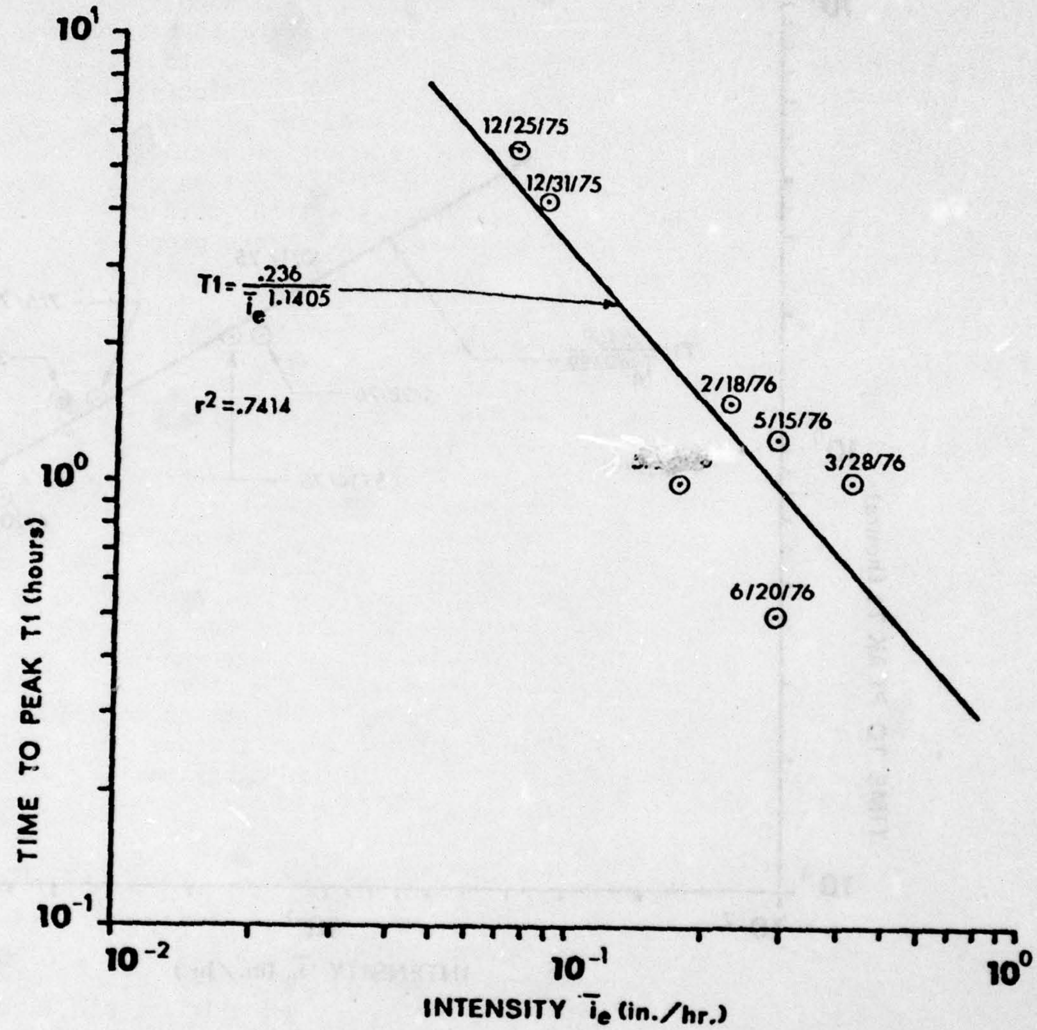

Figure 33. Anderson Branch, T1 versus Ie.

For illustrative purposes, assume that a component is 85 percent explained by drainage area and 15 percent by watershed shape. Further assume that another component is 75 percent explained by drainage density and 25 percent by sinuosity. The modeler could choose to use only drainage area and drainage density in the predictive equations for unit response function parameters. Accuracy of prediction would be sacrificed to a certain degree but the modeler would gain the capability of testing the sensitivity of individual physical characteristics to watershed response. This approach would also permit the drawing of an inference as to what the drainage process is.

7. REGIONALIZATION OF STORMWATER QUALITY

a. Land Use and Activity

What should be apparent from the analyses thus far presented is that stormwater discharge response is predictable in terms of storm characteristics, land use, soils, and watershed geometry, but stormwater quality is not only a function of these factors but is also a function of the activity within the watershed. Activity factors include automobile traffic, construction, aircraft sorties, industrial operation, etc., feedlots and strip mining. Hence, stormwater quality must be site specific to a much greater extent than stormwater runoff response. Therefore, stormwater quality response must be calibrated with site activity, and regionalization of stormwater quality is apparently an infeasible task. However, an approach toward regionalization is presented herein and an example model simulation will be presented including a procedure for transferring the results to an Air Force Base.

b. Transport Density

An approach is presented here which related pollutograph response to hydrograph response. The approach results in a linear relation between pollutograph response and hydrograph response. This leads to the derivation of a transport density, a load modulus, and delivery modulus. These moduli are indicators of the pollutant delivery response characteristics of a watershed under a given land use and activity.

The results of Colston's study (Reference 14) shown in Table 16, indicates that the pollutograph (lb/min) is linearly related to the hydrograph (cfs). This is consistent with the results reported by EPA (Reference 16), which indicates that storm pollutant load is proportional to storm runoff volume. Hence,

$$C = \delta Q \tag{34}$$

TABLE 16. EQUATIONS DESCRIBING URBAN RUNOFF POLLUTANT FLUX (lb/min) FOR DURHAM, NORTH CAROLINA AS A FUNCTION OF DISCHARGE RATE (cfs) AND TIME FROM STORM START (TFSS) (hr)[a]

Equation	r^2
COD = 0.51 $CFS^{1.11}$ $TFSS^{-0.28}$	0.90
TOC = 0.16 $CFS^{1.0}$ $TFSS^{-0.28}$	0.84
Total solids = 3.35 $CFS^{1.14}$ $TFSS^{-0.18}$	0.85
Volatile solids = 0.58 $CFS^{1.09}$ $TFSS^{-0.11}$	0.92
Suspended solids = 1.89 $CFS^{1.23}$ $TFSS^{-0.16}$	0.76
Volatile suspended solids = 0.25 $CFS^{1.18}$ $TFSS^{-0.17}$	0.83
Kjeldahl nitrogen = 0.0032 $CFS^{0.87}$ $TFSS^{-0.29}$	0.73
Total phosphorus as P = 0.003 $CFS^{1.03}$ $TFSS^{-0.29}$	0.92
Aluminum = 0.0443 $CFS^{1.05}$ $TFSS^{-0.15}$	0.89
Calcium = 0.045 $CFS^{0.60}$ $TFSS^{-0.09}$	0.82
Cobalt = 0.0003 $CFS^{1.18}$ $TFSS^{+0.13}$	0.92
Chromium = 0.0008 $CFS^{0.96}$ $TFSS^{+0.06}$	0.89
Copper = 0.00035 $CFS^{1.10}$ $TFSS^{+0.08}$	0.94
Iron = 0.0238 $CFS^{1.24}$ $TFSS^{-0.18}$	0.87
Lead = 0.0013 $CFS^{1.125}$ $TFSS^{-0.29}$	0.83
Magnesium = 0.0434 $CFS^{0.98}$ $TFSS^{-0.16}$	0.94
Manganese = 0.0023 $CFS^{1.11}$ $TFSS^{-0.27}$	0.94
Nickel = 0.0005 $CFS^{1.03}$ $TFSS^{+0.01}$	0.94
Zinc = 0.0011 $CFS^{1.10}$ $TFSS^{-0.22}$	0.89

a Reference 14

Since C is in lb/min and Q is in units of cfs, it follows that δ is in units of lb-sec/ft^3-min and is defined herein as the transport modulus and represents the pounds of pollutant delivered to the watershed outlet per cubic foot of runoff volume.

c. Load Modulus

The load (lb) associated with an instantaneous response function is

$$L(U) = \int_0^\infty C(U)dt = \int_0^\infty U\ o,t\ dt \qquad (35)$$

where

L(U) is the unit load in lb

C(U) is the instantaneous pollutograph in lb/sec

δ is the transport density in lb/ft^3, and
U o,t is an instantaneous response function

The load per acre associated with the instantaneous response function is defined herein as the load modulus, ϕ and is expressed as

$$\phi = \frac{L(U)}{DA} = 3630\ \delta \qquad (36)$$

where ϕ is in lb/ac and δ is in lb/ft^3. Hence, load modulus and transport density are related by a constant factor.

d. Illustrative Example

A comparison of the load modulus for Colston's urban (1.87 mi^2) basin and two of the stripped mine basins is shown herein. Colston's data was reanalyzed and the derived transport density and the associated statistical parameters were:

(Urban) $\delta = 0.00127$ (lb/ft^3)

$$R^2 = 0.855 \qquad (37)$$

On Bill's Branch the same analysis produced

(Bill's) $\delta = 0.00097$ (lb/ft^3)
10 percent $R^2 = 0.8$ (38)
stripped

(Green) δ = 0.00222 (lb/ft^3)

24 percent stripped $R^2 = 0.892$ (39a)

Comparison of the three basins indicated that a unit storm would produce the following load moduli for iron:

DA = 1200 ac. ϕ (Colston) = 4.61 lb/ac

DA = 429 ac ϕ (Bill's) = 3.51 lb/ac

DA = 883 ac ϕ (Green's) = 8.05 lb/ac (39b)

This analysis can be extended to any pollutant of interest, and comparisons may be made for any two basins regardless of size. It can be seen from Equation (39) that iron production per acre for Green's Branch (24 percent stripped) is more than double that of Bill's Branch (10 percent stripped). Further, construction in Colston's urban watershed development indicates that it is generating more iron per unit.

SECTION III

EXAMPLE MODEL SIMULATIONS OF STORMWATER RESPONSE

Given: A one square mile (640 acre) rural watershed. The basin is to be fully developed as urban with a combination of single and multifamily residences and commercial land use.

Problem: Evaluate the impact of development by simulating the stormwater hydrograph, pollutograph and total storm load for suspended sediment. Also, compare load moduli for suspended sediment and determine the degree of removal for suspended sediment associated with both the existing and proposed land use and activity scenario.

Assumptions: It is assumed that all required parameters have been regionalized and are predictable to an acceptable level of reliability.

Solution:

(1) **Before Development**

Parameters: $CN = 65$

$\mu = 40$ minutes

$\delta = 0.00002\ \text{lb/ft}^3$

$M_T/DA = 350$ lb/ad

$RO_c = 0.10$ in

Runoff Volume from SCS Curve Numbers (SRO)

$$S = \frac{1000}{65} - 10 = 5.38 \text{ in}$$

$$SRO = (1.2 - 1.07)^2/(1.2 + 0.8) = 0.0085 \text{ in}$$

Average Rain Excess Intensity - i_e

$$i_e = 0.0085/0.50 = 0.017 \text{ in/hr}$$

Time of Concentration - $t_c = 1.6t_L$ (lag)

$$t_c = 40/(0.017)^{0.4} = 204 \text{ minutes}$$

TVA - SWM - Parameters

$T1 = 2.8$ hr

$UP = 0.28$ in/hr

$UR = 0.05$ in/hr

$T2 = 5.6$ hr

$Q_P = 0.28 \text{ (in/hr)} * 640 \text{ ac} * 0.0085 = \underline{1.52 \text{ cfs}}$

Suspended Load

$L/DA = 2 \times 10^{-5} \times 43,560 \times (0.0085/12) = 0.0006$ lb/ac

or $L = 0.0006 \times 640$ ac $= 0.394$ lb

The load modulus is

$\phi = 3630\ \delta = 0.073$ lb/ac

Percentage Removal (PR)

$PR = 1 - \exp(-0.0085/0.10) = 0.081$

This corresponds to $L/M_T = 1.7 \times 10^{-6}$

The result of these computations indicates good agreement between the pollutograph simulation and the storm volume removal simulation. Further, this indicates that the 2 year storm removes less than one percent of the potential sediment load. However, overall the peak flow and the load modulus are very low.

By comparison, the peak flow before development was only 1.52 cfs--hence, insignificant. The storm hydrograph and associated pollutograph are shown in Figure 34.

Suspended Load

$L/DA = 1.2 \times 43,560 \times (0.48/12) = 2090$ lb/ac

or $L = 2090 \times 640 = 1.33 \times 10^6$ lb

The load modulus is

$\phi = 3630\ \delta = 4,356$ lb/ac

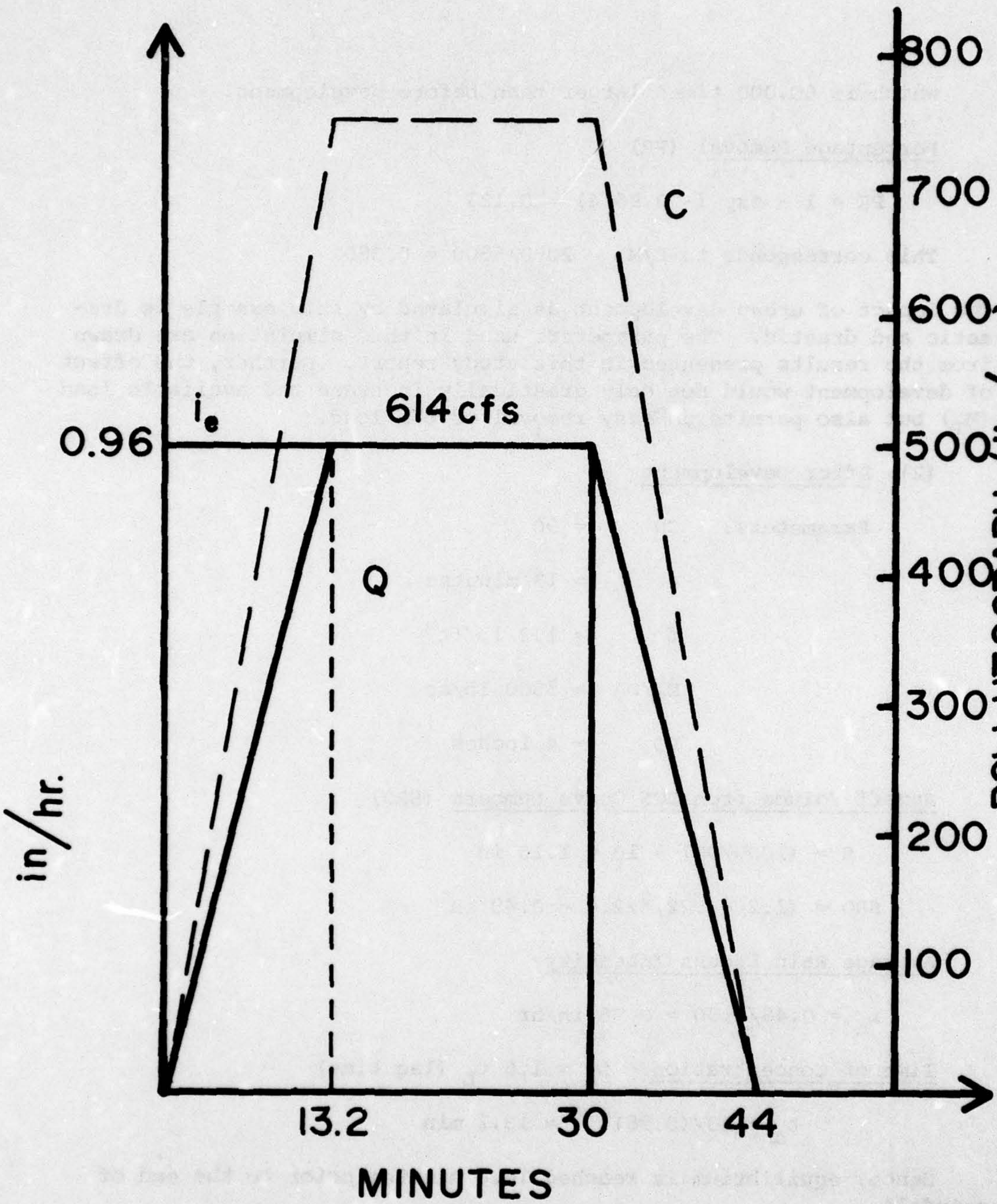

Figure 34. Stormwater Hydrograph and Pollutograph With Development

which is 60,000 times larger than before development.

Percentage Removal (PR)

$$PR = 1 - \exp(-0.96/4) = 0.123$$

This corresponds to $L/M_T = 2090/5500 = 0.380$

The impact of urban development as simulated by this example is dramatic and drastic. The parameters used in this simulation are drawn from the results presented in this study report. Further, the effect of development would not only drastically increase the available load (M_T) but also permits an easy removal of the load.

(2) After Development

Parameters:	CN	= 90
	μ	= 13 minutes
	δ	= 1.2 lb/ft^3
	M_T/DA	= 5500 lb/ac
	RO	= 4 inches

Runoff Volume from SCS Curve Numbers (SRO)

$$S = (1000/90) - 10 = 1.10 \text{ in}$$

$$SRO = (1.2 - .22)^2/2.0 = 0.48 \text{ in}$$

Average Rain Excess Intensity

$$i_e = 0.48/0.50 = 0.96 \text{ in/hr}$$

Time of Concentration - $t_c = 1.6\ t_L$ (lag time)

$$t_c = 13/(0.96)^{0.4} = 13.2 \text{ min}$$

Hence, equilibrium is reached 16.8 minutes prior to the end of rainfall.

TVA-SWM-Parameters (DT = 13.2)

T1 = 13.2 min

UP = 4.54 in/hr

UR = 0.10 in/hr

T2 = 26 min

The peak flow is Q_p = 0.96 x 640 = 614 cfs

SECTION IV

CONCLUSIONS ON THE TRANSFERABILITY OF MODELS TO AIR FORCE BASES

The following calibration procedure is proposed for the purpose of transferring the model development presented herein to a US Air Force Base.

1. The lag modulus for the Air Force Base should be derived from stormwater data monitored at the base. Since there will be a high degree of imperviousness associated with Air Force bases, lag modulus can be simulated by the technique reported by Overton and Meadows (Reference 3).

2. Transport density, δ, is to be derived from the correlation of pollutographs with the associated hydrographs. Delta is to be derived experimentally for each pollutant of interest.

3. From the transport density, load modulus can be derived simply by multiplying it by 3630.

4. M_T and RO_c need to be determined by optimization operating upon total load and the associated runoff volume.

5. There is a need to delineate the relative pollutional sources. It should be determined to what extent dustfall (atmospheric) relative to ground generated sources contribute to stormwater pollution. Following the procedure detailed in the report will lead to such a determination. It is essential that the accumulation and removal process be specified.

6. Finally, a framework has been developed whereby watershed response (discharge and quality) can be compared notwithstanding size, land use or activity. Development of an Environment Impact Settlement should involve the relative comparison of stormwater response impace rather than single out any particular land use activity complex as a pollutor, flood generator or affector of water supply.

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