Publication No. 58

LONG-TERM PERFORMANCE OF STORMWATER DETENTION FACILITIES: A COMPARISION OF DESIGN METHODOLOGIES

by

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FLORIDA WATER RESOURCES RESEARCH CENTER

RESEARCH PROJECT TECHNICAL COMPLETION THESIS

ENGINEERING AND INDUSTRIAL EXPERIMENT STATION

PROJECT NUMBER 80-W31

THESIS SUBMITTED JUNE, 1981

The work upon which this thesis is based was supported in part by funds provided by the Water Research Program, Engineering and Industrial Experiment Station, University of Florida, Gainesville

ACKNOWLEDGEMENTS

Interaction is free; as such, I owe thanks to many individuals at Black Hall and the Center For Wetlands who have contributed to the completion of this thesis and to my career at the University of Florida. The diversity of professionals in the department has provided a constant challenge to maintain an awareness of the many, yet similar, facets of environmental engineering sciences. A large debt is acknowledged to Bob Dickinson who several times pulled me up when I was close to drowning while SWMMing. Thanks also go to Steve Nix for his help on S/T. The use of the computer resources at the Center For Wetlands, Black Hall and the Northeast Regional Data Center was invaluable. Thanks to Anelia Crawford for the drafted figures. The direction of this thesis is credited to Dr. J. P. Heaney; the stability of the content is credited to Dr. W. C. Huber; the influence of Dr. H. T. Odum is reflected in the holistic approach to the problem assessment and solutions. Their guidance and attention are greatly appreciated.

Certainly the greatest debt is owed to my wife, Karen. Her patience, sacrifice, programming and typing ability and overall good spirits in the face of adversity will be forever appreciated.

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LIST OF SYMBOLS USED

a, b	coefficients of runoff quality power equation
ai	percentage of flow passing through basin i
A	cross-sectional area of flow
c	effluent concentration
C	flow volume capture efficiency
cO	influent concentration
Cro	runoff conversion factor
CV	coefficient of variation
dr	individual rainfall event duration
Dr	mean rainfall event duration
Dro	mean runoff event duration
e	base of natural logarithm
H	height of settling zone
Hd	height at which discharge begins and ends
hp	height of particle entering settling zone
k	first-order reaction coefficient
I	percent of catchment area that is impervious
ir	individual rainfall event intensity
Ir	mean rainfall event intensity
Iro	mean runoff event intensity
kO	initial first-order reaction coefficient
ki	linear flow coeficient of basin i
L	length of flow element

n	turbulence coefficient
N	return period for design storm
Q	volumetric flow rate
Qc	constant discharge rate
QcTi/Vro	normalized discharge rate
R	pollutant removal efficiency
SA	surface area of settling zone
S/T	storage/treatment
t	elapsed time
td	detention time of system
Ti	mean interevent time
v	velocity of flow
V	volume of flow element
Vb	empty volume of basin
Vb/Vro	normalized basin volume
Ve	effective volume
Vi	volume of basin i
vo	overflow rate (surface loading rate)
vp	particle settling velocity
Vr	mean rainfall event volume (per unit surface area)
Vro	mean runoff event volume (per unit surface area)
Vs	volume of settling zone
xo	percentage of particles with vp less than vo

Abstract of Thesis Presented to the Graduate Council of the University of Florida in Partial Fulfillment of the Requirements for the Degree of Master of Engineering

LONG-TERM PERFORMANCE OF STORMWATER DETENTION FACILITIES: A COMPARISON OF DESIGN METHODOLOGIES

by
Gary F. E. Goforth
June 1981

Chairman: James P. Heaney
Major Department: Environmental Engineering Sciences

A general overview of empirical, analytical, statistical and simulation techniques for evaluating stormwater detention systems is presented. The benefits and limitations of these methods in designing a control device for water quality improvement are emphasized. A detailed analysis compares continuous simulation utilizing the Environmental Protection Agency's Storm Water Management Model with the statistical techniques advanced by Hydroscience, Inc. The general dynamics of storage and flow

elements are discussed, emphasizing the importance of detention time in defining a time frame for evaluating systems.

Chairman

INTRODUCTION

GENERAL OVERVIEW

In the urban environment, combined and separate storm sewer overflows contribute the same amount of contaminants to receiving waters as do secondary treatment effluents (Heaney, et al. 1975, C.E.Q. 1978). Presently, simple storage/treatment devices, i.e. one or two component systems such as a stormwater detention basin, provide a cost-effective tool for quantity as well as quality control of these storm flows. While the design of these devices has traditionally been based upon a single storm event, the additional information provided by long-term analyses has recently encouraged their adaptation. The engineer or planner concerned with the design of a detention facility for the quality control of stormwater runoff has a variety of solution methodologies available: empirical approaches utilizing average annual values; analytical methods based on solutions to the flow governing equations; simulators amenable to rigorous search techniques; and statistical techniques involving rainfall-runoff parameter distributions. No one method, or coordinated coupling of methods, has been documented as the most cost-effective for

all applications. This is in part due to the lack of an available long-term data base, but also reflects the lack of comparative studies.

This thesis evaluates methodologies available for analyzing the long-term performance of stormwater runoff control devices. A detailed description of these methods is not the intent. The manuals referenced for each provide that service. Rather, the benefits and limitations of these methods in designing a control device for water quality improvement are emphasized. The comparison consists of three criteria:

1. problem assessment, i.e. how does the particular method define the system;
2. ease of application, e.g. data or computer requirements, or cumbersome techniques, and associated costs; and
3. accuracy of results, both absolute and relative to data requirements.

Because the long-term performance of detention facilities has not been well documented, there is no available data base to definitively compare the accuracy of the methods. Objectively, this study presents an opportunity to establish relative estimates of the long-term performance of storage/treatment devices designed for stormwater quality control. The optimal design of a control device will depend on problem specific constraints such as discharge quality standards and economic considerations.

SYSTEM DESCRIPTION

The following definition of a system is presented as a framework to maintain conceptual consistency.

> A system is any structure, device, scheme, or procedure, real or abstract, that interrelates in a given time reference, an input, cause, or stimulus, of matter, energy or information, and an output, effect, or response, of information, energy, or matter. (Dooge 1973, p. 4)

This functional interrelationship of inputs and outputs for a given time reference provides a basis for addressing water quality problems in a spectrum of hydrologic units, from urban stormwater systems to lakes threatened with cultural eutrophication.

Before evaluating the performance of a particular system, the hierarchy of systems which influence that performance must be recognized. A classical representation of the hydrologic cycle is presented in Figure 1. Storages and flows of water are the principal elements in the system, although the influences of solar energies, land morphologies and other factors are implicitly included. The system depicted in Figure 1 can be partitioned into discrete subsystems defined by characteristic storages and flows, as shown in Figure 2. These subsystems can be further subdivided into individual components, or control units, whose boundaries similarly reflect the storages and flows emphasized. The basic hydrologic characteristics which define the performance of a control unit, as depicted in Figure 3, are:

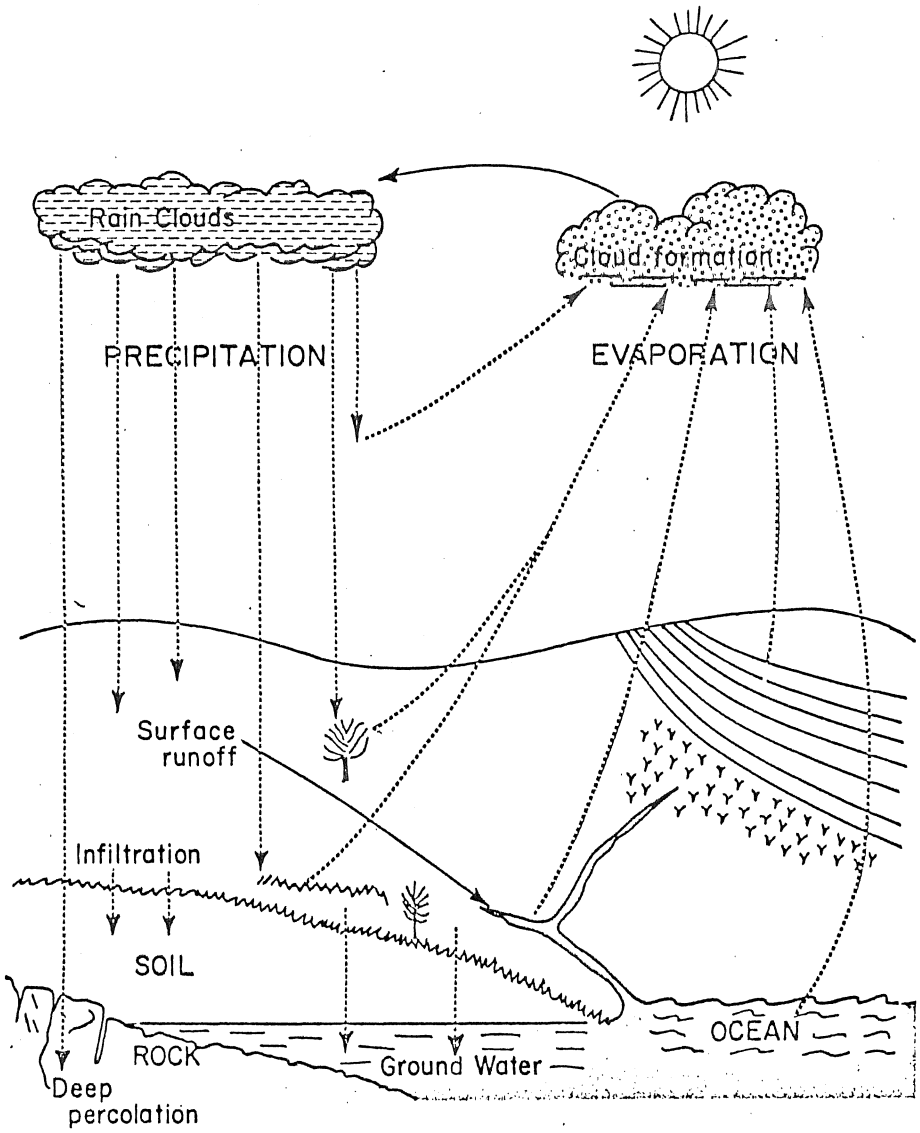


Figure 1. Representation of the hydraulic cycle.

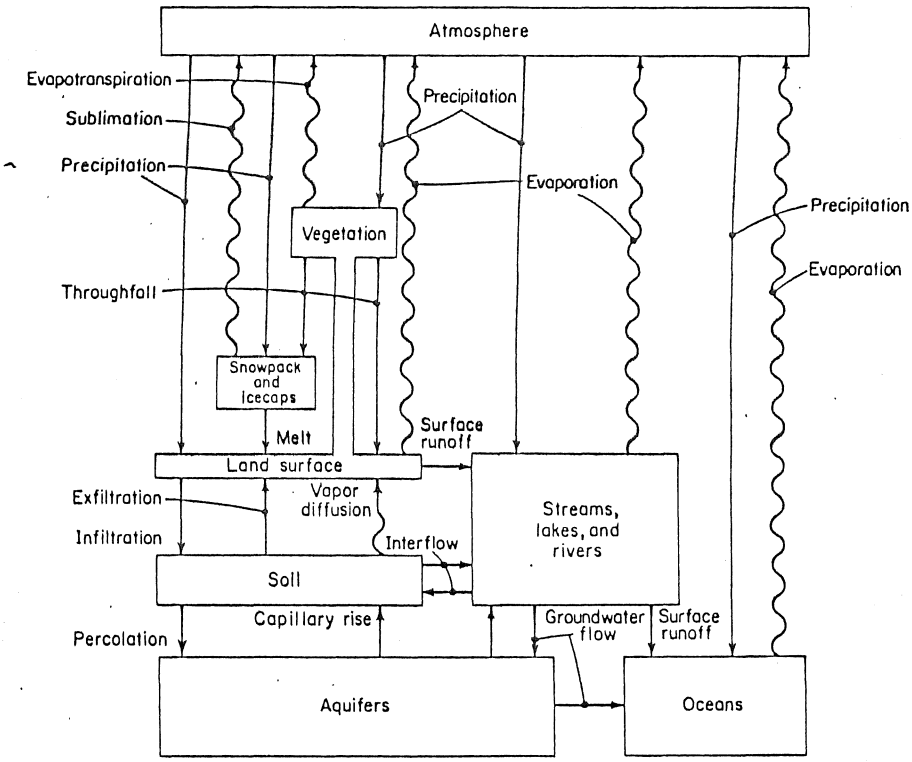


Figure 2. Discretized subsystems of the hydraulic cycle.

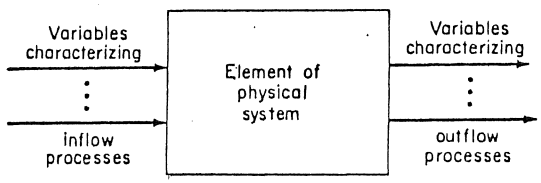


Figure 3. Processes defining the performance of a control unit.

1. The source of the mass, the unit's place in the system and its relation to other units;
2. the dynamic storage and flow conditions; and
3. the removal mechanism.

DEFINITION OF CONTROL UNIT

The following analysis centers on the long-term pollutant removal effectiveness of a hypothetical detention facility. The control unit is a single basin which receives the stormwater runoff from an urban catchment, and discharges to an undescribed receiving water. A schematic of the system is presented in Figure 4. The data source is a 24.6-year record of hourly rainfall values obtained from the National Weather Service. The system boundaries of the control unit are drawn at the inlet and outlet. As such, it is not just the rainfall which is the forcing function, but the runoff, a result of the rainfall's interaction with the catchment. The pollutant source is the constituent contaminants of the runoff - sand, debris, dust, etc. The removal mechanism responsible for pollutant control is sedimentation, and the removal characteristic is based on the treatment time in the basin. The removal kinetics are defined by the hydraulics within the basin, as determined by basin geometry and the inflow and discharge characteristics. These are the major influences on control efficiency and become the design parameters.

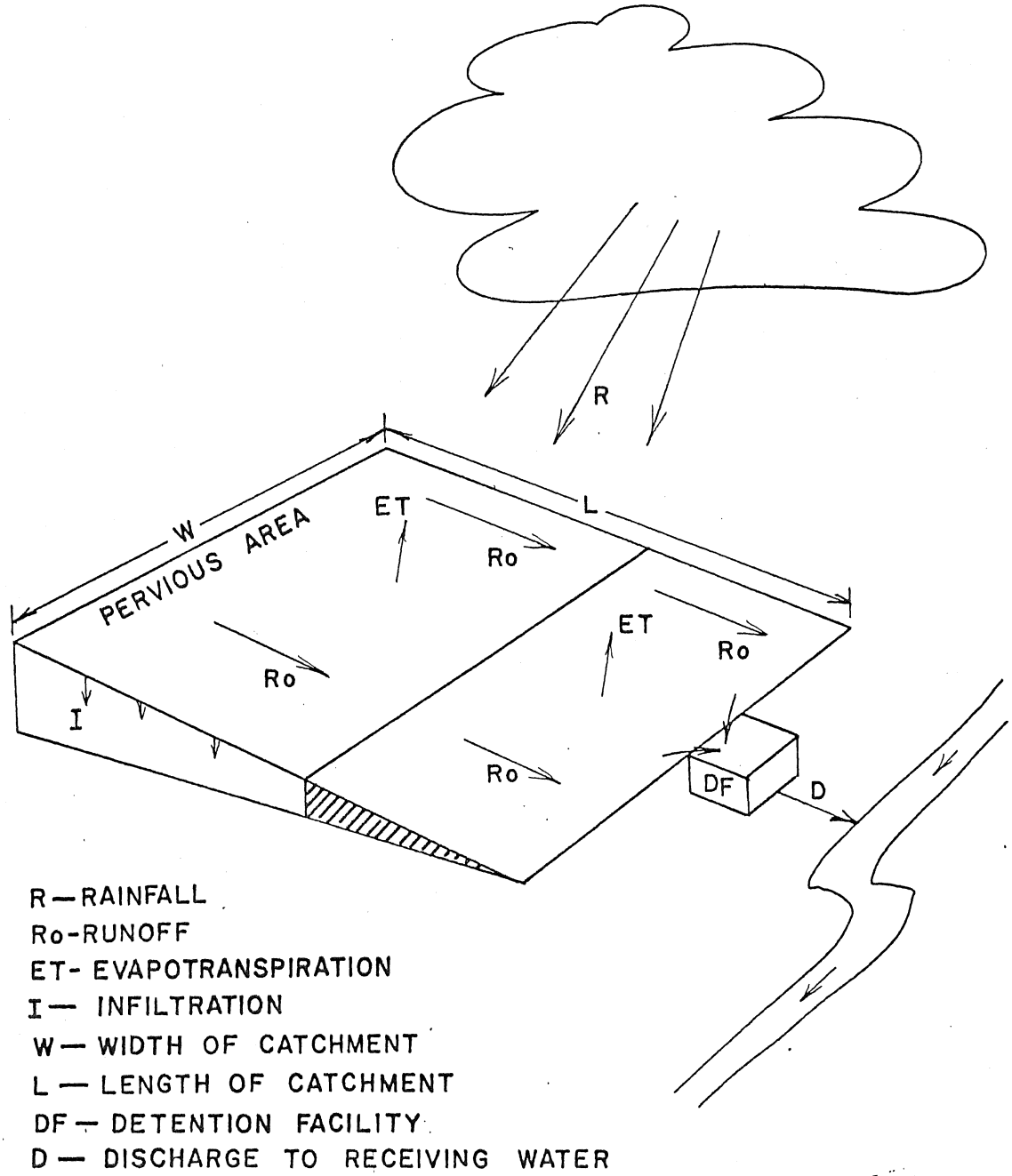


Figure 4. Schematic of Case Study catchment area.

CLASSIFICATION OF METHODS

Several methodologies are available for estimating the long-term performance of stormwater detention facilities. The approaches are all models of the same complex process, yet differ conceptually and mechanistically. Figure 5 is a schematic depicting the relationship of the various methodologies. For the purpose of this thesis, the following classifications will be used:

1. Analytical approaches utilize some combination of the general mass continuity equation and the advective-dispersion equation to describe the flows, storage, and pollutant removal characteristics of a control unit.
2. Empirical approaches are derived from or guided by experience. Although literally implying the lack of a theoretical background, the expression is used to denote methods which have been developed in scientific and engineering practice.
3. Two types of statistical techniques are widely used (Chow 1964). Frequency analysis methods are based on approximating the value of a random variable with a probability density function, from which frequencies of occurrence may be assigned. Regression and correlation analyses deal with the description of the relationship between two or more variables.

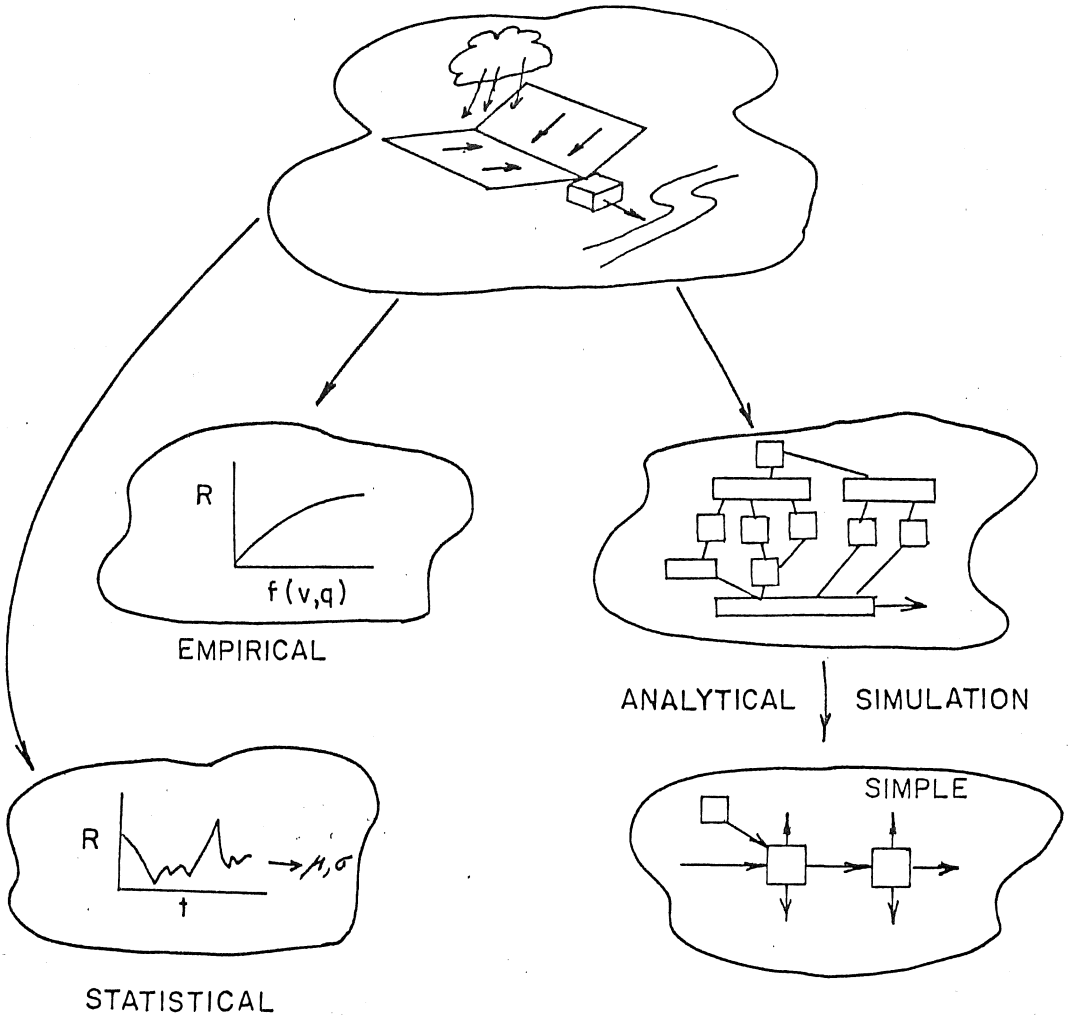


Figure 5. Characterization of the various methodologies.

4. Digital simulation methods were developed to exploit the ability of high-speed computers to manipulate mathematical expressions. The main objective of these methods is to deterministically model the dynamic processes in a physical system.

Rarely does a definitive demarcation exist between solution methodologies; there are overlaps and extensions from one to the next. As a model, each method represents simplifications, compromising between ease of application and accuracy. The empirical and statistical approaches provide first-cut approximations based on a small data requirement. The more complicated simulations are generally regarded as more accurate, although they may have extensive data or computational requirements.

CASE STUDY

CATCHMENT CHARACTERIZATION

User-supplied catchment data are input for most models. The extent of the data collection is dependent on the requirements of the particular method employed. As indicated in Figure 4, there are no streams, lakes or groundwater flows. For simplicity, there was no initial abstraction, areas of depression storage or other consumption of water. The flow routing geometry was kept as simple as possible. Conceptually, the catchment was a sloping plane with no gutter or pipe networks. All the runoff flowed directly to a dummy outlet on the downslope side. The runoff from the entire catchment was routed to the proposed basin, and was subsequently discharged to a local receiving water. The data were based on observed values for a drainage basin in Gainesville, Florida, and are presented in Table 1 (Huber, et al. 1981). In an actual catchment, waste characteristics would be obtained by running column settling tests on runoff samples.

Table 1. Case study catchment characteristics.

Total area = 24.7 acres
Impervious area = 37 percent
No depression storage or initial abstraction
Average catchment slope = 0.040 ft/ft = 211 ft/mile
Maximum infiltration = 2.5 in/hr
Minimum infiltration = 0.52 in/hr
Evaporation = 0.1 in/day
Population density = 500 people/square mile

RAINFALL-RUNOFF CHARACTERIZATION

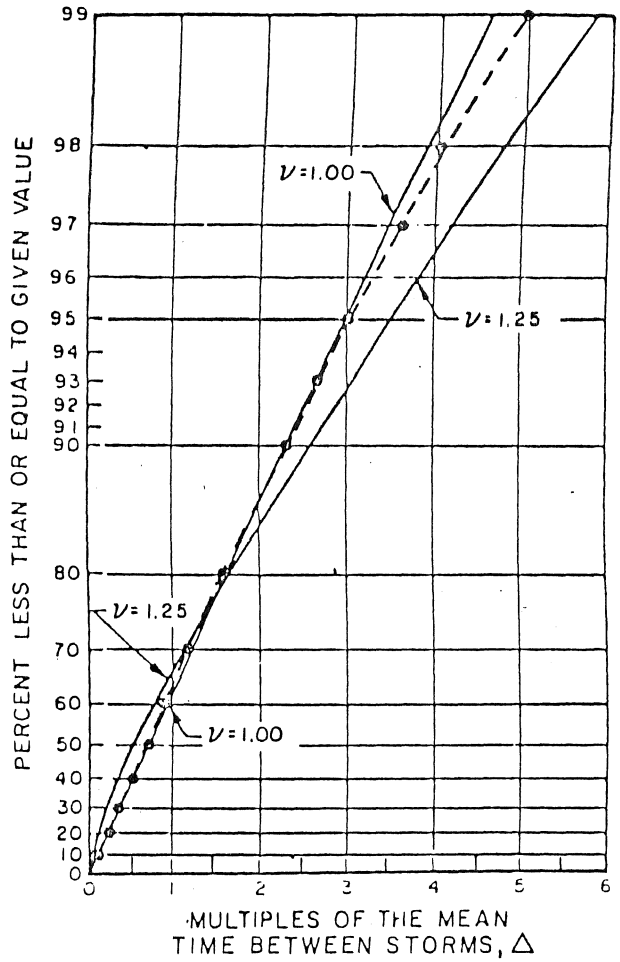
Rainfall Data

As the storages and flows of water are the principal elements in the catchment system, rainfall is the driving force. Long-term rainfall data are available on several time bases, e.g. continuous gages or discrete hourly, daily, monthly or yearly records. Rainfall data are characterized by volume (depth over the catchment area), average intensity, duration and time between events. The rainfall data source utilized for this study was the National Weather Service (NWS) tape for 24.6 years (June 1948 - December 1972) of hourly rainfall at Atlanta, Georgia. The standard NWS format is to record hourly rainfall values in hundredths of an inch on days when there is rain. Days without rain are not recorded on the tape. Hourly data for the first day of each month are recorded regardless of whether it rained or not.

SYNOP

For methods requiring average event statistics, hourly rainfall data may be analyzed with SYNOP, a computer package developed by Hydroscience, Inc. (1979) to determine synoptic statistics of data time series.

Rainfall volumes, intensities, durations and interevent times are the principal parameters evaluated in SYNOP. Available options include complete statistics on an event basis and time basis, e.g. yearly averages. Cumulative conditional probabilities (i.e., given that rain has occurred) and return periods for hourly magnitudes are also calculated, based on the California method of probability plotting. The grouping of hourly data into storm events is based on the minimum number of dry hours between rainfalls, an input variable termed the minimum intervent time. Assuming that the storm events occur as a Poisson process, the time between events is exponentially distributed. The exponential distribution is a special case of the gamma distribution with the coefficient of variation equal to unity. Figure 6 demonstrates the relationship between theoretical and observed results for the cumulative distribution of interevent times. The gamma distribution has been widely applied in hydrology (Haan 1977). To define events, the minimum interevent time is varied to obtain a value close to unity for the coefficient of variation (cv) associated with the interevent time. The SYNOP manual



LEGEND:
—— THEORETICAL GAMMA DISTRIBUTION
—•— OBSERVED DISTRIBUTION

NOTE
MINIMUM 6 DRY HOURS BETWEEN STORMS
(Δ = 84 HR, ν_δ = 1.02).

Figure 6. Comparison of theoretical and observed distributions of interevent times for Minneapolis/St. Paul airport.

recommends an initial trial of three hours for the minimum interevent time. SYNOP was run on the entire 24.6-year record of Atlanta rainfall to determine the storm statistics. The results of these runs, presented in Table 2 and Figures 7 and 8, give some idea of the sensitivity of the results to the choice of the minimum interevent time. The computer costs averaged $6.25 per run. With eight hours specified as the minimum number of dry hours defining an event, the coefficient of variation for the mean interevent time was 1.004. Values for the means of the parameters were taken from this run, e.g. the mean volume (Vr) of a rainfall event was 0.495 inches. Notice that Vr does not equal the product of Ir and Dr. This is because Vr is the mean of the products of the individual events' intensity (ir) and duration (dr), which is not necessarily equal to the product of the mean intensity (Ir) and the mean duration (Dr), i.e. Vr = mean (irdr) which is not the same as (mean ir)(mean dr) = IrDr.

Runoff Quantity

The interaction of rainfall and the catchment generates runoff. The quantity of runoff is determined by the influence of infiltration, evaporation, consumption patterns and land use (Eagleson 1970). The watershed system response to these interactions has been evaluated by hydrologists for many years. A comparison of rainfall and runoff time

Table 2. SYNOP results of 24.6 years of Atlanta, Georgia rainfall.

Minimum # of dry hours	Number	Volume Vr (in)	cv	Duration Dr (hr)	cv	Intensity Ir (in/hr)	cv	Interevent time Ti (hr)	cv
3	3215	0.367	1.540	4.642	1.126	0.078	1.372	66.73	1.269
5	2596	0.454	1.424	6.646	1.124	0.077	1.348	82.62	1.067
8	2381	0.495	1.384	7.824	1.134	0.077	1.356	90.10	1.004
12	2134	0.552	1.332	9.817	1.143	0.074	1.334	100.55	0.917

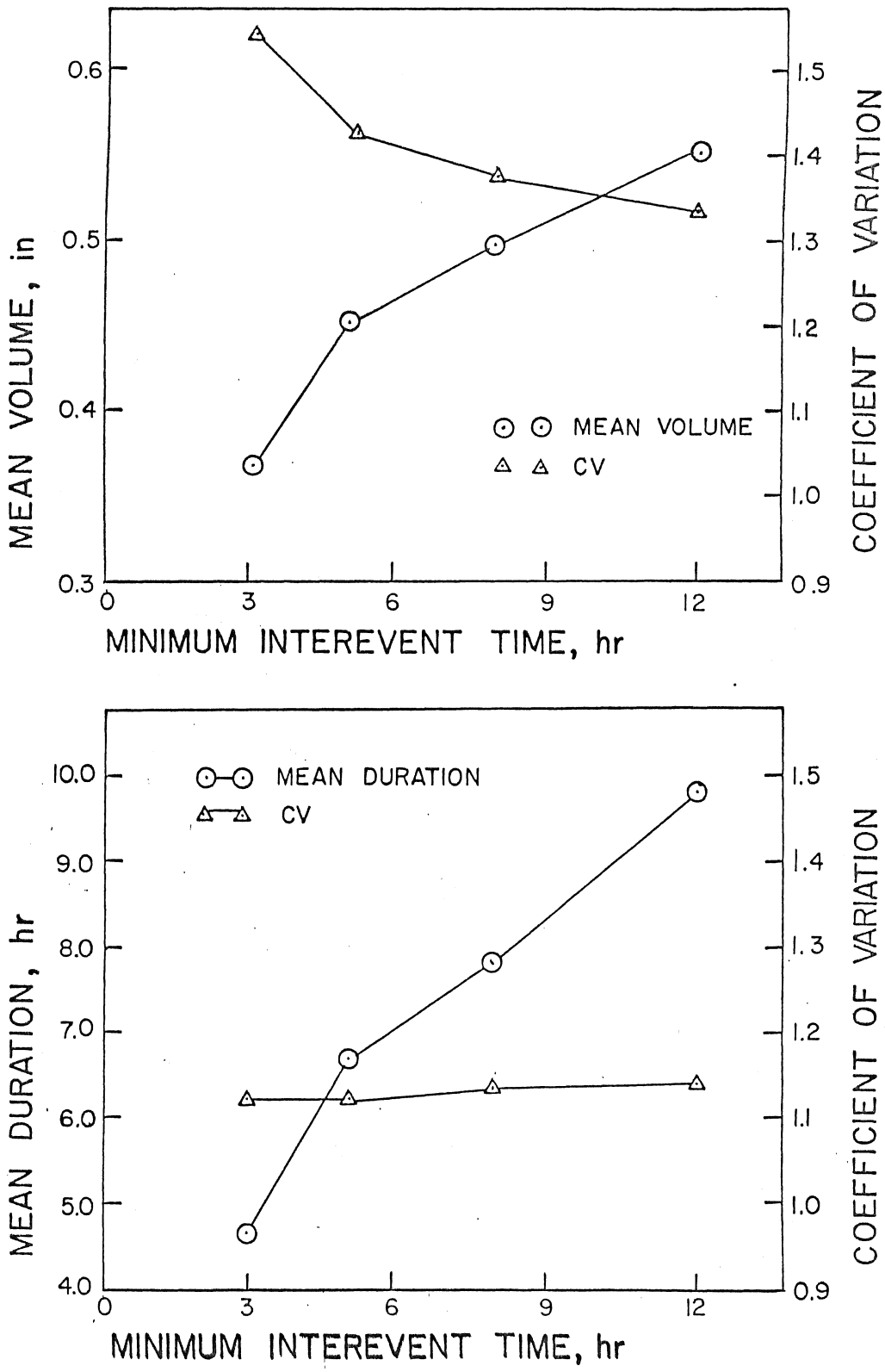


Figure 7. SYNOP values for mean event volume and duration as a function of minimum interevent time.

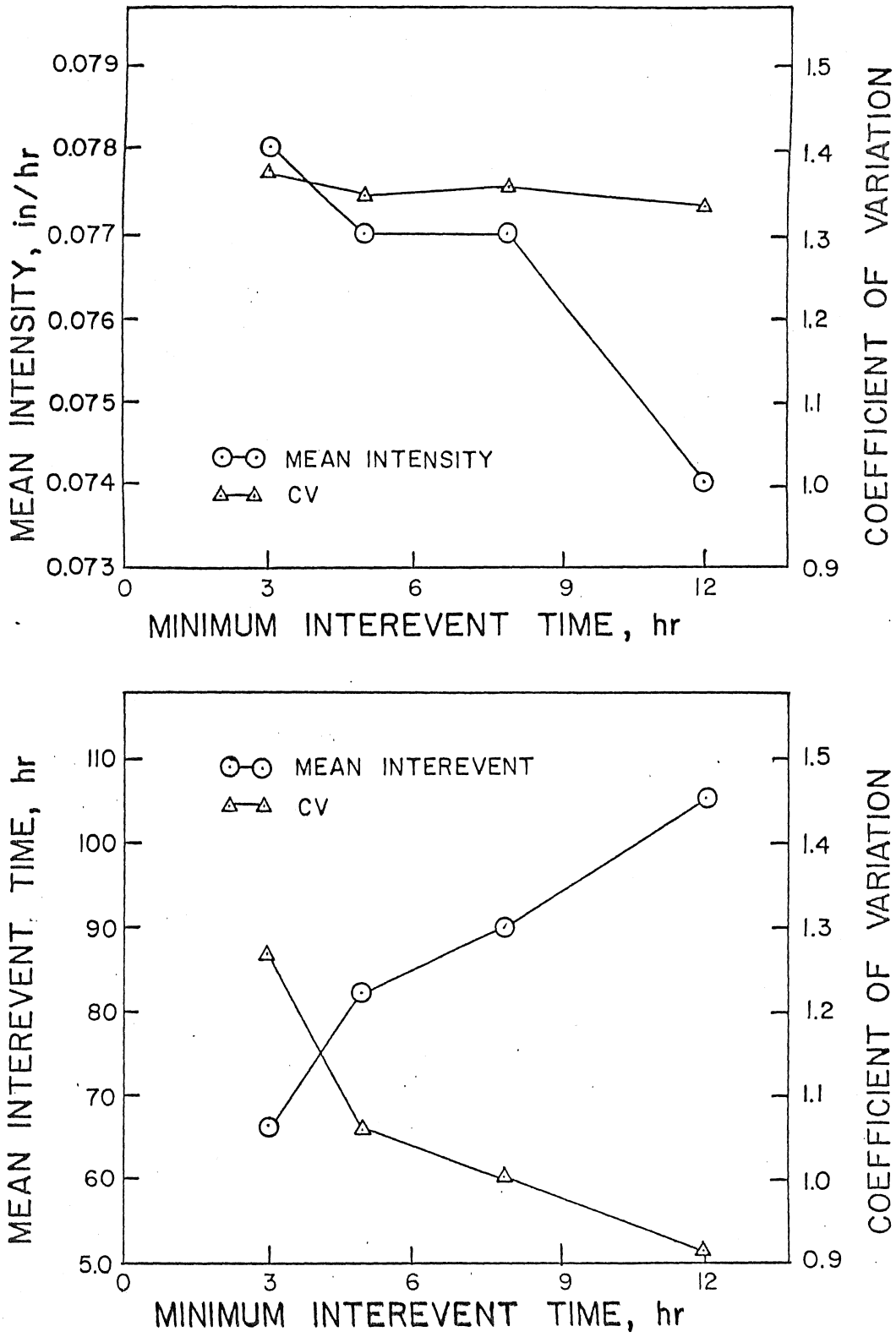


Figure 8. SYNOP values for mean event intensity and interevent time as a function of minimum interevent time.

series, presented in Figure 9, depicts two phenomena characteristic of the rainfall-runoff process:

1. a reduction in the number of events due to the capture of low volume storms by the indigenous catchment storage capacity, e.g. depression storage and soil moisture capacity; and,
2. a reduction in the volume of the events due to the catchment storage and flow interception, e.g. infiltration rates.

Analyses that deal with single runoff events are not sufficient to characterize these phenomena because the catchment storage and interception capacities are functions of antecedent soil moisture conditions, and are not constant. It becomes necessary to retain as much information as possible on the time between successive events. The event duration defines the reference time frame for differentiating between storage and interceptor elements. A storage element can detain up to a maximum runoff volume per event, i.e. its detention time is greater than the event duration. An interceptor, on the other hand, can capture up to a maximum flow rate before bypassing . Stormwater runoff control devices can also be characterized by storage and interception capacity, as presented in Figure 10.

A representation of a time series of runoff flows is presented in Figure 11, a series of flow pulses separated by

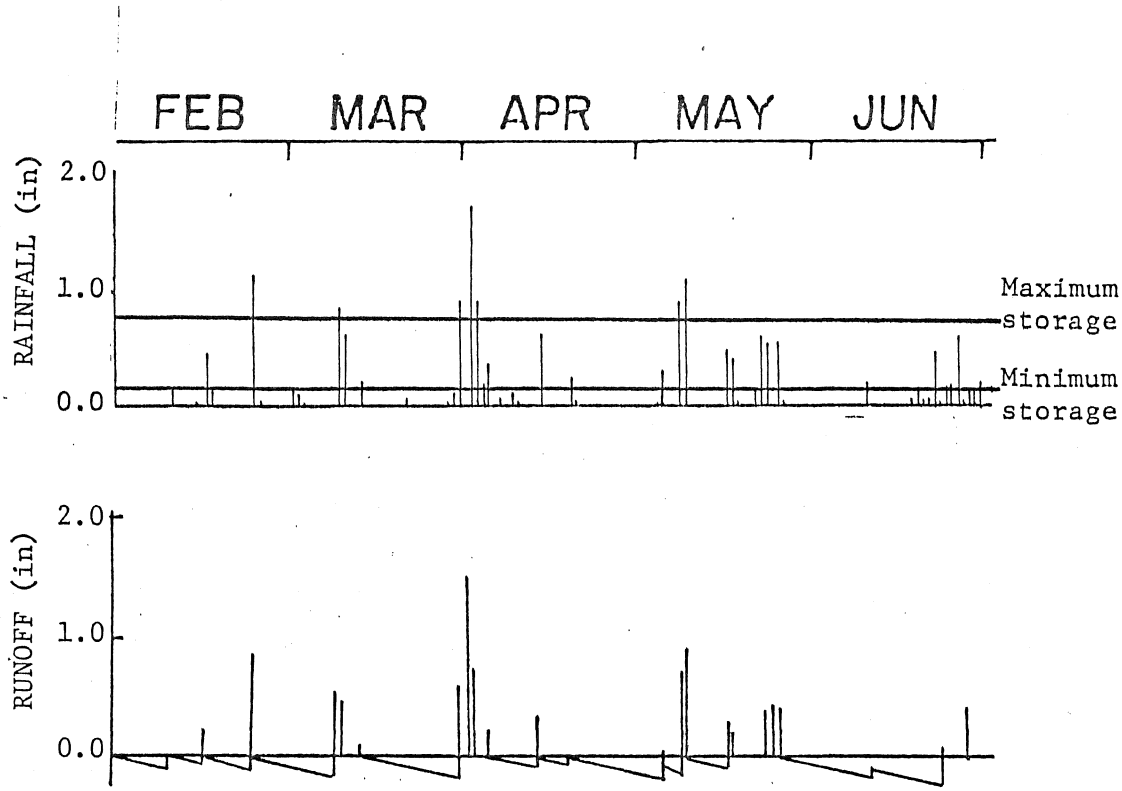


Figure 9. A comparison of rainfall and runoff time series depicting the reduction in number of events and the reduction in the event volume.

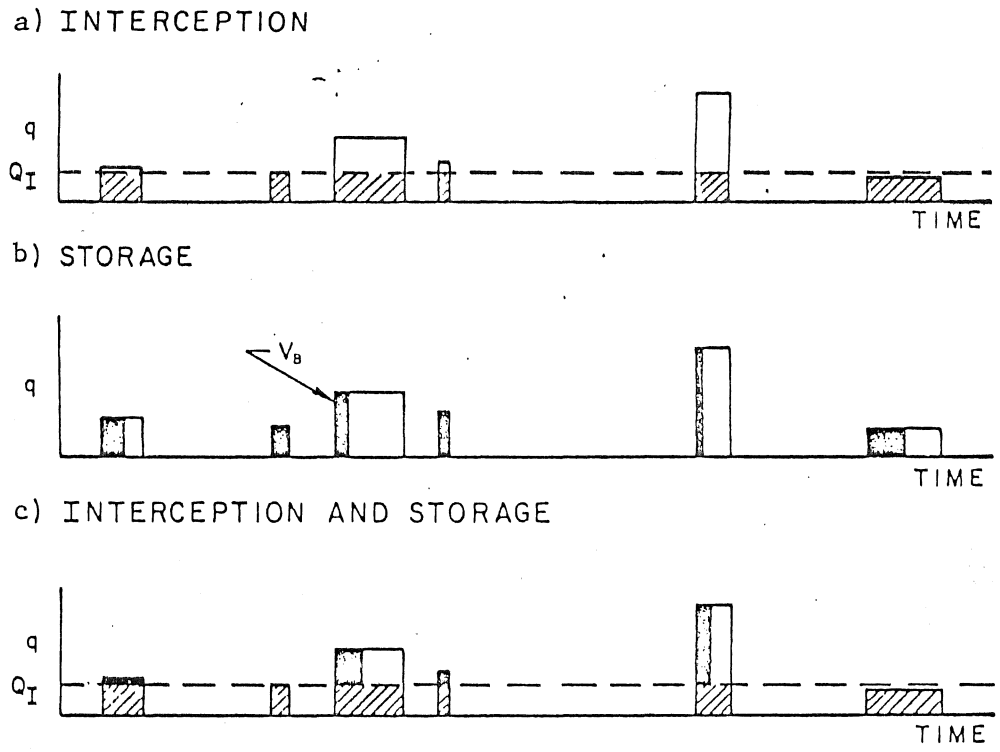


Figure 10. Definition of interception and storage for storm events.

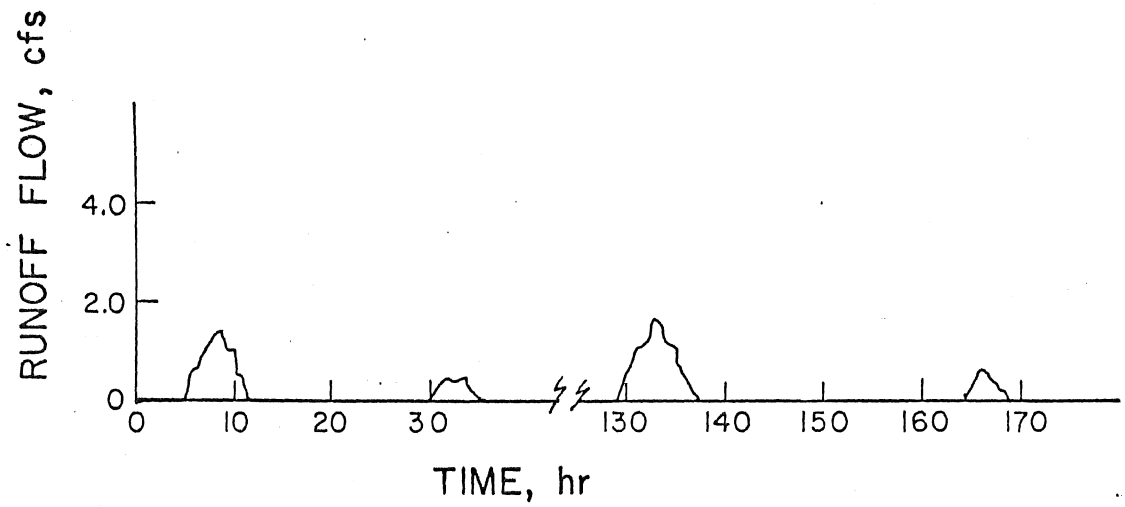


Figure 11. Representation of a time series of runoff flows.

relatively long peroiods of no flow. The determination of runoff characteristics is a major step in the solution process and is where the methods vary the most. The approaches compared here offer a sharp contrast in the representation of the rainfall-runoff process. The statistical and empirical approaches summarize runoff generation via linear conversion factors applied to rainfall statistics. On the other hand, the simulation technique utilizes some of the most refined concepts in deterministic hydrology.

BASIN CHARACTERIZATION

Flow conditions

The input to the basin is the runoff from the catchment area. The time series of runoff events depicted in Figure 11 suggests two realms of kinetics: rapid, relatively well-mixed during the runoff event, followed by slower (less dispersion, turbulence) kinetics and possibly quiescent conditions during dry weather. Characterizing this time series of discontinuous flows entering the basin is a major obstacle in solution methods.

Basin discharge may be either variable, as in the case of gravity drainage, or constant, via a pump or outlet restriction. Negative feedback is inherent in gravity systems, i.e. when the water level is high, the outflow is high, and as such, tends stabilize the flow. It is

difficult to deal with this nonlinearity analytically; it is far easier to analyze a constant discharge rate. There has been no evidence to suggest that one is better than the other for pollutant removal.

Removal

Theory of settling

Pollutant removal via settling is the most widely useful operation in water and wastewater treatment (Fair, Geyer and Okun 1968, Liptak 1974). In the design of sedimentation basins, basic assumptions are incorporated:

1. inlet zone - the influent is transformed to a uniform vertical distribution of particles.
2. settling zone - there is steady, uniform flow and quiescent, discrete and unhindered settling.
3. bottom zone - solids which enter the bottom zone are not resuspended.
4. outlet zone - solids that do not enter the bottom zone leave in the effluent.

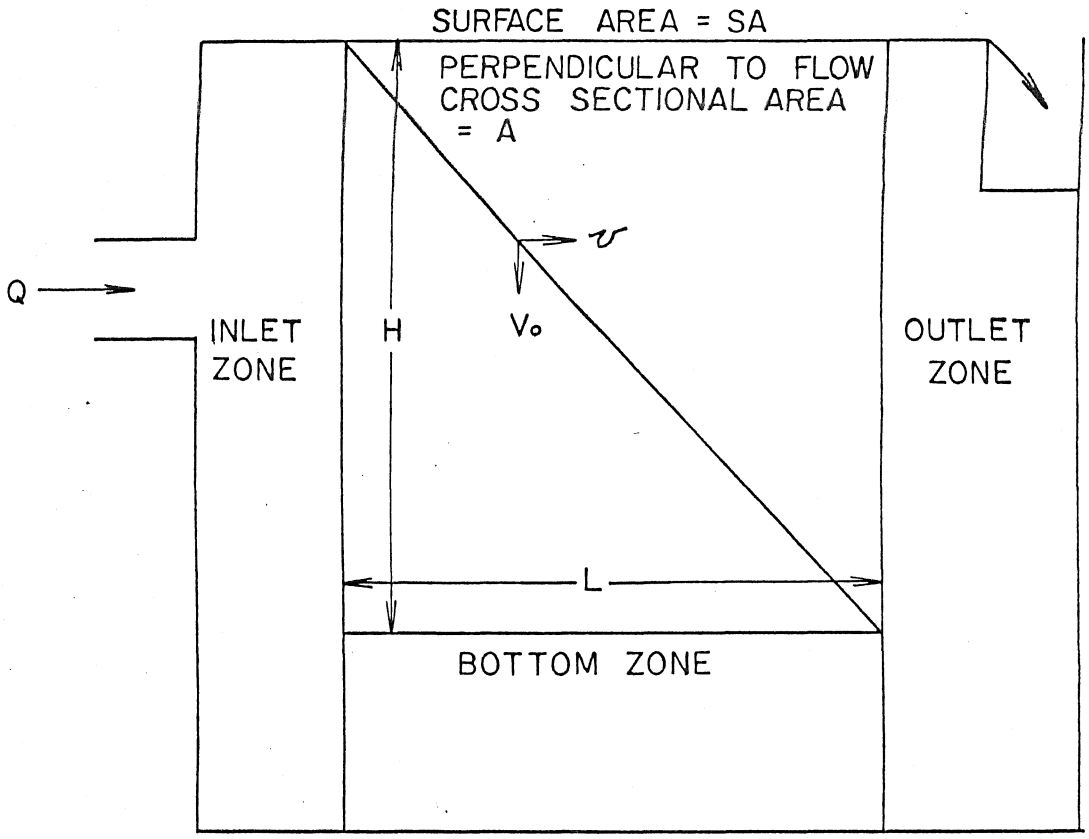
These four zones and particle settling paths are shown in Figure 12. The major design parameter is the overflow rate (vo), defined as

$$vo = H/td = (Vs/SA)/(Vs/Q) = Q/SA \qquad (1)$$

where H is the depth of the settling zone [L],

td is the detention time of the settling zone [T],

Vs is the volume of the settling zone [L3],



$$\text{TRAVEL TIME} = \frac{L}{v} = \frac{L}{Q/A} = \frac{L \cdot A}{Q} = \frac{V}{Q}$$

$$\text{OVERFLOW RATE} = V_o = \frac{H}{t} = \frac{H}{v/Q} = \frac{Q}{SA}$$

Figure 12. Development of overflow rate in an ideal settling basin.

SA is the surface area of the settling zone [L2], and

Q is the flow in the settling zone [L3/T].

The settling process is relatively slow, such that it is the removal rates that are important, rather than the equilibrium state (Rich 1974). Under ideal conditions, particles settle at a velocity (vp) governed by Stoke's Law, and are removed if vp is greater than vo. Additional particles are removed which enter the settling zone at a height (hp) less than

$$hp = vp*td \tag{2}$$

The total basin removal is given by

$$R = (1-xo) + (1/vo) \int_{0}^{xo} vp \, dx \tag{3}$$

where R is the pollutant removal efficiency, and

xo is the proportion of particles with vp less than vo.

Rarely do detention facilities perform under ideal conditions. Most often, design efficiencies are reduced due to violations of ideal assumptions caused by short circuiting and turbulence, which alter the kinetics from ideal quiescent conditions. Short circuiting is induced through thermal currents, wind action, influent inertia, etc. Resuspension of solids may occur as the flow rate exceeds the scouring velocity. Thomas and McKee derived the

effect of longitudinal dispersion in a basin consisting of n completely mixed plugs (Fair, Geyer and Okun, 1968). Figure 13 presents the relative effluent concentrations for an instantaneous injection of dye undergoing a first order decay as it passes through the basin. A completely mixed basin is shown as n=1, while an ideal plug flow basin (n=infinity) would be represented by a spike at t/td of unity. The net effect of altering the flow regime from quiescent to more turbulent conditions is the reduction of the reaction coefficient k (Fair, Geyer and Okun 1968; Rich 1974). Although there is no way to predict before operation the reduction for a particular basin, the phenomena can be represented as in Figure 14, where the reduction of k is given as

$$k/k0 = (1-c/c0)^{n} \qquad (4)$$

where k is the reaction coefficient,

k0 is the reaction coefficient under quiescent conditions,

c is the effluent concentration,

c0 is the initial pollutant concentration, and

n is the coefficient indicating the degree of turbulence, which increases as the flow regime diverges from ideal conditions.

Plug flow conditions are represented by n=0 where k=k0 and the pollutant decays according to

$$c/c0 = e^{(-kt)} \quad \text{or,} \quad R = 1-c/c0 = 1-e^{(-kt)}. \qquad (5)$$

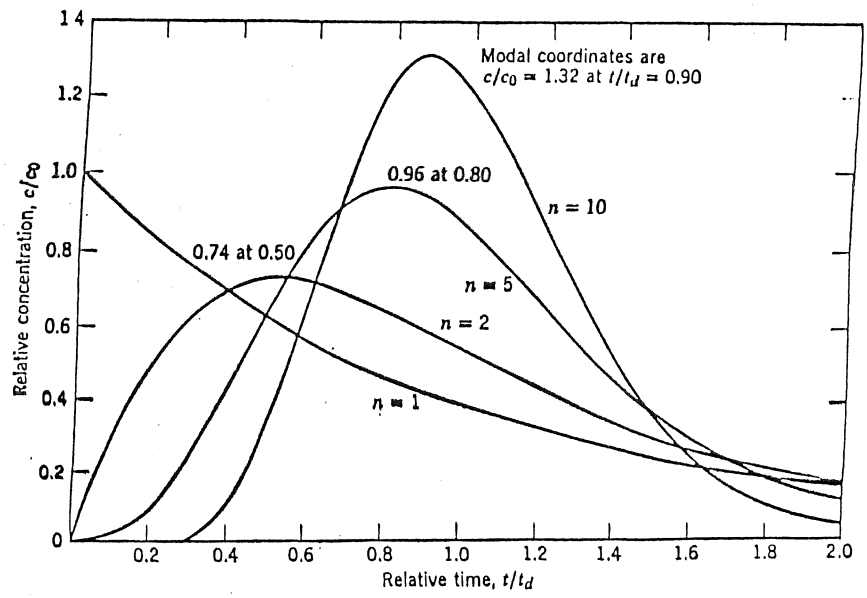


Figure 13. Effluent concentrations for a first-order removal process in n completely mixed plugs.

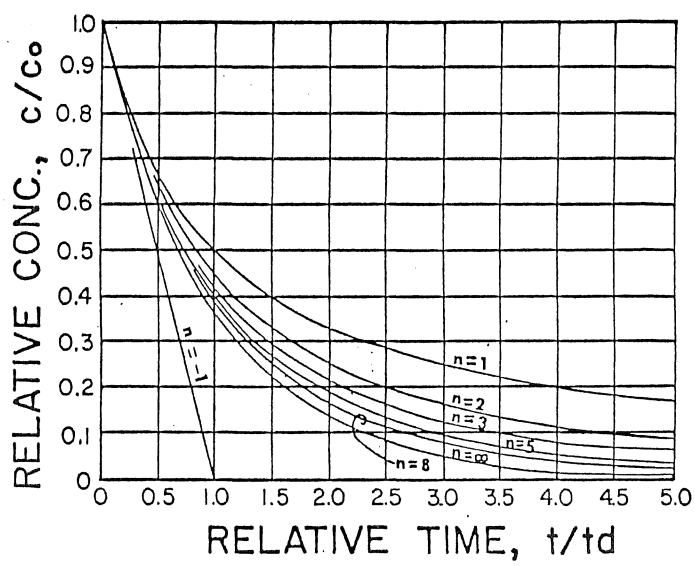


Figure 14. Removal efficiency for a first-order removal process demonstrating the effect of increased turbulence; n=-1 for quiescent and n=1 for completely mixed.

Comparison of Flow Conditions

Plug flow and completely mixed removal regimes define the extremes of pollutant removal performance. By definition, the hydraulic regime in each plug is complete mixing. Therefore, under static conditions, e.g. a column settling test, there is no difference between the two. The applicability of transferring column settling test results to dynamic conditions was recently re-examined by White (1976). In tank studies, he observed reasonable agreement with column tests, although the results were highly dependent on the waste characteristics. The effect of longitudinal dispersion on plug flow performance has been studied to correlate plug flow and complete mixing under dynamic conditions (Weber 1972). Ideal plug flow is represented as having zero dispersion while complete mixing is assumed to have infinite dispersion. An example of the effect of dispersion is reflected in Figure 15, showing effluent responses to a step input. Weber (1972) presents steady-state solutions that were derived from the general continuity equation fitted to a dispersion model. For a first-order reaction, the plug flow solution (zero dispersion) is

$$c/c0 = e^{(-ktd)} \quad \text{or,} \quad R = 1-e^{(-ktd)} \tag{6}$$

For a completely mixed basin, the steady-state solution is

$$c/c0 = 1/(1+ktd) \quad \text{or,} \quad R = 1-1/(1+ktd) \tag{7}$$

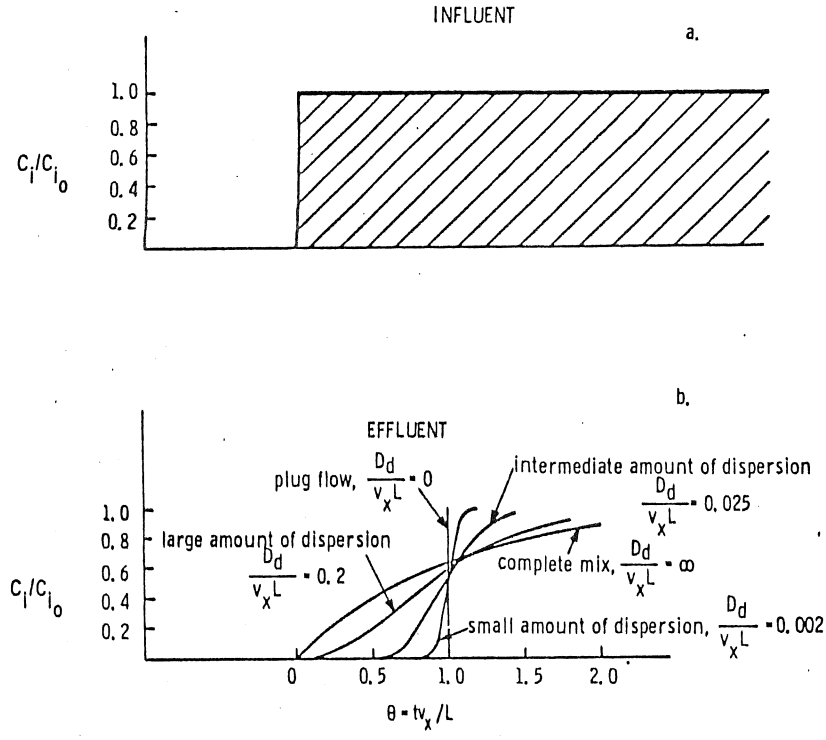


Figure 15. Effluent responses to a step input.

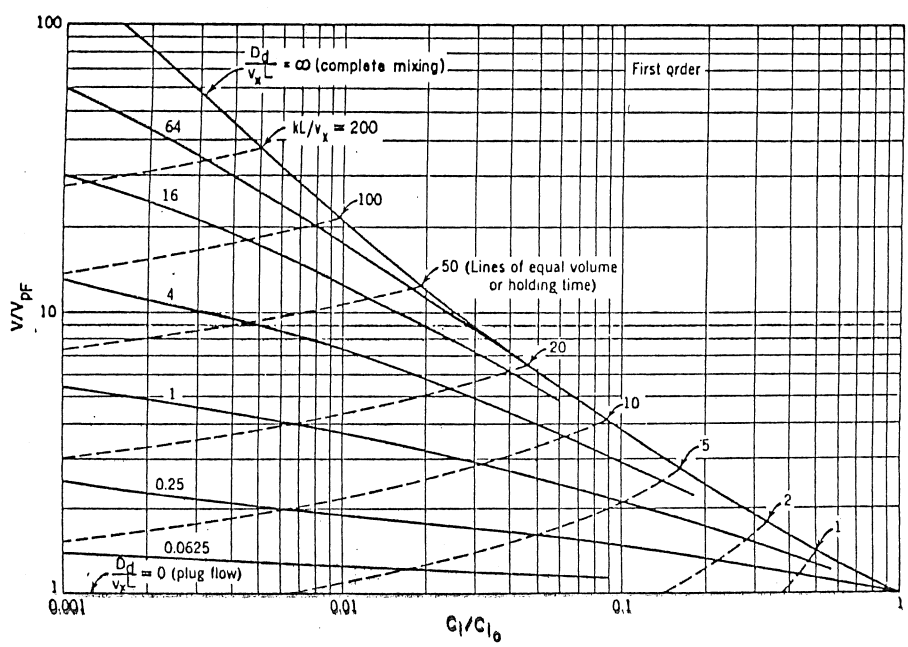


Figure 16. Comparison of real and plug flow reactor volumes for a first-order reaction.

These two solutions represent the removal efficiency extremes. The lower removal efficiency of complete mixing is a result of the basin concentration being continuously mixed with the influent, yielding a dilution of the influent but a concurrent increase in the basin concentration. In plug flow, the basin effluent more clearly reflects the removal process occurring in the separate plugs. The difference between the steady-state solutions for first-order reactions is graphically presented in Figure 16, which compares the volume of an ideal plug flow basin to the volume of a basin with dispersion yielding the same removal efficiency (Weber 1972). From Figure 16 it is possible to predict the effect of implementing dispersion reducing mechanisms such as baffles. For example, by reducing the dispersion factor from infinity (completely mixed) to 1.0, the volume necessary to provide 90 percent removal is reduced by a factor of 2.

There has been no widely used method for sizing S/T facilities with long-term stormwater quality control as the main objective. Heaney, et al. (1979) presented the mechanics for determining the optimal combination of S/T for steady state conditions, subject to economic constraints.

METHODOLOGIES

ANALYTICAL

Analytical models are developed from a combination of the general mass continuity and advective-dispersion equations. They range in complexity from simple steady-state approximations to more complicated variable flow solutions. The complexity of solution for detailed problems intimidates users and prohibits widespread application. Customarily, a simplification of the system's dynamic processes is necessary to obtain a solution. The analytical approach to stormwater detention facilities has been advanced by Medina (1976). Constant and variable (linear) flow and storage conditions were studied. Conservative and nonconservative pollutant routing were included. Effluent concentrations were derived for "simple" forcing functions. Application of this method requires the runoff flows and pollutants be converted to one of the input functions for which solutions have been derived. The solutions are limited in application due to their complexity and sensitivity to storage and flow conditions, i.e. the need to reduce the complexity of the dynamics to facilitate the analytical solution. The complexity of these solutions

precludes their application to stormwater runoff events without the use of detailed simulation.

EMPIRICAL

A widely used method for characterizing the rainfall-runoff process is the application of a conversion factor to mean rainfall values. Conversion factors are catchment specific and have been correlated to imperviousness, land use, population density and depression storage (Hydroscience, Inc. 1979, Chow 1964). While this method accounts for an average reduction in the volume of storms, it does not correct for the decreased number of events, i.e. it doesn't account for the small volume storms that are retained in the catchment storage.

Empirical relationships for runoff water quality have also been developed. Receiving water studies have determined runoff coefficients for with nutrient loadings based on land use (Reckhow 1980). In the urban arena, Smolenyak (1979) determined coefficients of the power equation

$$load = a(flow)^{**}b \qquad (8)$$

where load is the pollutant load in the runoff [M],
flow is the runoff flow [L3/T], and
a, b are coefficients.

Values determined from data in the Urban Rainfall-Runoff Quality Data Base are presented in Table 3.

The practice of water quality control via settling is inherent in most applications of natural waters. Water and wastewater treatment employ several variations of settling basins. Natural hydrologic systems incorporate detention for both the storage of kinetic energy and the deposition of sediment load. As a result of the implementation of reservoirs, siltation studies have contributed to the identification of the relationships between sedimentation and hydraulic parameters. The most popular sediment trap studies are those of Brown, Brune, Churchill and Camp (Ward and Haan 1977). Chen (1976) presents the historical development of theoretical analyses of sediment retention. These methods require the removal assessment of detention facilities based on an annual time frame. As there was no way to reconcile this with the stochastic nature of runoff events, these empirical methods were not investigated further. A recent review by Nix, et al (1980) summarizes the benefits in utilizing detention facilities for stormwater quality improvement. Several combinations of empirical and statistical approaches were suggested for design purposes.

TABLE 3.

Relationships between Pollutant Loads and Flow Volume (FLOW).

Dependent Variable	R^2	Sig. Level F-Test	No. of Events	Model Load = a(FLOW)^b Reg. Coef. a	b
BOD	.28	.99	80	34.0	1.12
COD	.76	.99	157	29.8	1.08
NH3N	.44	.99	20	.215	.72
NITN	.80	.99	21	.119	.80
NTOT	.57	.99	103	.0400	.71
ORGN	.88	.99	40	.856	1.04
TOTN	.74	.99	37	.304	1.07
DOP	.83	.99	34	.0648	.98
TOP	.46	.99	119	.0104	.78
TOTOP	.27	.90	11	.0800	.55
TOTP	.66	.99	53	.426	1.5
TPHOS	.91	.99	8	.105	1.05
TOTS	.69	.99	41	279	1.41
TSS	.56	.99	260	44.2	1.10

Source: Smolenyak 1979

SIMULATION

Several models are available for the simulation of the rainfall- runoff process and detention pond performance. Simulation is amenable to a trial and error routine, leading to search techniques for determining an acceptable design.

STORM

The Storage, Treatment and Overflow Model (STORM) of the Corps of Engineers models wet-weather flow through separate storage and treatment facilities (U.S.A.C.O.E. 1974). Runoff quantity is generated from an empirical conversion factor applied to rainfall excess. Runoff quality is generated as a function of land use. While flow is routed through the treatment plant, there is no capacity to model the quality improvement provided by detention.

SWMM Overview

The Storm Water Management Model (SWMM) is one of the most comprehensive and well documented models available for the analysis and design of urban stormwater systems (Huber et al. 1980). SWMM was developed by Metcalf and Eddy, Inc., the University of Florida and Water Resources Engineers, Inc. under a contract for the Environmental Protection Agency. Version I was released in 1971 and has been continually undergoing revision and updating. This study

utilized Version III, released in 1980, and also incorporated further refinements. While the model allows for extensive watershed simulation, this study restricted the analysis to a simplified Runoff Block and concentrated on the Storage/Treatment Block.

The Runoff Block deterministically models the rainfall-runoff process in a catchment. Physical data such as depression storage, infiltration rates, soil characteristics, catchment area, ground slope, gutter network, evapotranspiration rates and hourly rainfall values are included as the model input. Pollutant build-up and wash-off functions are available, as is pollutant generation based on catchment land use. The Runoff Block generates hydrographs and pollutographs as options. Several inherent adjustment factors are available for calibration of the model.

The Storage/Treatment (S/T) Block of SWMM models the flow and pollutant routing through a storage and/or treatment device which can be either a detention or non-detention unit. Geometric and hydraulic relationships, e.g. depth to surface area, and evaporation rates, and incoming flows are included as data input for the S/T block. Discharge can be modeled as either constant or variable outflow. The package provides for a variety of removal mechanisms and flow routing options, capable of tracking both stormwater flows and constituent pollutants.

Data Input for Runoff Block

The Runoff Block accepts hourly rainfall data in the National Weather Service (NWS) format. No gutter or pipe networks were used in the example catchment; all runoff exited the area via a dummy outlet. The impervious area was modeled separately from the pervious area. Runoff generated on either area went directly to the dummy outlet without passing through the other area. Also, there were no areas of depression storage in the modeled catchment. The data input for the Runoff Block consisted of the catchment characteristics presented in Table 1. A complete listing of the data input is given in Appendix A.

Currently, continuous SWMM can only be run with hourly rainfall input. As an aside, the hourly rainfall data were transformed into daily and weekly records. SWMM Runoff Block was run on the first 19 months of the Atlanta data and the results are compared in Table 4.

Table 4. Comparison of Runoff Block results using hourly, daily, and weekly rainfall input.

Rainfall Time Step	Average Intensity	Total Rainfall	Infilt. (in)	Evap. (in)	Runoff (in)
HOURLY	HOURLY	81.68	51.72	6.39	28.68
DAILY	DAYTOT/Dr	80.53	50.46	8.18	28.00
WEEKLY	WEEKTOT/2Dr	81.62	51.73	6.11	28.62

Reasonable agreement was obtained, suggesting that if hourly data were not available, continuous SWMM could be run using daily or weekly rainfall data.

Data Input for Storage/Treatment Block

The basin geometry and hydraulic characteristics required as input data for the S/T block were obtained from a separate computer program, BASIN, written for that purpose. BASIN calculates stage to surface area, stage to volume and stage to discharge relationships for basins given the dimensions, side slope and outlet configuration. Examples are provided in Figure 17. The program development and listing is provided in Appendix B.

As a reference, basin volumes (Vb) were normalized to the mean runoff volume per event (Vro), yielding a normalized volume ratio (Vb/Vro). Currently, SWMM does not define storm event statistics, so the SYNOP program was employed. This involved running the Runoff Block with the complete 24.6-year (June 1948 - December 1972) rainfall record and generating 24.6 years of simulated runoff data. These data were transformed to the format of the NWS rainfall data, which is compatible with the SYNOP input format. As with the rainfall data, the minimum interevent time was varied to obtain the coefficient of variation (cv) for the interevent time close to unity. The results of these runs are presented in Table 5.

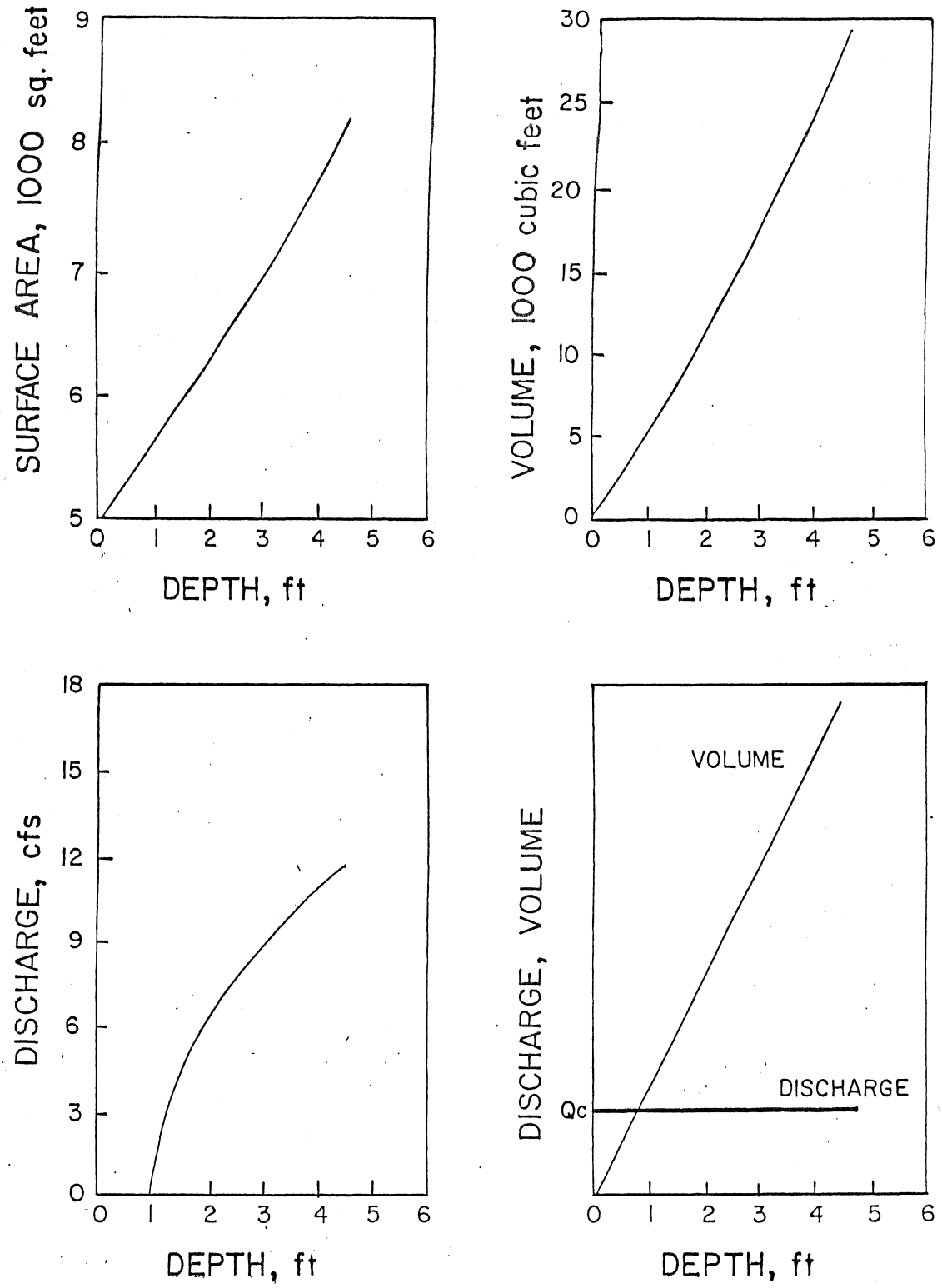


Figure 17. Stage relationships as calculated by BASIN; constant and variable discharge.

Table 5. SYNOP results of 24.6 years of simulated runoff data.

Minimum # of dry hrs	Number	Volume Vro (in)	CV	Duration Dro (hr)	CV	Intensity Iro (in/hr)	CV	Interevent Time Ti (hr)	CV
3	2124	0.180	1.253	4.507	0.964	0.041	1.057	101.35	1.172
4	1998	0.192	1.221	4.980	0.978	0.041	1.058	101.75	1.114
5	1903	0.201	1.196	5.428	0.989	0.041	1.063	113.13	1.070
8	1760	0.217	1.194	6.438	1.030	0.040	1.073	122.32	0.999
12	1646	0.274	1.181	7.448	1.093	0.039	1.052	130.80	0.940

A minimum interevent time of four hours resulted in a cv of 1.000. The mean runoff volume per event (Vro) was determined to be 18022 cubic feet, based on the mean depth of 0.201 inches over 24.7 acres. The number of events was reduced from 2381 rainfall events (Table 2) to 1998 runoff events. Continuity is checked by comparing the product of the mean event volume and the total number of events with the amount of runoff generated by the Runoff Block.

(0.192 inches/event)(1998 events) = 383.61 inches;

from Runoff, 388 inches.

Test basin volumes (Vb) were obtained from BASIN to closely approximate volume ratios (Vb/Vro) of 0.5, 1.0, 2.0, 4.0, 10.0. For example, a Vb/Vro ratio of 0.50 implies that the empty basin volume is 50 percent of the mean storm event volume. The basin volumes used are presented in Table 6.

Table 6. Determination of normalized volume ratios (Vb/Vro).

Basin volume (Vb) (cubic feet)	Normalized volume ratio Vb/Vro
7744	0.430
15014	0.833
29064	1.613
74420	4.129
167835	9.313

The input data for the Runoff Block are provided in Appendix A. The S/T Block utilized the runoff values generated from the Runoff Block. To save execution time and money, the Runoff Block was run once and the output stored on an interface data disk. The interface data set served as the input to S/T for the subsequent simulations. A constant suspended solids concentration of 100 mg/l was assigned to the influent. This was chosen as opposed to generating pollutants from the catchment area for four reasons:

1) it avoids concern over how the pollutants are generated;
2) it provides a base value (100 mg/l) for future comparisons;
3) it provides a blocked-off step input as shown in Figure 18; and,
4) a constant influent concentration establishes that the percent of flow bypassed is numerically equal to the percent of the pollutant bypassed.

The actual flow condition in a basin is neither plug flow nor completely mixed, but somewhere between, termed "intermediate mixing". The complete mixing option of S/T was chosen for the flow routing regime for its analytical and computational simplicity, resulting in lower simulation costs than the plug flow method.

Two options of basin discharge were explored: variable outflow based on hydraulic head above an outlet, and a

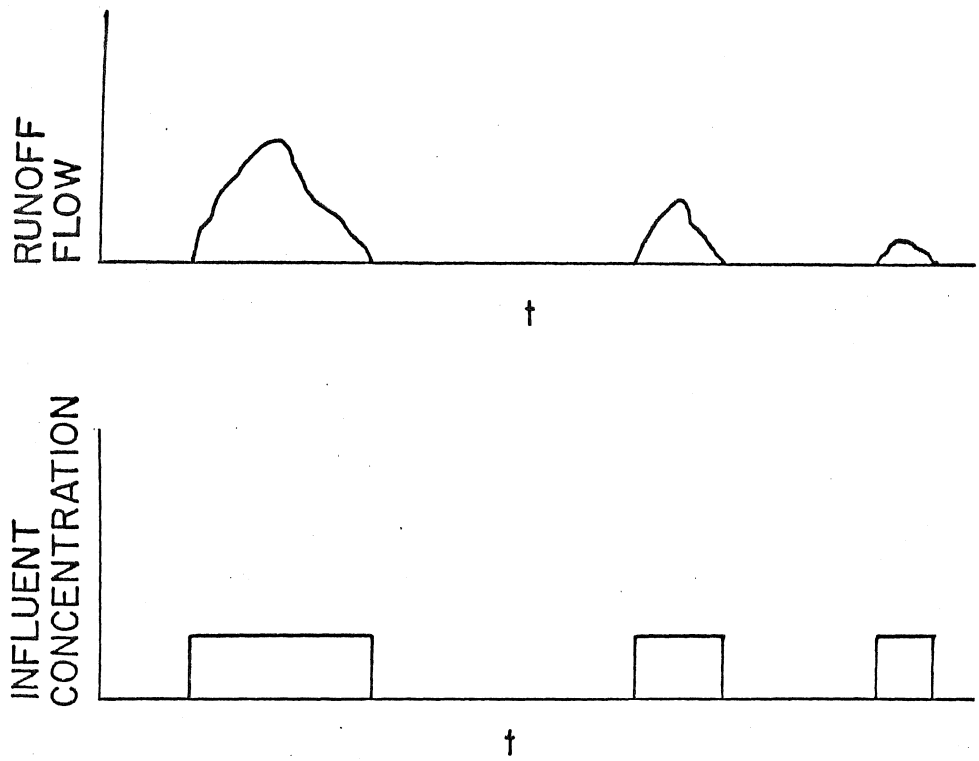


Figure 18. Time series of constant influent pollutant concentration.

drawdown scheme which emptied the basin at a constant rate (see Figure 17). The pumping option was run for comparison with the statistical technique, while the variable discharge option was utilized for application to basins with gravity drainage.

Constant Discharge Simulation

A variable volume, constant outflow unit was simulated by using the pumping option of S/T. As a reference, the constant drawdown rate was normalized as QcTi/Vro

where Qc is the drawdown rate in cubic feet per hour,

Ti is the mean interevent time in hours, and

Vro is the mean runoff volume in cubic feet (Hydroscience, 1979).

As Ti and Vro are constants determined from SYNOP, variable values of the ratio reflect different drawdown rates. These rates were calculated to yield ratios of 1, 2, 4, 7, and 10 and are presented in Table 7.

It was assumed that drawdown occurs whenever there is water in the basin. The effect of drawdown height (Hd) and drawdown rate on capture and removal efficiencies was analyzed.

Table 7. Calculation of constant discharge rates (Qc) (Ti = 111.17 hours; Vro = 18022 cubic feet).

Normalized discharge ratio QcTi/Vro	Drawdown rate (Qc) (cubic feet/hr)
1	162
2	324
4	648
7	1135
10	1621

Variable Discharge Simulation

A variable volume, variable outflow control unit was simulated with the S/T Block by utilizing a power equation for basin discharge based on hydraulic head. This simulates the hydraulics in a basin with gravity drainage. The outlet characteristics were arbitrarily assigned as a six inch circular opening placed one foot above the bottom. Instead of a solution surface, as was provided in the constant discharge simulations, a single removal curve was determined for basin performance versus basin volume. In an analogous manner to the drawdown height and rate combinations, outlet elevation and cross-sectional area were recognized as design parameters for basin performance and were analyzed. Evaporation in the S/T unit was arbitrarily assigned a value of 0.1 inch per day.

Removal Mechanism

A removal equation was chosen of the form

$$R = Rmax(1-e**(-kt)) \quad (9)$$

where R is the pollutant removal efficiency,

Rmax is the maximum removal efficiency,

k is the first-order rate coefficient [1/T], and

t is the treatment time [T].

Fair, Geyer and Okun (1968) presents general removal curves with k near 1.4 per hour for TSS and 0.50 per hour for BOD, with Rmax of 0.75 and 0.45, respectively. For design purposes, values for Rmax and k would be determined from column settling tests with representative pollutants. Values of 1.0 and 0.6 per hour, respectively, were arbitrarily assigned for these parameters. In the S/T Block, removal is accounted for once per time step, with the length of the time step, one hour, as the treatment time. The removal equation is presented in Figure 19. Because the time step was held constant throughout the simulation there was a constant percent removal (45 percent) of pollutant per time step. A comparison of effluent concentrations for removal governed by this equation in an ideal plug flow basin, an ideal completely mixed basin and the S/T complete mixing regime is presented in Figure 20. As shown, the S/T results lie within the extremes of pollutant removal efficiency provided by ideal plug flow and complete mixing.

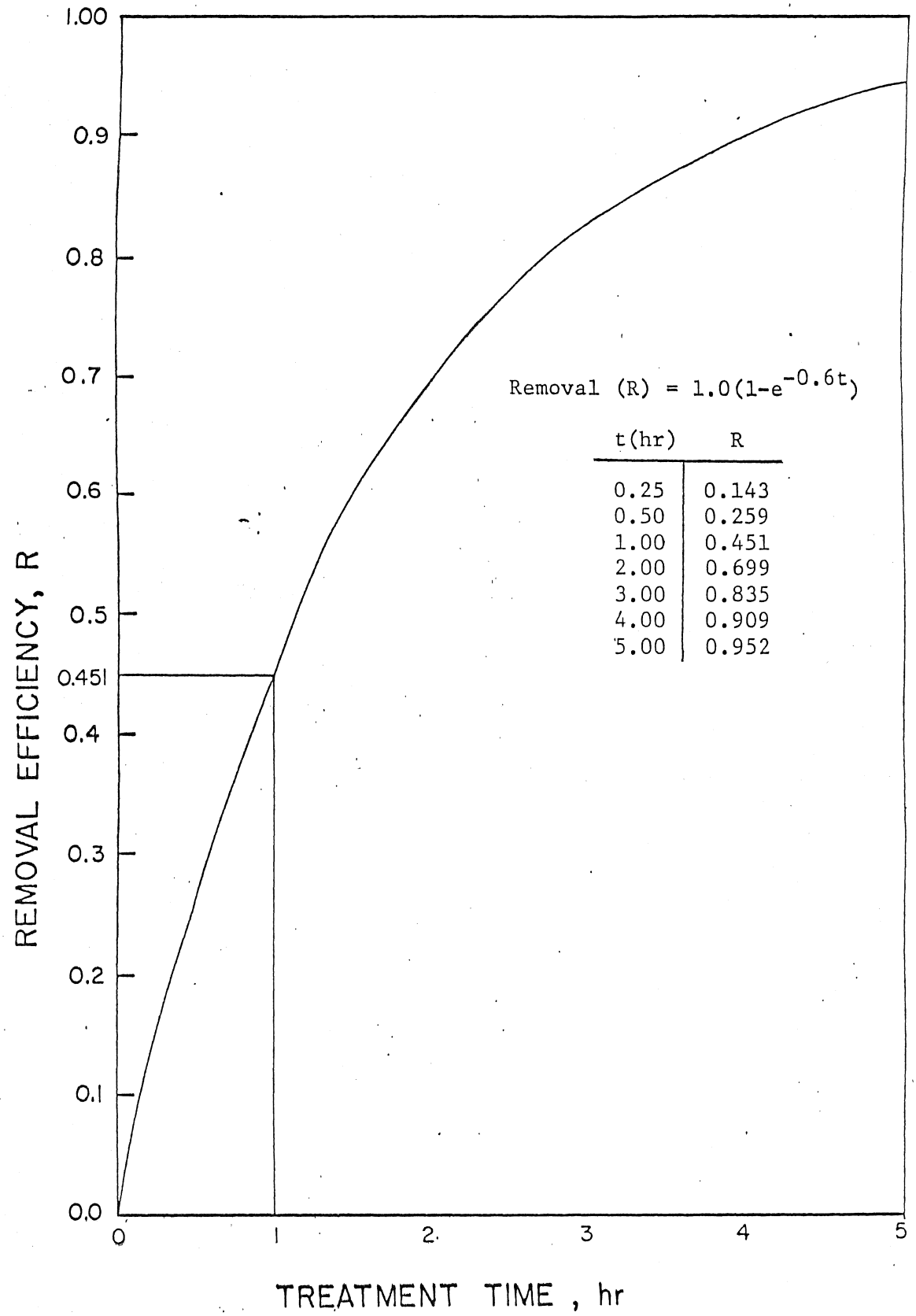


Figure 19. Removal equation used in SWMM S/T Block.

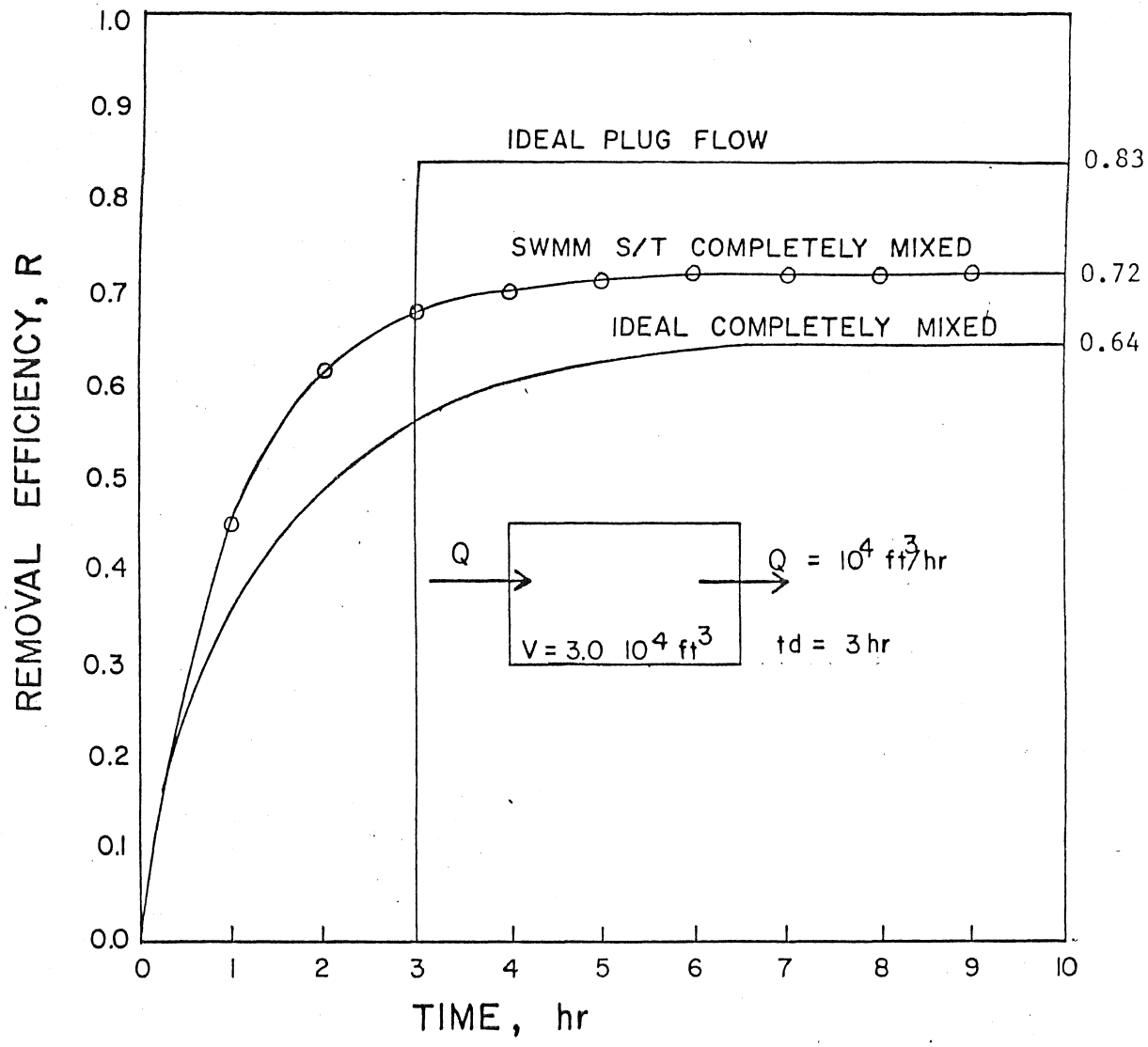


Figure 20. Comparison of effluent concentrations under ideal plug flow, ideal completely mixed and as calculated with SWMM S/T completely mixed routing; step input of pollutant.

One-year Versus 24.6-year Simulation

The initial decision to use one year of data as opposed to the full 24.6-year history was based on economic considerations; mistakes and debugging were expensive enough without extra data magnifying the costs. The first 12 months of the input record (June 1, 1948 - May 31, 1949) were utilized to get the simulator running. It was realized that if a "typical" runoff year's simulation adequately reproduced long-term basin performance, the costs of the analysis would be reduced by as much as an order of magnitude. A "typical" year, 1953, was chosen on the basis of similar synoptic statistics, as determined by the SYNOP run of the 24.6-year runoff data. A comparison of 1953 runoff parameters with those of the 24.6-year time series is presented in Table 8.

The adequacy of one year's simulation for describing the long-term basin performance was analyzed in a series of simulations; the results are presented below.

Runoff Block Results

A variety of summary print options are available in the Runoff Block, from detailed hourly results to the total simulation summary, as presented in Table 9. On the hypothetical catchment, 704 inches (60 percent) of the total 1179 inches of rain left via infiltration. A total of 388 inches (33 percent) accumulated as runoff, while 150 inches

Table 9. Runoff Block 24.6-year simulation summary.

	MILLION CUBIC FEET	INCHES OVER TOTAL BASIN
TOTAL PRECIPITATION (RAIN PLUS SNOW)	105.512	1176.79
TOTAL INFILTRATION	63.106	703.83
TOTAL EVAPORATION	7.846	87.51
TOTAL GUTTER/PIPE/SUBCAT FLOW AT INLETS	34.830	388.47
TOTAL WATER REMAINING IN GUTTER/PIPES	0.000	0.00
TOTAL WATER REMAINING IN SURFACE STORAGE	0.000	0.00
$ ERROR IN CONTINUITY, % OF TOTAL PRECIP	-0.256	
* * * * * RUNOFF SIMULATION ENDED NORMALLY * * * * *		

Table 8. Comparison of mean runoff event parameters for 1953 with the 24.6-year record (minimum interevent time = 4.0 hours).

	24.6-year	1953
Volume (in)	0.201	0.217
cv	1.222	0.948
Duration (hr)	5.066	6.190
cv	0.972	1.087
Intensity (in/hr)	0.043	0.044
cv	1.148	1.164
Interevent time (hr)	111.71	104.83
cv	1.000	0.986

(8 percent) were lost to evaporation. Mass continuity was preserved within 0.3 percent over the total 24.6-year simulation.

Storage/Treatment Block Results

The format for the S/T results are similar to the runoff output with more emphasis on quality parameters. Again, as shown in Table 10, the results were presented to facilitate continuity checks.

Table 10. Examples of Storage/Treatment Block summaries.

DETENTION UNIT CHARACTERISTICS:

POLLUTANT ROUTING METHOD : COMPLETELY MIXED
RESIDUALS DRAW-OFF SCHEME: NEVER DRAWN OFF 0.0

DEPTH-AREA-STORAGE-FLOW RELATIONSHIPS :

DEPTH, FT.	SURFACE AREA, SQ. FT.	STORAGE, CU. FT.
0.0	5000.0	0.0
0.50	5304.0	2576.0
1.00	5616.0	5306.0
1.25	5775.0	6729.9
1.50	5936.0	8193.7
2.00	6264.0	11243.7
2.50	6600.0	14459.7
3.00	6944.0	17845.7
3.50	7296.0	21405.7
4.00	7656.0	25143.7
4.50	8024.0	29063.7

* GOVERNED BY PUMPING

PUMPED OUTFLOW:

DEPTH AT WHICH FIRST PUMPING RATE BEGINS, FT.	:	0.0
DEPTH AT WHICH SECOND PUMPING RATE BEGINS, FT.	:	0.0
FIRST PUMPING RATE, CFS	:	0.45
SECOND PUMPING RATE, CFS	:	0.45
DEPTH AT WHICH ALL PUMPING STOPS, FT.	:	0.0

UNIT	PARAMETER	VOLUME (CU. FT.)	CAT KAKA LBS.
1	INFLOW, TOTAL	0.1456E+07	0.4547E+04
	INFLOW, NET	0.1386E+07	0.4327E+04
	BYPASS	0.7027E+05	0.0
	TREATED OUTFLOW	0.1364E+07	0.1422E+04
	RESIDUAL FLOW	0.0	0.0
	REMOVED BY DECAY	–	0.2913E+04
	REMAIN. TOT. VOL.	0.5285E+04	0.3216E-01
	EVAPORATION	0.1700E+05	–

Constant Discharge Simulation

Performance results from the constant discharge simulations are presented in Figures 21, 22 and 23, and in Table 11.

Table 11. Estimates of flow capture efficiency (C) and pollutant removal efficiency (R) as a function of basin volume and constant discharge rate: simulation results.

		Normalized Volume Ratio							
Normalized Discharge ratio		0.43		0.83	1.61		4.13	9.31	
		1953	24.6 yr	1953	1953	24.6 yr	1953	1953	25-yr
1	C	0.344	0.331	0.476	0.613	0.640	0.852	1.000	0.952
	R	0.343	0.322	0.476	0.598	0.638	0.847	0.989	0.941
2	C	0.398	*	0.552	0.721	*	0.945	1.000	*
	R	0.373	*	0.527	0.690	*	0.906	0.961	*
4	C	0.468	0.461	0.621	0.804	0.795	0.962	1.000	0.993
	R	0.373	0.383	0.532	0.712	0.705	0.864	0.905	0.896
7	C	0.542	*	0.699	0.839	*	0.971	1.000	*
	R	0.365	*	0.524	0.661	*	0.792	0.817	*
10	C	0.611	0.608	0.744	0.868	0.876	0.982	1.000	0.999
	R	0.342	0.351	0.483	0.603	0.619	0.713	0.728	0.736

* indicates that simulation was not run.

The 1953 simulations duplicated the performance results of the 24.6-year simulations within five percent over the entire spectrum of basin volumes and discharge rates. Figure 21 presents the solution surface for capture efficiency as a function of drawdown rate and basin volume.

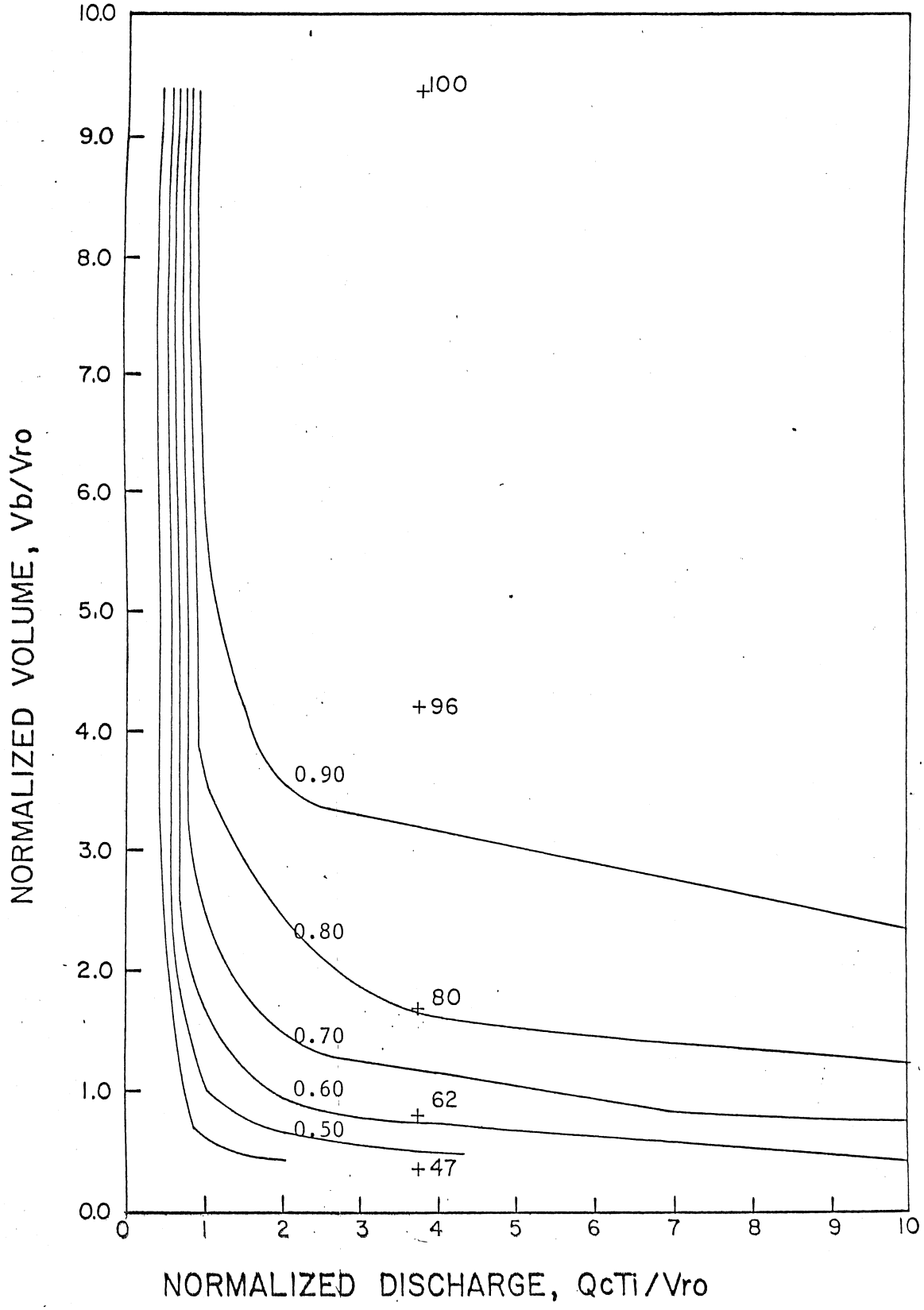


Figure 21. Solution surface of flow capture efficiency (C) as a function of basin volume and constant discharge rate: 1953 simulation results.

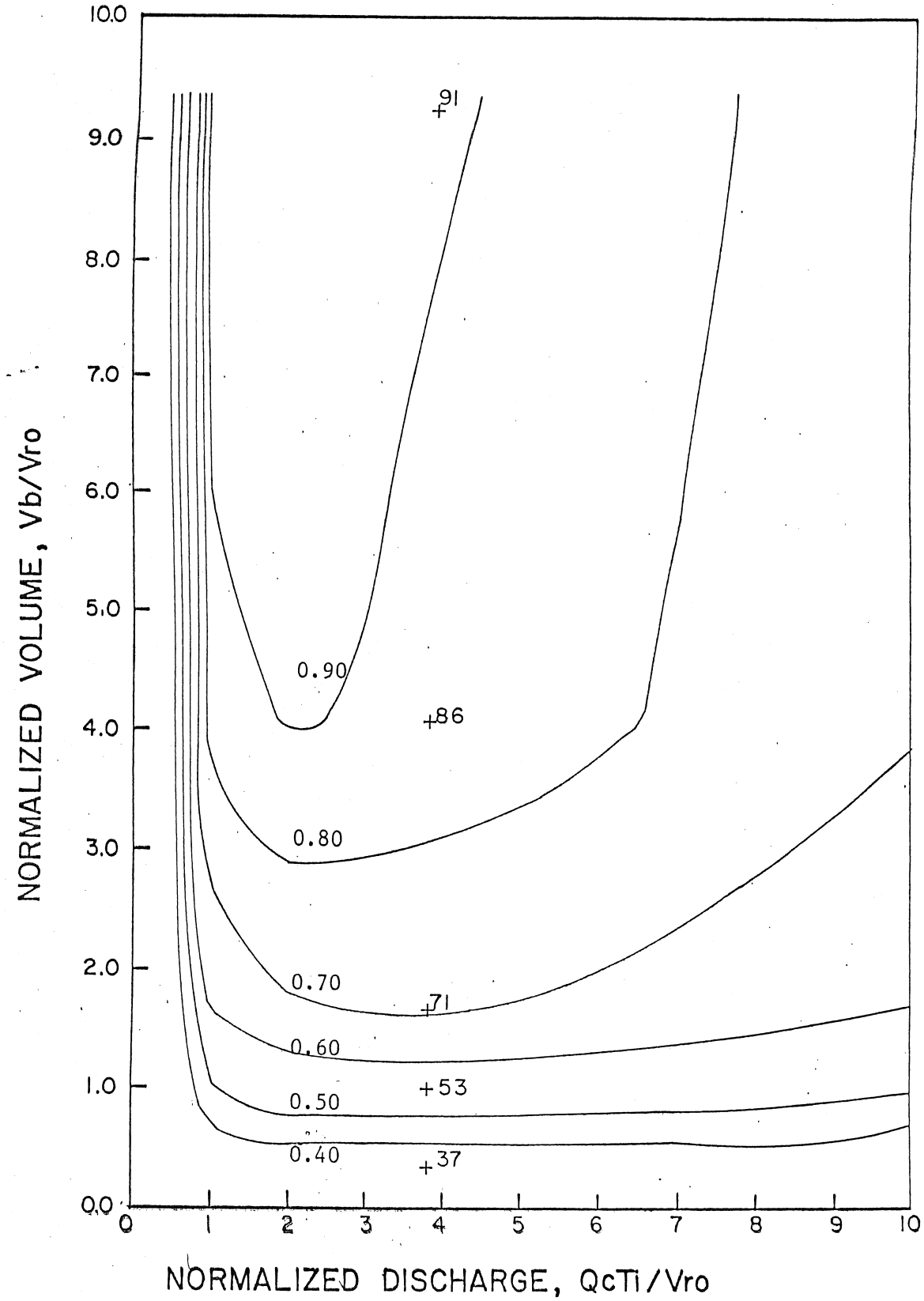


Figure 22. Solution surface of pollutant removal efficiency (R) as a function of basin volume and constant discharge rate: 1953 simulation results.

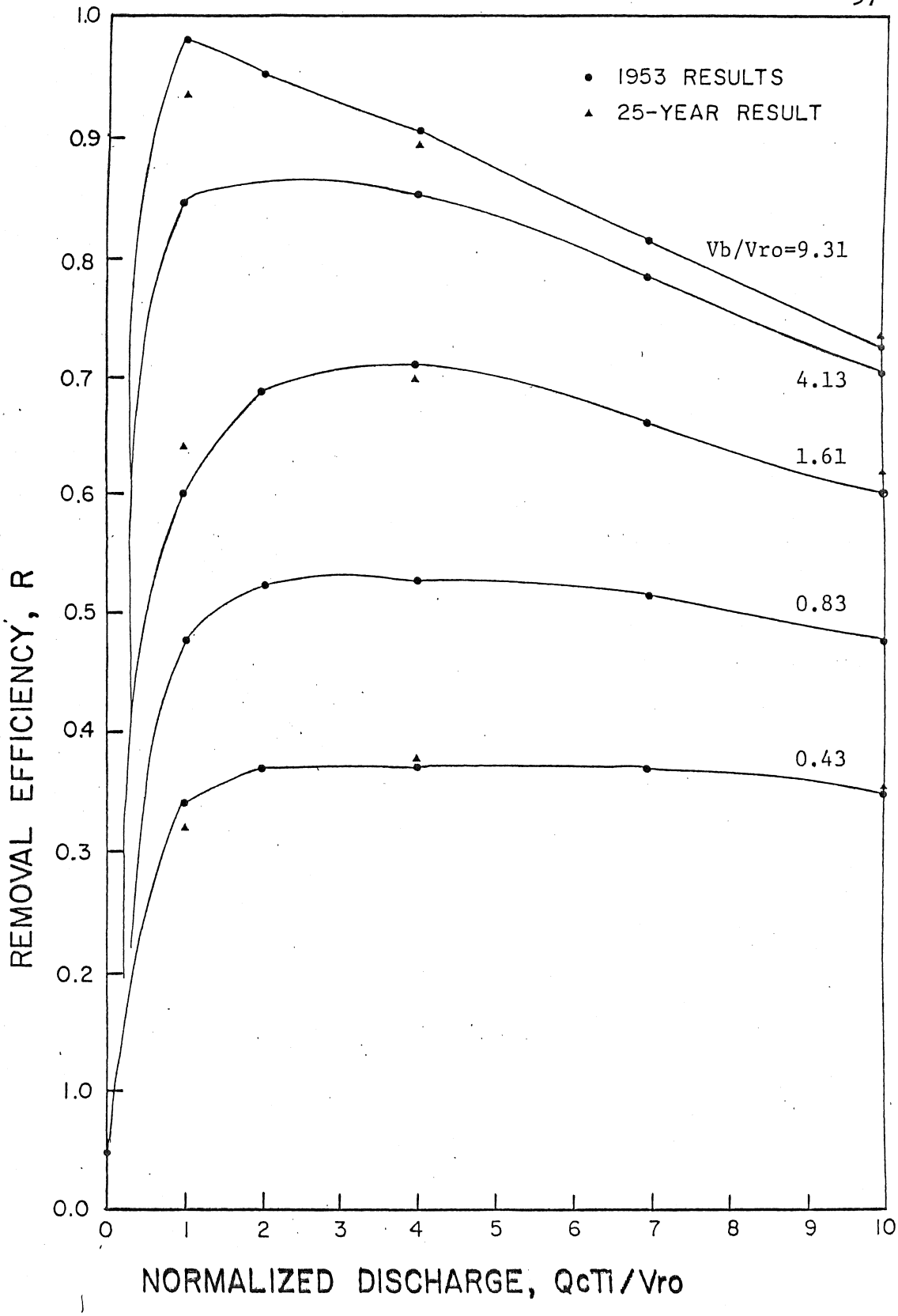


Figure 23. Pollutant removal efficiency (R) as a function of basin volume and constant discharge rate: 1953 and 24.6-year simulation results.

Isopleths of percent capture were drawn by linear interpolation between calculated values. As expected, flow capture was greater as the drawdown rate increased, due to an increase in the effective volume. Also, as the basin volume increased, the capture efficiency increased due to less bypass. The vertical distance between the isoquants represents the sensitivity of capture performance to basin volume; the smaller the distance, the greater the sensitivity. for drawdown rates greater than 4.0, there appears to be uniform sensitivity to basin volume. The lowest sensitivities occur at the lower drawdown rates (QcTi/Vro less than 2.0). The isoquants converge slightly toward the upper end of the abscissa. The horizontal distance separating the isoquants represents the sensitivity of capture performance to the drawdown rate. The isoquants become parallel to the abscissa above QcTi/Vro of 4.0, implying relative insensitivity to drawdown rate. Sensitivity is increased as the drawdown rate is decreased.

Figure 22 presents the solution surface for pollutant removal efficiency as a function of basin volume and drawdown rate. Unlike the solution surface of capture performance, the isoquants in Figure 22 slope upward after an initial negative slope. The result is a solution surface which allows more than one drawdown rate at a specific basin volume to achieve the same removal performance. This demonstrates the performance tradeoff of providing a larger

effective volume by emptying the basin quicker versus providing a longer treatment time, although bypassing more flow.

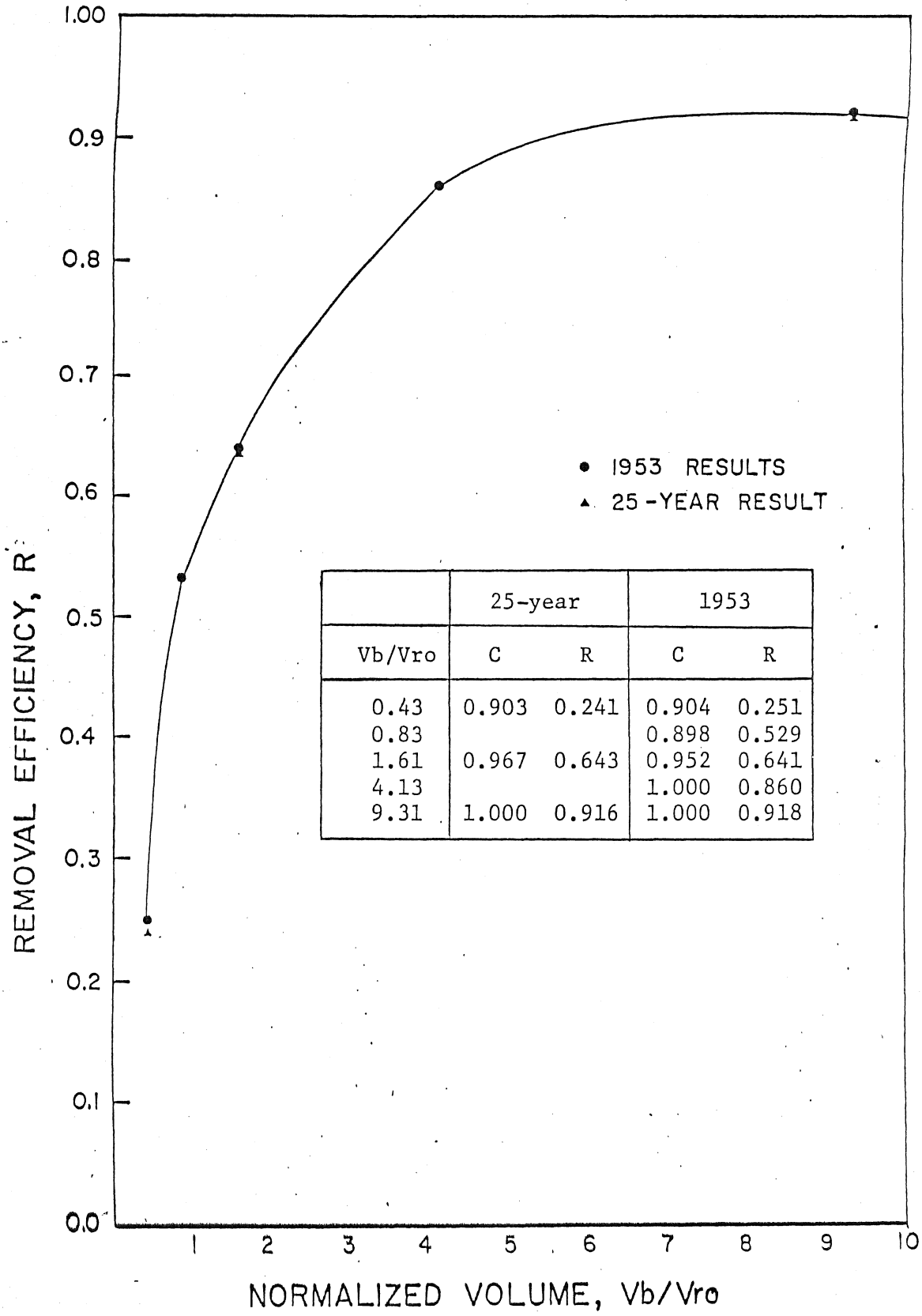
Combinations of basin volume and drawdown rate yielding equivalent removal efficiencies are depicted along isoquants. For example, the removal performance obtained by a Vb/Vro of 4.1 and a QcTi/Vro of 2.0 was the same as a Vb/Vro of 9.3 and a normalized discharge rate of 4.0. The greatest removal occurred in the region of large basin volumes (Vb/Vro > 4.0) and low drawdown rates (QcTi/Vro < 4.0). Figure 23 presents the removal performance in a different manner than in Figure 22. There is no increase in information by presenting the results in this way, although the communication of information is improved. For example, in Figure 23, it is easier than in Figure 22 to see that the sensitivity of removal to drawdown rate increases as the volume ratio increases. Combinations of basin volume and drawdown rates yielding equivalent removal as well as the sensitivity of the removal performance to drawdown rate are demonstrated. For example, the sensitivity of performance to drawdown is represented as the slopes of the curves, and is seen to increase as the volume ratio increases. As the volume ratio increases, the maximum removal efficiency for each volume occurs at decreasing drawdown rates. The 1953 performance curve for a Vb/Vro of 9.3 is depicted as a straight line. This represents a divergence (5 percent at

QcTi/Vro = 1.0) from the 24.6-year results, possibly due to the lack of a large storm during 1953. All of the curves converge to 4.3 percent removal at QcTi/Vro of 0.0, that is, in the case where there is no outlet. Long-term removal efficiency would undoubtedly be smaller for this case, tending to zero percent.

Variable Discharge Simulations

The effect of basin volume on capture and removal performance in basins with variable outflow rates was analyzed by running the SWMM S/T Block on five sets of basin geometry and hydraulic characteristics. The results presented in Figure 24 follow an intuitive removal relationship with increased removal as the storage capacity increases. The regions below Vb/Vro of 0.43 and above Vb/Vro of 9.31 were not explored because of the unlikeliness of such a small volume ratio. The resultant removal curve is neither an exponential nor a power equation for the range observed.

Only three 24.6-year simulations were run due to their low marginal benefit, i.e. the one-year simulations gave estimates close enough to the 24.6-year results to avoid spending the extra money for the long-term simulations. The costs of the runs averaged $0.60 for one-year and $6.25 for 24.6-year simulations.



	25-year		1953	
Vb/Vro	C	R	C	R
0.43	0.903	0.241	0.904	0.251
0.83			0.898	0.529
1.61	0.967	0.643	0.952	0.641
4.13			1.000	0.860
9.31	1.000	0.916	1.000	0.918

Figure 24. Pollutant removal efficiency (R) as a function of basin volume under variable discharge conditions: 1953 simulation results.

Optimal Basin Design

During preparation of the initial performance solution surfaces, it was recognized that the heights at which discharge began and ended would combine with the discharge rate to affect basin performance. Intuitively, increasing the height would decrease the capture efficiency, but due to the completely-mixed flow routing regime, the remaining volume would provide dilution of the influent.

Combinations of drawdown height (Hd) and rate (Qc) and similarly outlet diameter and invert height, were simulated in an attempt to develop guidelines for the optimal design of detention facilities.

A basin with a Vb/Vro of 1.61 was utilized for these simulations. The results from the constant drawdown simulations indicated that a basin with this ratio had the greatest performance sensitivity (26 percent capture and 11 percent removal) over the range of drawdown rates. In the variable discharge runs, a basin with a Vb/Vro of 1.61 yielded results in the knee of the removal curve. It was felt that a basin with this ratio was sensitive enough to reflect the effect of height and discharge combinations on basin performance.

The results of the constant drawdown rate simulations are presented in Figures 25 and 26 and Table 12.
The solution surface in Figure 25 indicates that maximum performance is achieved by a normalized discharge ratio

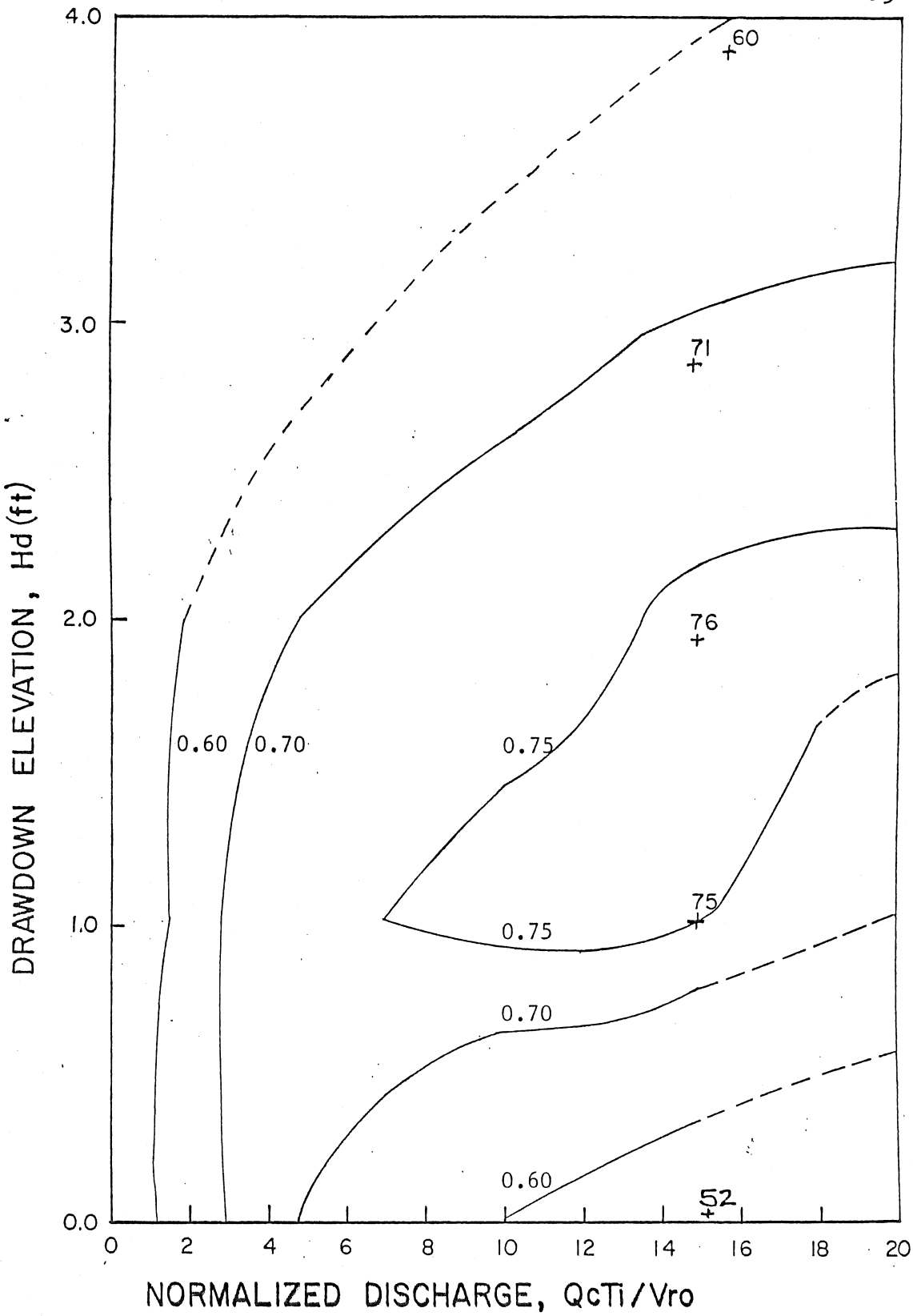


Figure 25. Solution surface of pollutant removal efficiency (R) as a function of drawdown height and drawdown rate (Vb/Vro = 1.61): 1953 simulation results.

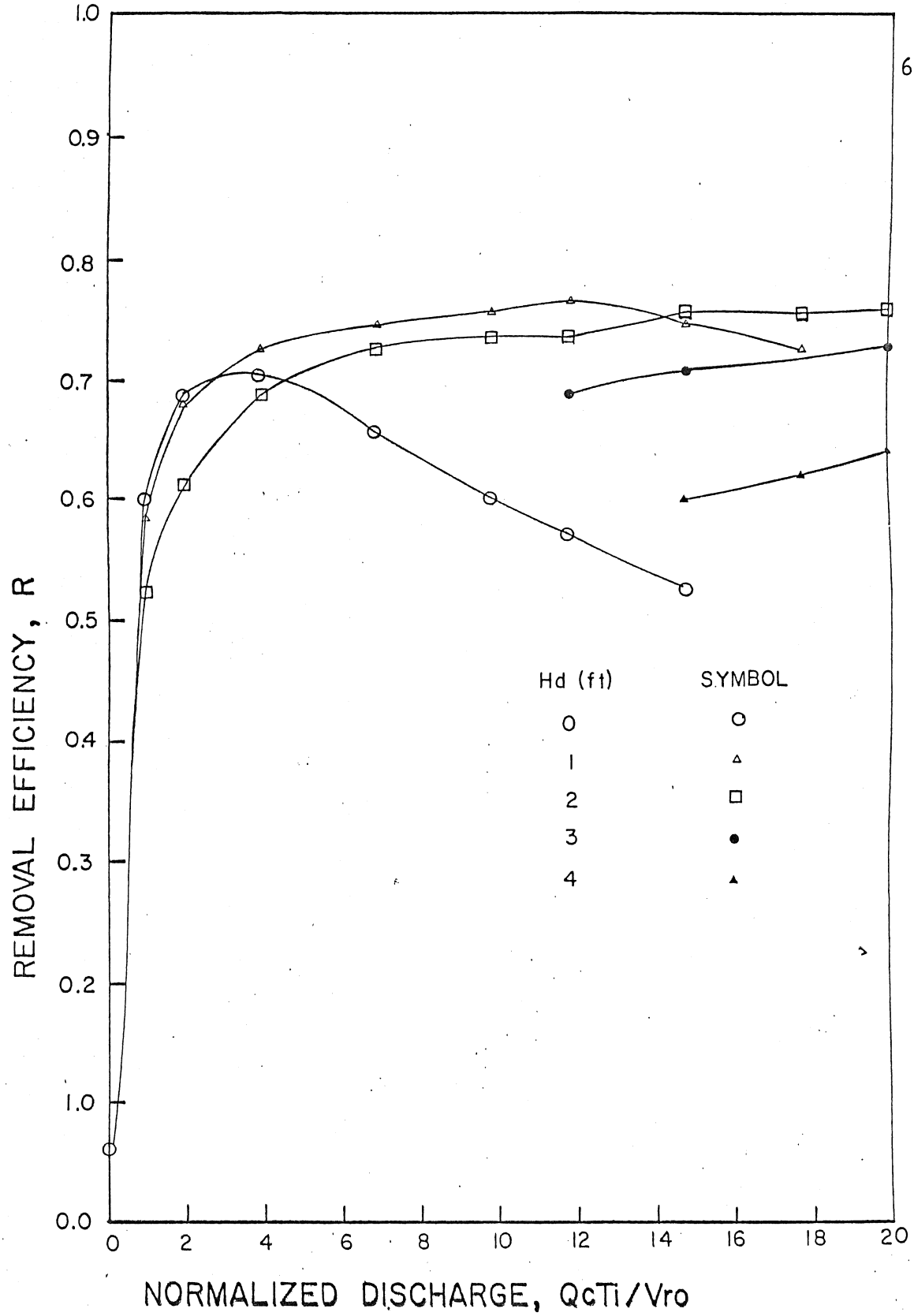


Figure 26. Pollutant removal efficiency (R) as a function of drawdown height and constant discharge rate (Vb/Vro = 1.61): 1953 simulation results.

Table 12. Estimates of flow capture efficiency (C) and pollutant removal efficiency (R) as a function of drawdown height and rate (Vb/Vro = 1.61): 1953 simulation results.

Normalized Discharge Ratio		Drawdown Height (ft) 0	1	2	3	4
1	C	0.613	0.578	0.517	*	*
	R	0.598	0.578	0.517	*	*
2	C	0.721	0.679	0.610	*	*
	R	0.690	0.679	0.610	*	*
4	C	0.804	0.756	0.687	*	*
	R	0.712	0.734	0.687	*	*
7	C	0.839	0.804	0.749	*	*
	R	0.661	0.752	0.728	*	*
10	C	0.868	0.837	0.778	*	*
	R	0.603	0.761	0.738	*	*
12	C	0.883	0.860	0.795	0.717	*
	R	0.570	0.766	0.744	0.694	*
15	C	0.900	0.874	0.831	0.745	0.610
	R	0.520	0.749	0.763	0.713	0.597
18	C	*	0.885	0.845	0.765	0.648
	R	*	0.730	0.758	0.727	0.621
20	C	*	*	0.851	0.776	0.663
	R	*	*	0.761	0.731	0.636

* indicates that simulation was not run.

of 12.0 with an Hd of one foot. Removal efficiencies for basins with complete drawdown (Hd of 0.0) were lower than limited drawdown (Hd greater than 0.0) for all but the lowest rates. The general trend of the isoquants indicates that similar performance can be achieved by a low drawdown

ratio and low height as well as a higher ratio and a corresponding higher drawdown elevation.

Figure 26 more clearly presents the removal performance associated with each height. As the height increased, maximum removal occured at higher drawdown rates. As the drawdown rate increased, removal efficiency increased until a maximum was reached, after which, further increase in drawdown rate yielded decreased removal.

The results of the variable outflow simulations are presented in Figures 27 and 28 and Table 13.

Table 13. Estimates of flow capture efficiency (C) and pollutant removal efficiency (R) as a function of outlet height and outlet diameter (Vb/Vro=1.61): 1953 simulation results.

Outlet Diameter (ft)		Outlet Elevation (ft)					
		0	1	2	2.5	3	4
0.0	C	0.043	0.043	0.043	0.043	0.043	0.043
	R	0.043	0.043	0.043	0.043	0.043	0.043
0.25	C	0.887	0.845	0.775	0.721	0.635	*
	R	0.708	0.763	0.737	0.696	0.619	*
0.50	C	0.976	0.952	0.922	0.900	0.876	0.678
	R	0.293	0.641	0.737	0.753	0.757	0.625
1.00	C	1.000	1.000	1.000	1.000	0.995	0.951
	R	0.018	0.460	0.650	0.705	0.747	0.791
1.50	C	*	*	*	1.000	1.000	1.000
	R	*	*	*	0.690	0.734	0.798

* indicates that simulation was not run.

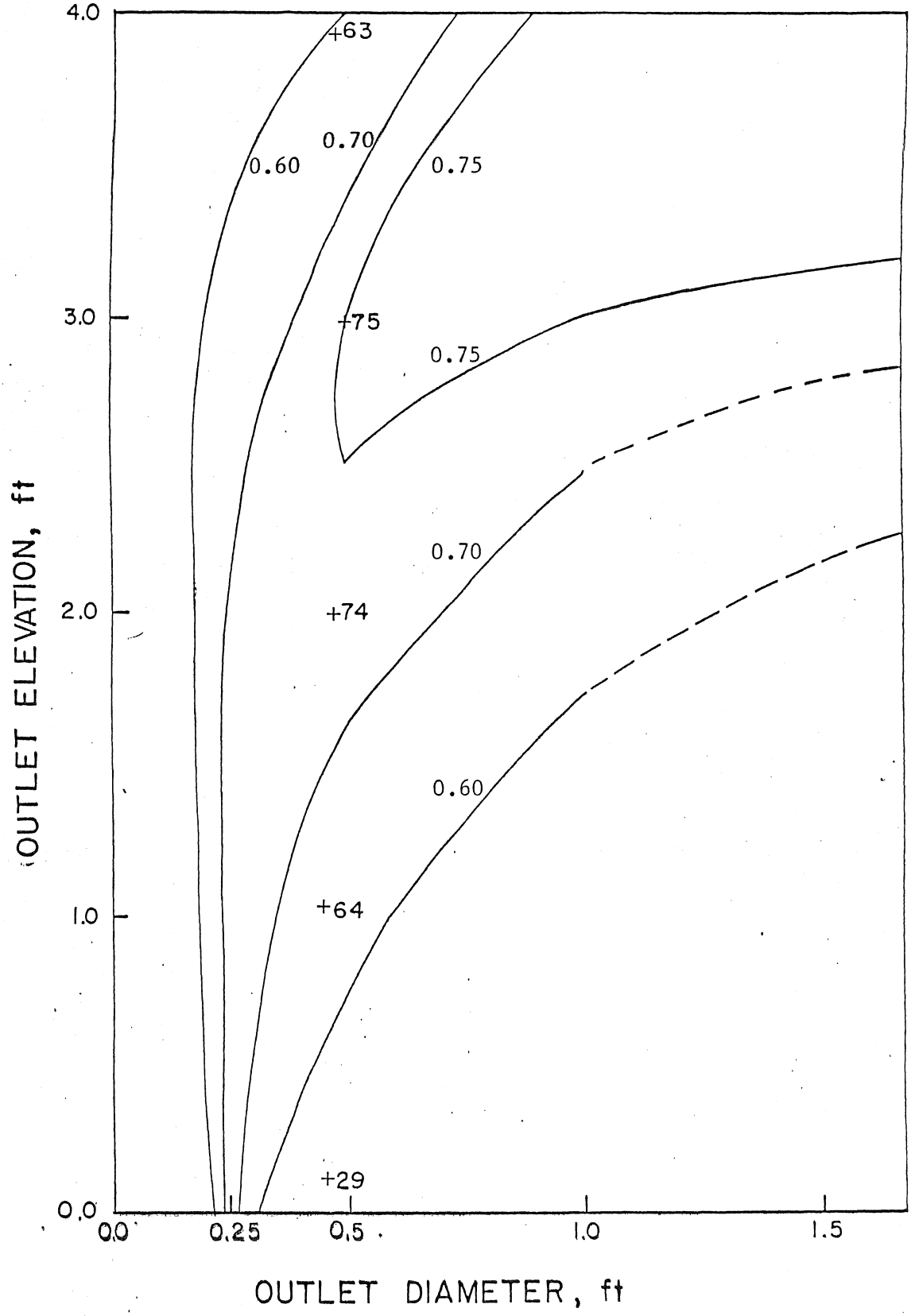


Figure 27. Solution surface of pollutant removal efficiency (R) as a function of outlet height and outlet diameter (Vb/Vro = 1.61): 1953 simulation results.

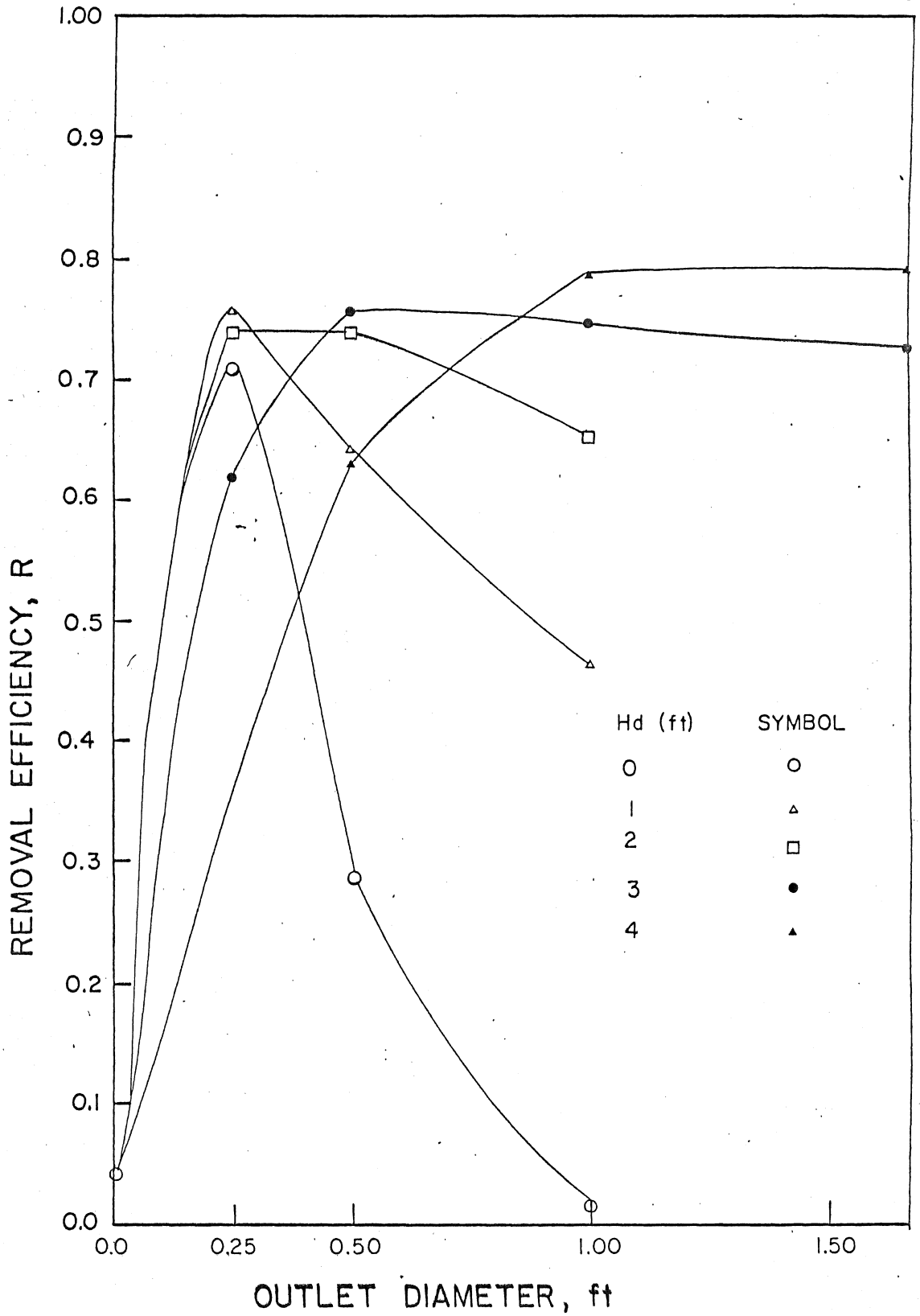


Figure 28. Pollutant removal efficiency (R) as a function of outlet height and outlet diameter (Vb/Vro = 1.61): 1953 simulation results.

The capture performance describes intuitive results. As the outlet height increases, the flow captured decreases. As the outlet diameter increases, the capture efficiency increases. The preliminary increase in removal efficiency as the outlet height increases (holding diameter constant) demonstrates the effect of dilution in a completely mixed basin. Capture efficiency decreases, but the remaining volume undergoes continuous pollutant decay, creating a sizeable dilution potential for the influent. For each outlet diameter, a unimodal removal curve was observed as the outlet height increased. This reflected the effect on removal efficiency of decreased capture performance combined with increased dilution.

As the outlet diameter increased for a constant outlet elevation, the capture efficiency increased. As observed before, the removal performance peaks and then decreases as the diameter increases, again reflecting the tradeoff of capture and treatment.

The maximum removal was observed at an outlet height of 4.0 feet and an outlet diameter of 1.5 feet. Even though the effective volume is small, the rate of discharge was so great that 100 percent of the year's runoff flows were captured.

This suggests that a basin with a large outlet, possibly a weir, offers the best capture and removal performance. This is a different case than a basin with no

outlet. With no outlet the maximum depth is maintained (minus evaporation , etc. losses) and any runoff is immediately bypassed with no treatment. With a large outlet below the maximum depth, the runoff will pass through the basin and receive some degree of treatment by dilution. Extrapolating these results to areas outside the solution surface, the limiting maximum removal configuration would be a drawdown rate as high as the maximum inflow rate, with the discharge height at the top of the basin, i.e. no bypass, but all the runoff would be diluted to some degree as it passed through the basin. These trends suggest that optimal quality control would be provided by a basin that is partially full of water to provide dilution. This conclusion is based on the supposition that pollutant removal follows the exponential removal curve and that there is complete mixing in the basin. The latter assumption is hardly appropriate for sedimentation.

The computer costs associated with the simulations are presented in Table 14.

STATISTICAL TECHNIQUES

Event Definition

A preliminary task in statistical analysis is the grouping of raw data into independent events. A commonly used method is the separation of hourly rainfall values by a minimum number of hours with no rain. Serial

Table 14. Computer costs of simulations.

Procedure	Cost * Dollars
Runoff Block	
24.6-year	3.25
One-year	0.75
Storage/Treatment Block	
24.6-year	6.25
One-year	0.60
Synop	
Interface	1.25
24.6-year runoff	1.75

* Average costs for low priority execution on the University of Florida system. Normal priority is approximately four times as much.

autocorrelation has been used to define this minimum interevent time (Medina 1976). Howard (1976), Hydroscience (1979) and others have suggested that rainfall events occur as a Poisson process, and the time between events is exponentially distributed. The exponential distribution is a special case of the gamma distribution with the coefficient of variation (standard deviation divided by the mean) equal to unity. A cumulative distribution of time between events was presented in Figure 6. The gamma function has been widely applied in hydrology (Haan 1977).

Traditional Design of Flood Control Basins

The traditional sizing of stormwater detention facilities has been based on the control of a single "design" storm event. These basins are designed to capture the runoff resulting from a storm expected to occur for a given duration on the average once every N years (Haan 1977). Typical values of N range from 1-50, although incorporation of downstream risk assessment governs the value of N. The typical design storm analysis employs ranking particular storms (e.g. annual maximum 60 minute and 24 hour rainfalls) over a time period and assigning an extreme value probability distribution to the resultant sequence. From this distribution, return periods are assigned to storms of given magnitudes and durations. Figure 29 depicts the relationship between the parent distribution and resulting extreme value distributions. Figure 30 presents a cumulative extreme probability distribution (normal) used to determine the return period of river flood flows. Because this method does not retain information on the time between events, there is a loss of information and the method is said to be inefficient in terms of the data (Haan 1977). One unobtainable parameter whose value is necessary for accurate flow routing is the effective volume, defined as the actual storage volume available at the beginning of a storm, which is a function of antecedent conditions. Some municipalities require that

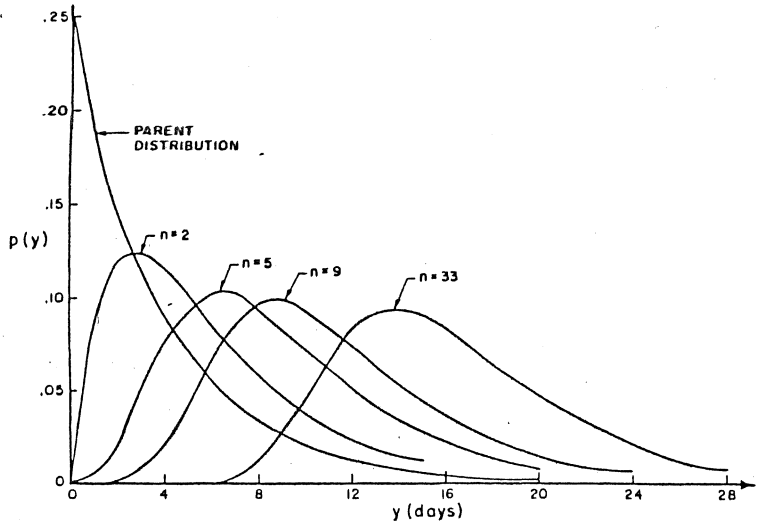


Figure 29. Distribution of the largest sample value from a sample size n from an exponential distribution.

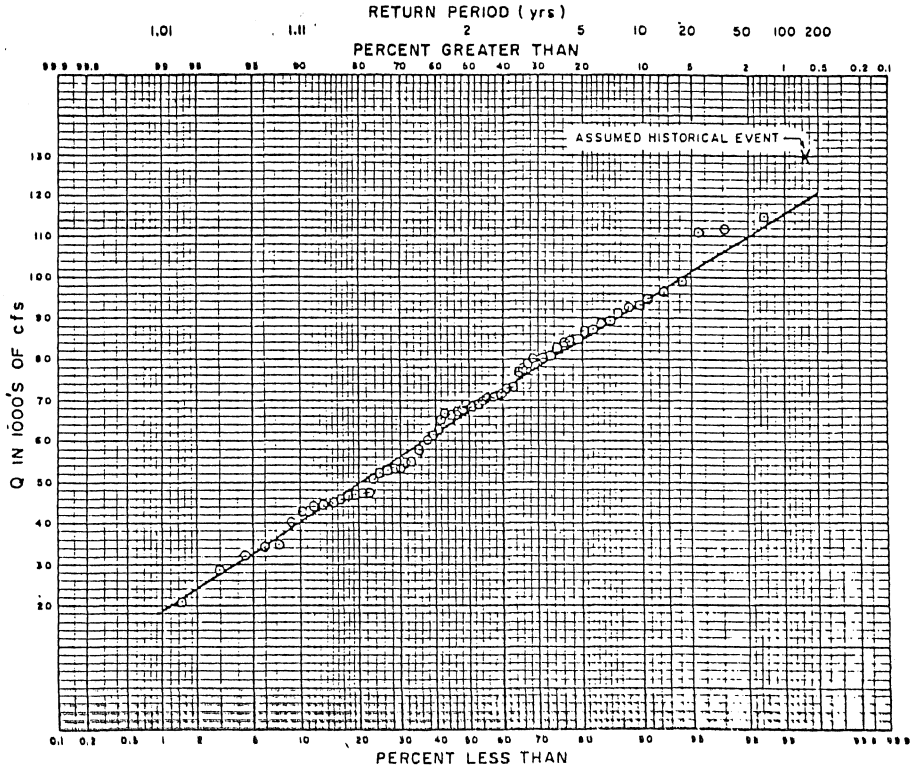


Figure 30. Normal probability plot of Kentucky River data.

ponds be built to contain the runoff from urbanized areas (subdivisions, parking lots, etc.) resulting from a design storm. The Rational Method, an extension of the design storm concept, is widely used for this purpose. The design storm method is relatively straightforward and does not require computer solution, hence it receives high ratings on ease of application, though as Haan (1977) points out, there is no direct theoretical connection between the solution and the underlying mechanisms governing the storm events. While this method is successfully used for flood control design, no criteria have been established for determining a design storm for quality control.

Aggregate Statistical Methods

Independently, two groups have developed statistical approaches for designing urban stormwater detention facilities. Howard (1976) presented the theoretical framework for analyzing the interevent times and volumes of combined sewer overflows resulting from various storage/treatment configurations. His derivation was based on approximating intensity, duration and interevent time as independent and exponentially distributed random variables.

As part of a study evaluating the long-term performance of stormwater control devices, Di Toro, et al. (1979) derived an analytical expression for the effective volume of a stormwater detention facility available at the beginning

of a storm event. Like Howard's method, this technique retains information on the time between events, an important parameter for flow routing analyses. The method is based on the SYNOP results for rainfall event statistics: intensity, depth, duration and interevent time; converting them to runoff values; setting up the respective equations for effective volume and solving them. The result is a set of graphs which enable the user to plot the solution surface for percent of flow captured (C) and effective volume (Ve), as a function of the constant drawdown rate (Qc) and the ratio of the empty basin volume (Vb) to the mean runoff volume (Vro). The algorithm requires repetitive application to obtain an optimal basin size for maximum capture efficiency. The method presents a straightforward procedure for a simple catchment, single basin system (Dever, 1980).

Data Input

The Atlanta rainfall event statistics were obtained from the previous runs of SYNOP. Runoff event parameters were obtained from the rainfall values via a linear conversion factor. The STORM equation was used to determine the volume conversion factor, based on the percent imperviousness for the catchment,

$$Cro = 0.15 + 0.75I \qquad (10)$$

where Cro is the runoff conversion factor, and

I is the percent of the catchment area that is impervious.

With I=0.37 for the catchment data, Cro=0.4275. The conversions produced a mean runoff volume of 19000 cubic feet based on the mean event depth of 0.212 inches over the catchment area of 24.7 acres. The mean event volume is similar to the value obtained by the SWMM simulation (18022 cubic feet). As mentioned earlier, the conversion method does not account for the reduction in the number of events resulting from catchment capture. This results in a total of 504 inches of runoff for the entire 24.6-year record, as compared with the 388 inches obtained by the Runoff Block.

Vb/Vro ratios were calculated to correspond to the volumes used in the simulations. The resulting ratios are presented in Table 15.

Table 15. Determination of normalized volume ratios.

Basin volume (cubic feet)	Normalized volume ratio Vb/Vro
7744	0.41
15014	0.79
29064	1.53
92420	3.74
167835	8.83

There was a misrepresentation of the normalized discharge ratio in Di Toro's work.

1. The product QcTi was expressed as the average drawdown between storms. The concept of average drawdown between storms implies that mass continuity be preserved. With no bypass, the average volume entering the basin is equal to the average runoff volume, and represents a limiting value of the average long-term drawdown between storms. The average drawdown would be even less if the average volume entering the basin is less than the average runoff volume, due to bypass. If the product of Qc and Ti were truly the average drawdown between storms, then the ratio (QcTi/Vro) would always be less than or equal to unity; yet relationships are developed for values up to infinity.
2. The product of Qc and Ti would only equal the average drawdown between storms if the time series was completey homogeneous, with mean event volumes occurring at mean interevent intervals. For all other time series:
   a. the product of Qc and Ti has no relationship to average values;
   b. the product of Qc and Ti is always greater than the true average drawdown between storms; and,

c. the ratio (QcTi/Vro) is not bounded by continuity constraints. This is why values greater than unity are realized.

The average drawdown concept developed by Hydroscience, Inc. is a plausible yet erroneous expression. The ratio QcTi/Vro is used in this study merely as a normalized discharge rate.

A solution surface of capture performance was prepared as a function of basin volume and pumping rate. The grid was created by using Figure 31 (see arrows) as follows:

1. enter the lower graph at the respective volume ratio (Vb/Vro);
2. move horizontally until intersecting with the normalized discharge curve (QcTi/Vro);
3. move to the upper graph at the effective volume ratio (Ve/Vro), the common side between the graphs;
4. continue up until intersecting the runoff volume coefficient of variation curve (cv); and
5. finally move horizontally and exit at the estimate of capture efficiency (C).

This process was repeated for 25 combinations of volume sizes and drawdown rates.

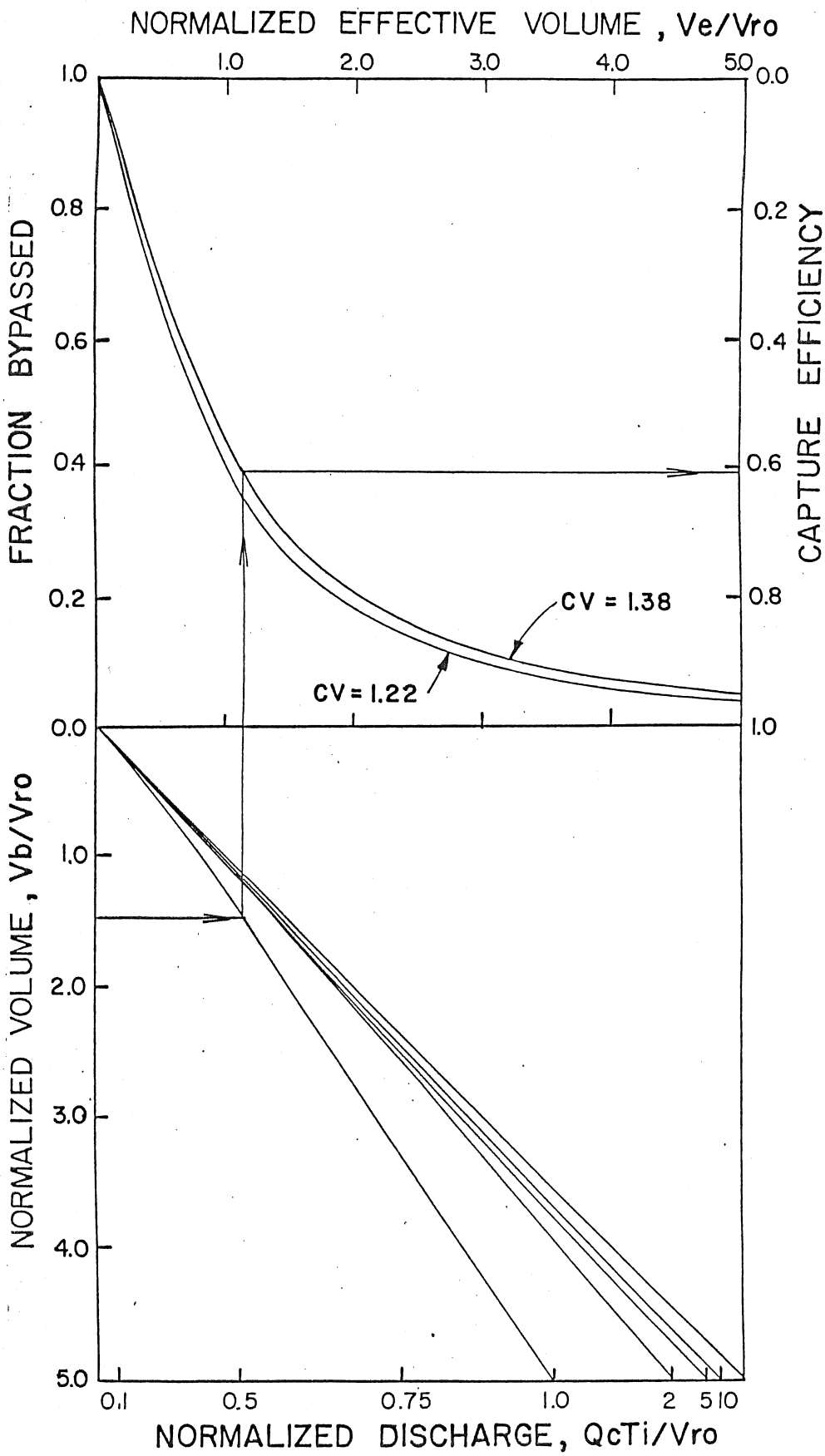


Figure 31. Relationship of capture efficiency (C) with normalized basin volume, normalized discharge rate and mean volume coefficient of variation.

Capture Performance Results

The results are presented in Figures 32 and 33 and Table 16.

Table 16. Estimates of flow capture efficiency (C) as a function of basin volume and constant discharge rate: statistical results.

Normalized Volume Ratio	Normalized Discharge Ratio 1	2	4	7	10
0.41	0.22	0.23	0.24	0.30	0.30
0.79	0.40	0.47	0.48	0.49	0.49
1.53	0.61	0.68	0.70	0.70	0.71
3.92	0.86	0.90	0.91	0.92	0.92
4.63	0.90	0.93	0.93	0.94	0.94

Isopleths of capture efficiency were drawn which emphasized the apparent insensitivitiy of the performance to varying pumping rates. This relationship exists due to the combined shapes of the upper and lower curves. The capture efficiency is most sensitive to the coefficient of variation and effective volume ratio (Ve/Vro) at the lower end of the Ve/Vro axis. However, at the lower end of the Ve/Vro axis, the effective volume ratio is relatively insensitive to the pumping rate, for the lines converge near a QcTi/Vro of 1.0. Conversely, where the effective volume is most sensitive to the pumping rate, at the upper end of the Vb/Vro axis, the percent capture is least sensitive to effective volume, for

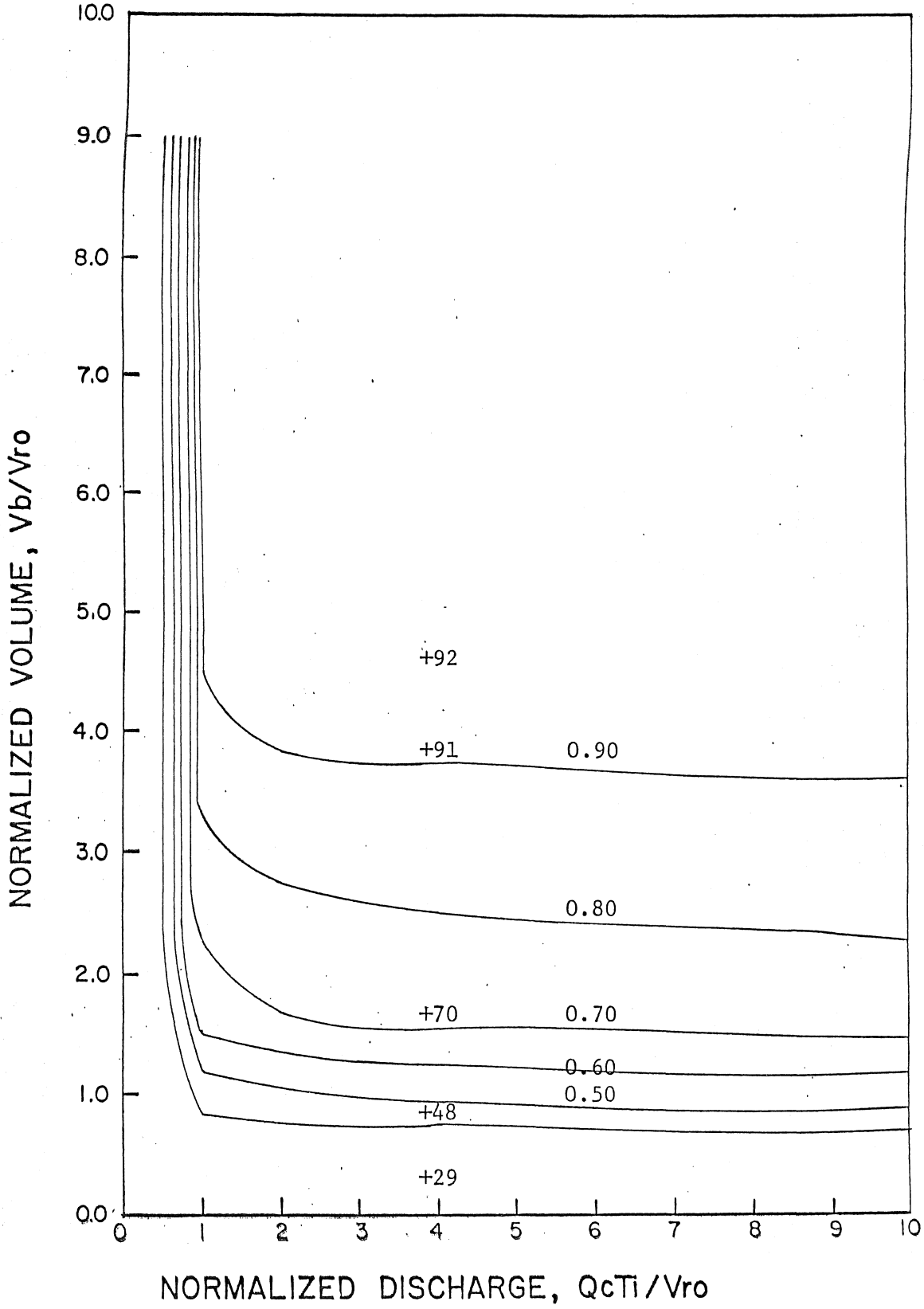


Figure 32. Solution surface of flow capture efficiency (C) as a function of basin volume and constant discharge rate: 1953 statistical results.

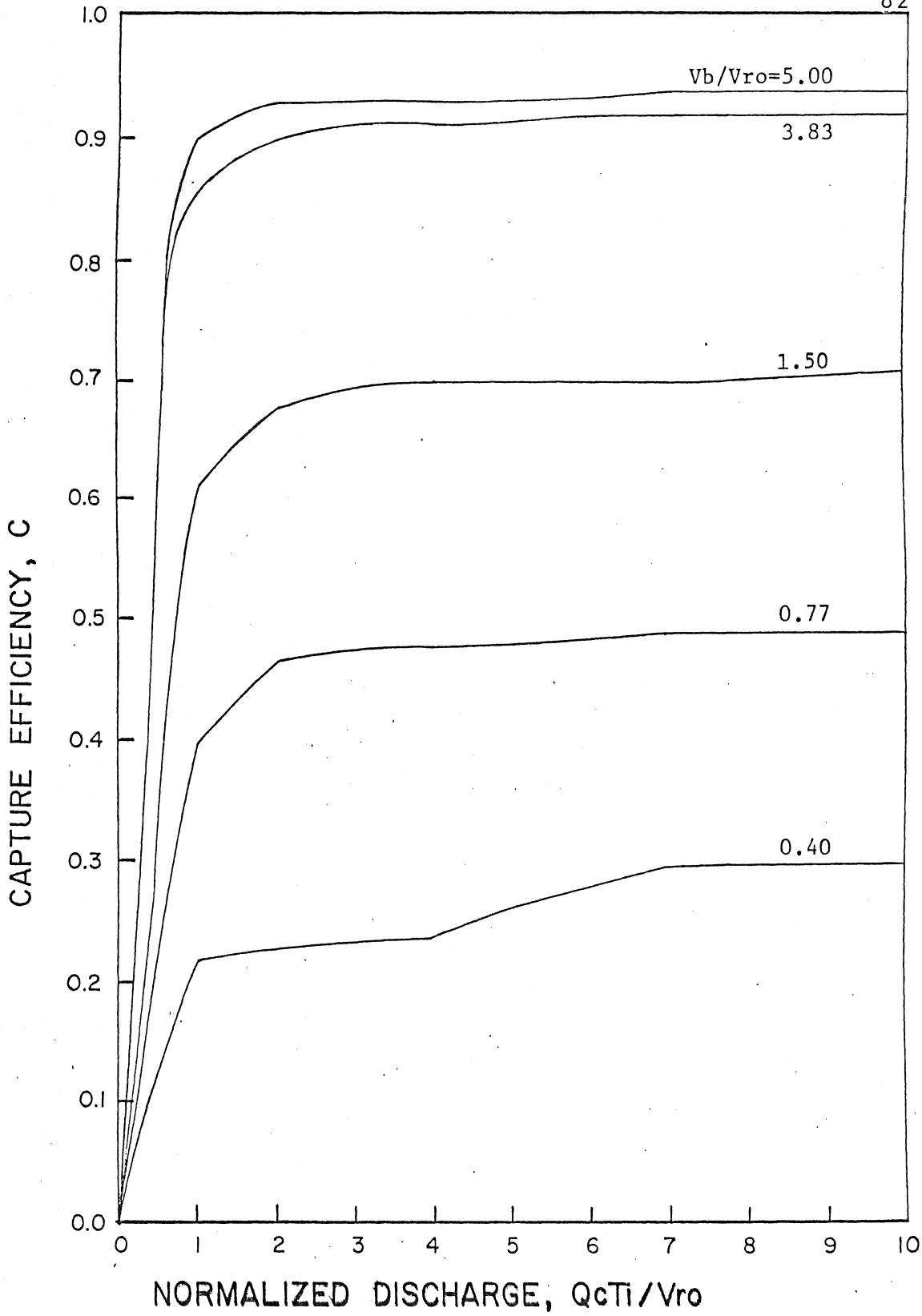


Figure 33. Flow capture efficienct as a function of basin volume and constant discharge rate: 1953 statistical results.

the slopes are the flattest. The greatest overall sensitivity lies in the "middle" region of the graphs. The maximum variation for any given volume was an 11 percent increase from 64 percent to 75 percent for a ratio of 1.6. The largest Vb/Vro ratio available in Figure 31 is 5.0. This limitation precluded a complete comparison with simulation results, where Vb/Vro ratios up to 9.3 were analyzed.

Removal

A major weakness of using the statistical method for estimating quality control is the inherent assumption of absolute pollutant removal efficiency. In the design of a detention facility for quality control, the determination of a removal efficiency is the primary objective. It was difficult, therefore, to assign a removal expression. It was tempting to use the constant removal value incorporated during each time step in the S/T simulation, however, the two terms are not conceptually equivalent. The statistical method essentially treats the removal mechanism in the basin as a black box in which the constant removal refers to the total captured flow during the complete time history. The simulation employs time steps to route the flow through the basin, and the constant removal term applies only to the volume within the basin during that time step. This implies that for any runoff volume which remains in the basin longer

than one time step, the total removal efficiency will be greater than the constant removal term.

To make an estimate of removal efficiency would be presupposing the solution in this study. However, an estimate of removal efficiency can be obtained by taking the product of the percent capture and an assigned constant removal percentage. The resulting solution surface will have the same shape as the capture performance presented in Figure 32, but the value of the isoquants will be altered by the removal factor.

Analysis Using Simulated Runoff Data

For further study, the statistical analysis was repeated on runoff data obtained from the SWMM simulation. The values determined by SYNOP for the data generated from the Runoff Block (see Table 5) of SWMM are compared to the converted values in Table 17.

Several differences were noted. The most obvious is that there are 24 percent more events modeled in the statistical method than the simulation. This difference affects the total amount of runoff predicted by the two methods. The simulation yields 308 inches of runoff while the use of a conversion factor yields 504 inches of runoff, or 30 percent more runoff over the 24.6-year period. This difference is not apparent from comparison of the estimates of mean runoff volume, where there is only a 5.47 percent difference in the

Table 17. Comparison of simulated runoff mean event statistics with rainfall conversion values.

	Simulated	Conversion
Number of events	1920	2381
Volume (in)	0.201	0.212
cv	1.222	1.384
Duration (hr)	5.066	7.824
cv	0.972	1.134
Intensity (in/hr)	0.043	0.032
cv	1.148	1.356
Interevent time (hr)	111.71	90.10
cv	1.000	1.004
Minimum interevent time (hr) to yield cv near 1.0	4	8

two methods. This is an interesting point. Because the statistical method deals with basin volumes normalized to the mean event volume, the 30 percent continuity difference between the methods is not reflected in the flow capture solution surface. This implies that satisfying the continuity equation may not be a requisite for a good solution methodology. An important difference is that the minimum interevent time was reduced from 8 hours to 4 hours to obtain a coefficient of variation for interevent time close to unity. At first, this appears to be due to the presence of an effective detention time inherent in the

catchment storage, characterized by the attenuation of magnitude and lengthening of duration. The values for the mean duration contradicts this thought, being two hours less than the rainfall mean. This decreased duration may be the result of round-off error in transferring runoff data into the NWS format for subsequent SYNOP runs. Flows less than 0.125 cubic feet per second are rounded down to 0.0 inches per hour, based on a catchment area of 24.7 acres. Volume ratios and pumping rates calculated from these new values are identical to those used in SWMM, as the same SYNOP run was the basis for those values. The performance determination was repeated on these new volume ratios and pumping rates. The results are summarized in Table 18.

Table 18. Estimates of flow capture efficiency (C) as a function of basin volume and discharge rate: statistical results with simulated runoff means.

Normalized Volume Ratio	Normalized Discharge Ratio				
	1	2	4	7	10
0.43	0.28	0.28	0.29	0.29	0.30
0.83	0.44	0.49	0.50	0.51	0.51
1.61	0.64	0.70	0.73	0.74	0.75
4.13	0.89	0.93	0.93	0.94	0.94
5.00	0.92	0.95	0.95	0.96	0.96

Because of the low sensitivity of flow capture efficiency to small changes in either volume or drawdown ratio, the solution surface obtained is almost identical to Figure 32.

The only computer costs associated with the statistical technique were for the SYNOP runs, which averaged $6.50 for rainfall and $1.75 for runoff data.

COMPARISON

In comparing the methods, it is necessary to recognize that the statistical, empirical and analytical techniques are first-cut approaches, while the simulation yields a greater design and analysis flexibility, although requiring a larger data input. As emphasized earlier, there was no established data base to definitively compare the results obtained. However, certain aspects of the statistical and simulation methodologies can be compared. Both techniques utilize the same rainfall data. The simulation generates its own runoff data, while the statistical technique relies on a conversion factor. With no established data base, there was no way to say whether the simulated runoff values are more accurate than the statistical conversions. However, previous implementations of SWMM which tested predicted runoff with documented values reinforce the assumption that the simulation results are accurate. Both methods allow calibration of results with observed data when

present. Mean event runoff values obtained from the two methods differed by only 5 percent, but the different number of events (2381 for the statistical method versus 1920 for the simulation) resulted in the statistical method predicting 30 percent more total runoff over the 24.6 year record.

Figure 34 is a comparison of the flow capture solution surfaces obtained from the simulation and statistical methods. For dischage ratios less than 1.0 the two methods are in reasonable agreement. However, at discharge rates above this ratio the estimates diverge, with the statistical estimates predicting up to 20 percent less capture than the simulation results. Figure 35 is a comparison of the estimates of removal efficiencies obtained from the simulation and statistical methods, where r is the constant removal term employed by the statistical technique. The most striking difference is the shape of the isoquants. The simulation estimates reflect the combination of flow bypass and treatment time in the slopes of the isoquants. The ability to depict this fundamental property of storage/treatment devices is relinquished by the statistical method with the direct inclusion of a removal expression, r. Estimates of r yielding comparable removal as the simulation results ranged from near unity at discharge ratios less than 1.0 to less than 0.8 at higher drawdown rates.

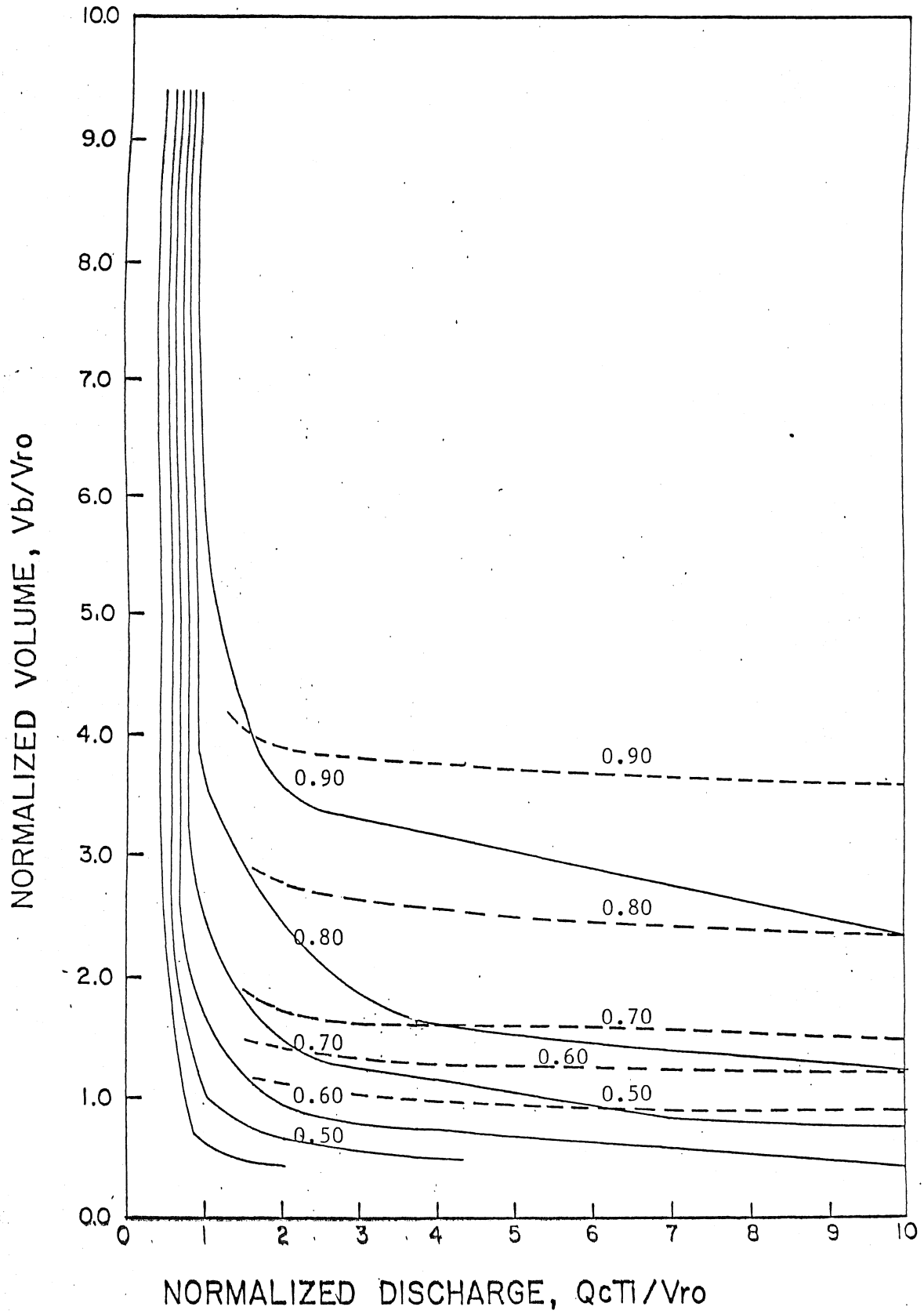


Figure 34. Comparison of flow capture efficiency.

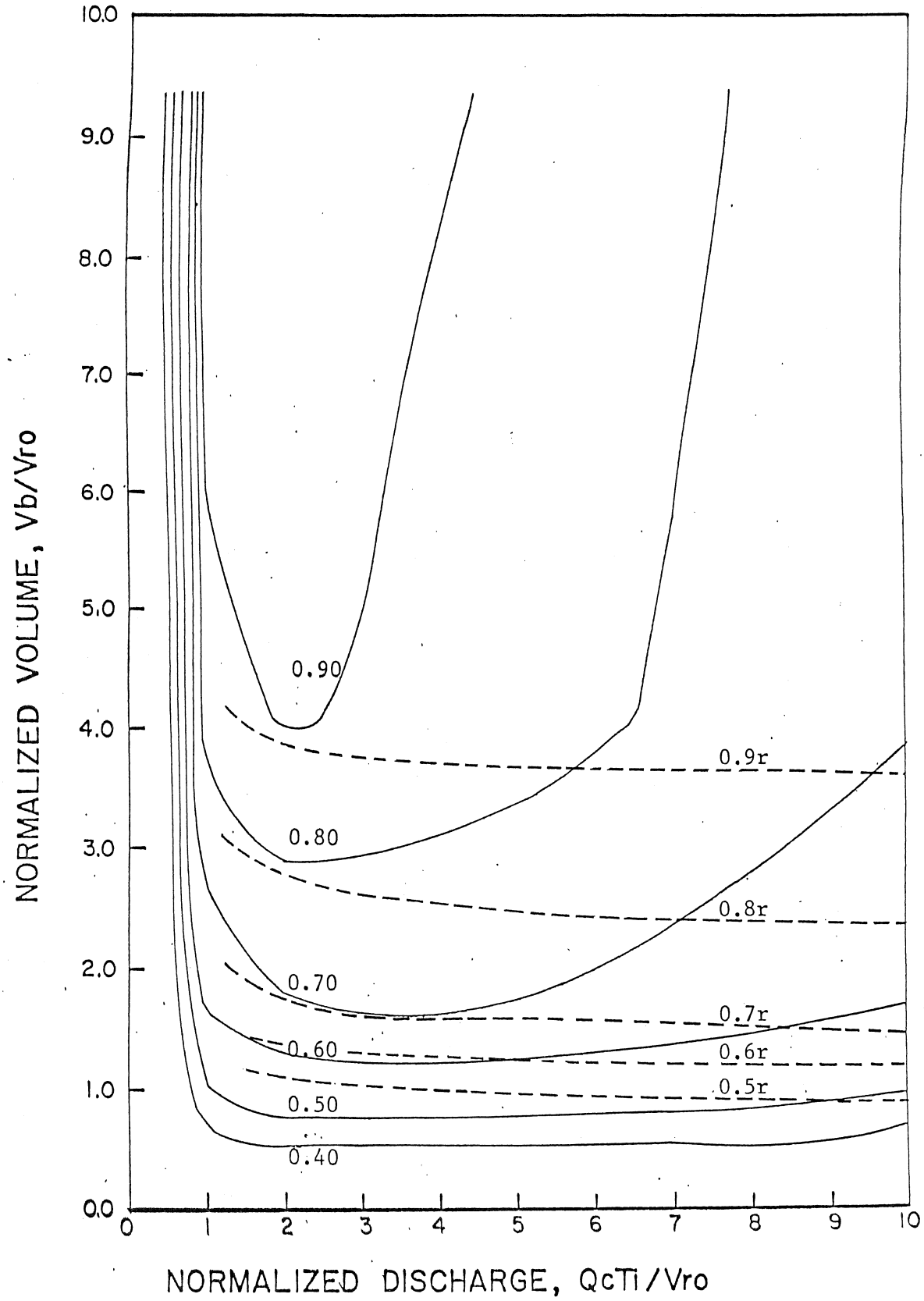


Figure 35. Comparison of pollutant removal efficiency.

DISCUSSION

This analysis was concerned with comparing solution methodologies. Because there are no actual data to obtain removal characteristics or calibrate the results, the results are not meant to stand alone, but are a function of the assumptions involved. It was felt that empirical and analytical methods were not applicable for determining long-term performance of stormwater detention facilities. Application of analytical methods to stormwater runoff events may be obtained with computer simulation (Medina 1980). Continuous simulation was found to provide the most detailed performance analysis. SWMM can handle complicated flow networks and S/T combinations, and give detailed summaries for any time period. The simulation provided more interesting (i.e. no explicit relationship between capture and removal performance) relationships between the overall removal efficiency and the basin parameters. There was a noticeable removal performance tradeoff between increasing the effective volume at the beginning of a storm and increasing the pollutant treatment time. These relationships were closer to observed results than the constant removal results of the statistical method (Heaney

1975). The simulation permits a sensitivity analysis of basin parameters and pumping strategies, while there is limited mechanism for this in the statistical technique. This feature allows the simulation to be used to search for the optimal basin characteristics for pollutant removal.

The relative effectiveness of the statistical technique in estimating flow capture performance for the hypothetical catchment was demonstrated by comparison with the results from the simulation. Conversion of rainfall volume was within 5 percent of the simulated results, although total volume of runoff over the 24.6 year record was 30 percent higher in the statistical method. The statistical method in the form utilized precludes determination of pollutant removal efficiency. Modifications are needed to account for variable removal efficiency within the detention facility.

GENERAL APPLICATIONS OF METHODOLOGIES

A spectrum of hydrologic control units is depicted in Figures 1 and 2. An example of these units and estimates of their volumes and detention times associated with a single rainfall event are enumerated in Table 19.

Both hydrologic storages and transport elements are classified as control units. While flows of water are not generally recognized as control elements, they possess the same characteristics necessary for pollutant removal, e.g. even though a detention facility is stationary, the volume of water passing through it is not. The hydrologic and functional characteristics alluded to in Figure 3 are inherent in all water bodies, whether it is a river reach which has a detention time of less than a day or a water supply reservoir with a detention time on the order of months. The classification of a hydrologic unit as a storage or flow is based on a relative time scale; a flow element turns over "faster" than a storage device. This time dependence dictates the reaction kinetics and is the basis for recognizing a spectrum of water bodies in Table 19. In all cases, the actual mechanisms of removal are not altered, but rather, the inherent kinetics which determine

Table 19. Estimates of hydraulic volume and detention time of various control units associated with a single rainfall event.

	Percent of Event Volume	Detention Time
1. interception by above ground structures	0-5	0-7 days
2. the initial abstraction volume	0-5	0-3 hours
3. the transport through the soil via infiltration	30-60	5-12 hours
4. the overflow runoff	20-35	1-18 hours
5. the flow in gutters	10-30	1-18 hours
6. areas of depression storage in the basin	0-10	0-1 month
7. evapotranspiration	0-10	0-12 hours
8. the flow in stream channels and flood plains	10-30	0-9 days
9. detention facilities	0-15	0-10 days
10. ponds and lakes	10-35	0.1-1000 years
11. the ultimate receiving water; here we recognize an even larger system, either subsurface, oceanic or atmospheric.	5-20	1-10000 years

Source: modified from Medina 1976.

the extent of performance are changed. The hydrologic kinetics dictate the predominance of diffusive transport or turbulent advective transport (Rich 1974).

The omission of the time scale criteria in evaluating system performance presented difficulties in the early works

on lake eutrophication (Vollenwieder 1968, Dillon 1975, Reckhow 1978). The derivation of the first models began with the mass continuity equation but quickly veered to empirical concepts. Vollenwieder (1968) originally developed a general expression for the trophic state of lakes based on aerial loading and mean depth. The same empirical model was used for all water bodies, ranging from urban ponds with detention times on the order of days, to mammoth lakes with detention times of hundreds of years. The early predictive models were based on data gathered from a number of large, public lakes threatened with water quality problems. Later studies revealed a set of lakes whose trophic conditions were considerably different than predicted. In one study, Dillon (1975) compared the phosphorus concentrations in two lakes of similar depths. Their areal phosphorus loadings and, hence, their predicted phosphorus levels, differed by a factor of twenty. Due to the difference in detention times, 26 days versus 1738 days, the observed concentrations in the lakes were similar. It was recognized that detention time, which had been neglected in the early models, was a fundamental factor in the lake concentration of phosphorus. The influence of detention time was obscured in the earlier studies as a result of using a one year time frame on lakes with detention times greater than one year. In 1973, Vollenwieder incorporated the detention time concept in the trophic state model with

an expression of overflow rate, defined as the ratio of mean depth to detention time (Vollenwieder 1976). Other researchers intuitively retained detention time from the continuity equations and derived relationships similar to the ones which Vollenwieder and others ultimately arrived at (Reckhow 1978, Schnoor 1975).

It is possible to apply the solution methodologies analyized in this thesis to problems of a similar nature. The solution process should follow the same framework as was presented:

1. identify the system;
   a. unit's place in system;
   b. influencing processes;
2. identify major flows and storages; and,
3. recognize the time dependence on removal mechanisms i.e. detention time, which affects the performance of the reaction kinetics.

APPENDIX A. PROGRAM LISTING AND DATA INPUT

Table A-1 Runoff Block Simulation

```
//SWMM  JOB (2006,3400,100,20,0),'GARY F. GOFORTH',CLASS=1,REGION=512K,
//  MSGLEVEL=(1,1)
/*PASSWORD     2,RAIN
/*ROUTE  PRINT REMOTE6
// EXEC FORTXCLE,LPARM='LIST,NOMAP,OVLY',OPTIONS='NOSOURCE,NOMAP'
//FORT.SYSIN DD *
      SUBROUTINE COMBIN
      RETURN
      END
      SUBROUTINE TRANS
      RETURN
      END
      SUBROUTINE RECEIV
      RETURN
      END
      SUBROUTINE EXTRAN
      RETURN
      END
      SUBROUTINE STRT
      RETURN
      END
/*INCLUDE RHYDRO1
/*INCLUDE RHYDRO2
/*INCLUDE RHYDRO3
/*INCLUDE GUTTER1
/*INCLUDE GUTTER2
/*
//LKED.SYSLMOD DD SPACE=(CYL,(5,1,1))
//LKED.SYSUT1 DD SPACE=(CYL,(5,2))
//LKED.LIB DD DSN=UF.A0063473.SWMM,DISP=SHR
//LKED.SYSIN DD *
 INCLUDE LIB(MAIN)
 INCLUDE LIB(GRAPH,CURVE,PINE,PPLOT,SCALE,HYSTAT)
 OVERLAY  ALPHA
 INCLUDE   LIB(RUNOFF,RBDATA,CTRAIN)
 INSERT        ABLK,ERSON,NEW76,NEW77,NEW79,NEW80
 OVERLAY  BETA
 INCLUDE   LIB(HYDR2,GAMP,QSHED,WSHED)
 INCLUDE   LIB(MELT,AREAL,FINDSC,GQUAL,HCURVE)
 INSERT WSHED
 OVERLAY  BETA
 INCLUDE LIB(PRNTR3)
//GO.FT01F001 DD UNIT=SYSDA,SPACE=(TRK,(50,10)),
//    VOL=SER=WORK01,DCB=(RECFM=VBS,BLKSIZE=4240,BUFNO=1)
//GO.FT02F001 DD UNIT=SYSDA,SPACE=(TRK,(50,10)),
//    VOL=SER=WORK01,DCB=(RECFM=VBS,BLKSIZE=4240,BUFNO=1)
//GO.FT03F001 DD UNIT=SYSDA,SPACE=(TRK,(50,10)),
//    VOL=SER=WORK01,DCB=(RECFM=VBS,BLKSIZE=4240,BUFNO=1)
```

Table A-1 (cont.)

```
0048 //GO.FT04F001 DD SYSOUT=A,DCB=(RECFM=FSA,LRECL=133,BLKSIZE=133)
0049 //GO.FT08F001 DD UNIT=SYSDA,
0050 //   DSN=UF.A0063473.RAINA,SPACE=(TRK,(100,50),RLSE),
0051 // DISP=(NEW,CATLG),DCB=(RECFM=VBS,BLKSIZE=4240, BUFNO=1)
0052 //GO.FT09F001 DD UNIT=SYSDA,SPACE=(TRK,(50,10)),
0053 //    VOL=SER=WORK01,DCB=(RECFM=VBS,BLKSIZE=4240,BUFNO=1)
0054 //GO.FT10F001 DD UNIT=SYSDA,
0055 //   DCB=(RECFM=FB,LRECL=80,BLKSIZE=6160),
0056 //    DISP=SHR,DSN=UF.B0063400.S6.RAIN
0057 //GO.SYSIN DD *
0058   10   8   8   9   8   9   9   8   0   8   8   0   0   9   9   0
0059    1   2   3   4
0060 RUNOFF
0061 THIS IS A SIMULATION RUN OF TWENTY-FIVE YEARS OF HOURLY
0062 RAINFALL (ATLANTA AIRPORT) ON 10 HA THROUGH A BASIN.
0063     1        0 0 60.0     1 25.0     0          0 0.5     7     6    48     1 0
0064    31   12   72     090451ATLANTA, GEORGIA
0065
0066     1    1    1 103724.7 37.0 0.04 0.0130.25 0.0520.184 2.5 0.52 0.00115
0067
0068     0
0069     0    1
0070 ENDPROGRAM
0071 /*
END OF WORK FILE
```

```
0000 //SWMM  JOB (2006,3400,200,50,0),'GARY F. GOFORTH',CLASS=1,REGION=512K,
0001 //  MSGLEVEL=(1,1)
0002 /*PASSWORD     2,RAIN
0003 /*ROUTE  PRINT REMOTE6
0004 //. EXEC FORTXCLE,LPARM='LIST,MAP,OVLY',OPTIONS='NOSOURCE,NOMAP'
0005 //FORT.SYSIN DD *
0006       SUBROUTINE COMBIN
0007       RETURN
0008       END
0009       SUBROUTINE TRANS
0010       RETURN
0011       END
0012       SUBROUTINE RECEIV
0013       RETURN
0014       END
0015       SUBROUTINE EXTRAN
0016       RETURN
0017       END
0018       SUBROUTINE RUNOFF
0019       RETURN
0020       END
0021 /*INCLUDE UNITDNT*
0022 /*INCLUDE STRT&&&
0023 /*INCLUDE STRDAT*
0024 /*INCLUDE PLUGS*
0025 /*
0026 //LKED.SYSLMOD DD SPACE=(CYL,(5,1,1))
0027 //LKED.SYSUT1 DD SPACE=(CYL,(5,2))
0028 //LKED.LIB DD DSN=UF.A0063473.SWMM,DISP=SHR
0029 //LKED.SYSIN DD *
0030  INCLUDE LIB(MAIN)
0031  INCLUDE LIB(GRAPH,CURVE,PINE,PPLOT,SCALE,HYSTAT)
0032  OVERLAY  ALPHA
0033  INCLUDE LIB(CONTRX,STCOSX)
0034  INSERT  S1,S2
0035  OVERLAY  BETA
0036  INCLUDE LIB(EQUATX,INTERX)
0037 //GO.FT01F001 DD UNIT=SYSDA,SPACE=(TRK,(50,10)),
0038 //    VOL=SER=WORK01,DCB=(RECFM=VBS,BLKSIZE=4240,BUFNO=1)
0039 //GO.FT02F001 DD UNIT=SYSDA,SPACE=(TRK,(50,10)),
0040 //    VOL=SER=WORK01,DCB=(RECFM=VBS,BLKSIZE=4240,BUFNO=1)
0041 //GO.FT03F001 DD UNIT=SYSDA,SPACE=(TRK,(50,10)),
0042 //    VOL=SER=WORK01,DCB=(RECFM=VBS,BLKSIZE=4240,BUFNO=1)
0043 //GO.FT04F001 DD SYSOUT=A,DCB=(RECFM=FSA,LRECL=133,BLKSIZE=133)
0044 //GO.FT08F001 DD UNIT=SYSDA,SPACE=(TRK,(50,50)),
0045 //    VOL=SER=WORK01,DCB=(RECFM=VBS,BLKSIZE=4240,BUFNO=1)
0046 //GO.FT09F001 DD UNIT=SYSDA,SPACE=(TRK,(50,10)),
0047 //    VOL=SER=WORK01,DCB=(RECFM=VBS,BLKSIZE=4240,BUFNO=1)
0048 //GO.FT10F001 DD UNIT=SYSDA,
0049 //   DCB=(RECFM=VBS,BLKSIZE=4240,LRECL=80,BUFNO=1),
0050 //   DISP=SHR,DSN=UF.A0063473.RAINAA
0051 //GO.SYSIN DD *
0052    10    8   10    8   10    8   10    8   10    8   10    8   10    8   10    8
0053     1    2    3    4
```

```
0054 STORAGE
0055                          BASIN ONE
0056
0057 B1        0          1                                  1          1          0    24
0058 C1   530101        0.0          0          0          0
0059 D1      0.1        0.1        0.1        0.1        0.1        0.1        0.1
0060 D1      0.1        0.1        0.1        0.1
0061 E11
0062 F1       BASIN ONE
0063 F2 10000.0          1        100        100        100
0064 G1 1.0
0065 G2        0          0          1
0066 G2
0067 G3                       -1.666E-04
0068 G3                                   1.0       -1.0
0069 H1
0070 I1        1          2         -3
0071 I3      0.0       5000                  0
0072 I3      0.5       5304                  0
0073 I3      1.0       5616                  0
0074 I3     1.25       5775              3.151
0075 I3      1.5       5936              4.457
0076 I3      2.0       6264              6.303
0077 I3      2.5       6600              7.719
0078 I3      3.0       6944              8.913
0079 I3      3.5       7296              9.965
0080 I3      4.0       7656             10.912
0081 I3      4.5       8024             11.791
0082 I3
0083 I5      0.0        0.0    0.450     0.450        0.0
0084 I6 0.0
0085 /*INCLUDE G1
0086 ENDPROGRAM
0087 /*
END OF WORK FILE
```

```
//FLY JOB (2006,3400,100,5,0),FLY,CLASS=1,REGION=512K,
// MSGLEVEL=(1,1)
/*PASSWORD     2,RAIN
/*ROUTE PRINT TCP
// EXEC WATFIV
$JOB
      DIMENSION OUTFLW(24),FLOW(24),POLL(24)
      DIMENSION IFLOW(24)
      DIMENSION IDATE(24)
      DIMENSION ISAVE(10),NDIM(10)
      CHARACTER SOURCE*4(5),PNAME*4(2,2),PUNIT*4(2,2)
      CHARACTER TITLE1*4(40),TITLE2*4(40)
      LEND=721231
      LOCAT=090451
      NQSDUM=1
      READ(10) TITLE1
      READ(10) IDATEZ,ZZERO
      READ(10) TITLE2
      READ(10) (SOURCE(K),K=1,5),NSTEP,DELT,INLETS,NQSDUM,TAREA
      READ(10) (ISAVE(K),K=1,INLETS)
      READ(10) ((PNAME(K,KW),K=1,2),KW=1,NQSDUM)
      READ(10) ((PUNIT(K,KW),K=1,2),KW=1,NQSDUM)
      READ(10) (NDIM(K),K=1,NQSDUM),QCONV
   10 ITFLOW=0
      DO 250 J=1,24
      READ(10) ATIME1,IDATE(J),TIME1,OUTFLW(J),POLL(J)
C*********************   CONVERT FROM CFS TO HUNDREDTHS OF AN INCH   ****
C   FLOW(CFS) * 3600SEC * 100% * 12IN PER FOOT / ( 43560FT2 PER ACRE *
C   24.7 ACRES) = FLOW * 4.015124
  200 FLOW(J)=OUTFLW(J)*4.015124
C
C**********  CONVERTING TO INTEGER AND ROUNDING UP ( >0.5 )  ***********
      IFLOW(J)=IFIX(FLOW(J))
      DIFF=(FLOW(J)-FLOAT(IFLOW(J)))
      IF(DIFF.GE.0.5) IFLOW(J)=IFLOW(J)+1
C
      ITFLOW=ITFLOW+IFLOW(J)
  250 CONTINUE
      JDATE=IDATE(1)
      INT=JDATE/100
      INT1=INT*100
      INT2=JDATE-INT1
      IF(INT2.EQ.1)GO TO 280
      IF(ITFLOW.LT.1)GO TO 10
  280 WRITE(8,300) LOCAT,JDATE,(IFLOW(J),J=1,12),LOCAT,JDATE,(IFLOW(J),
     1J=13,24)
  300 FORMAT(2I6,'1',12I3,/,2I6,'2',12I3)
      IF (JDATE.LT.LEND) GO TO 10
  350 CONTINUE
      STOP
      END
$ENTRY
//GO.FT08F001 DD UNIT=SYSDA,
//   DSN=UF.B0063400.S6.RAINC,SPACE=(TRK,(125,10),RLSE),
//   DISP=(NEW,CATLG),DCB=(RECFM=FB,BLKSIZE=6160,LRECL=80)
//GO.FT10F001 DD UNIT=SYSDA,
//   DCB=(RECFM=VBS,LRECL=80,BLKSIZE=4240,BUFNO=1),
//   DISP=SHR,DSN=UF.A0063473.RAINA
/*EOJ
```

Table A-4 SYNOP Execution

```
0000 //SYNOP JOB (2006,3400,50,5,0),'GRIZZLY',CLASS=1
0001 /*PASSWORD
0002 /*ROUTE  PRINT REMOTE6
0003 // EXEC FORTXCLE,SUBLIB1='CIRCA.FORTLIB',LPARM='LIST,NOMAP'
0004 //FORT.SYSIN DD *
0005 /*
0006 //LKED.LIB DD DSN=UF.A0063473.SYNOP,DISP=SHR
0007 //LKED.SYSIN DD *
0008   INCLUDE LIB(MAIN)
0009   INCLUDE LIB(COAD2,SETIA,NUMER,IDATE,SHELL,SHELR,DAIDA)
0010 //GO.SYSIN DD *
0011  90451    004110
0012 //GO.FT10F001 DD UNIT=SYSDA,DISP=SHR,DSN=UF.B0063400.S6.RAINC,
0013 //  DCB=(RECFM=FB,LRECL=80,BLKSIZE=6160)
0014 //GO.FT11F001 DD UNIT=SYSDA,SPACE=(TRK,(20,10)),
0015 //  VOL=SER=WORK01,DCB=(RECFM=FB,LRECL=80,DEN=3,BLKSIZE=800)
0016 /*
END OF WORK FILE
```

APPENDIX B

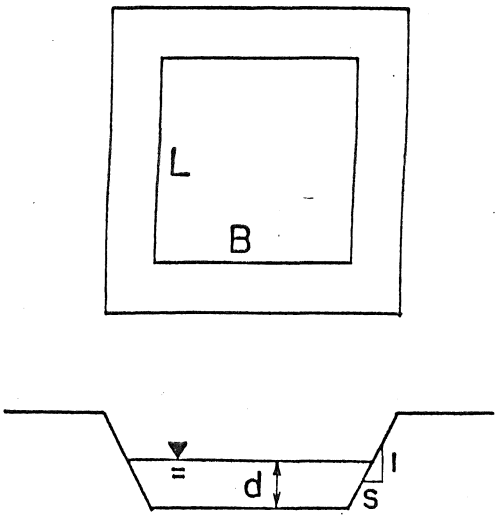
BASIN Development and Listing

BASIN is a small BASIC program written to generate hydraulic relationships for variety of basin geometries. This infomation is necessary as input to the Storage/Treatment Block of SWMM. The development of BASIN is depicted in Figure B1. The wetted surface area of a basin is a function of the basin side slope and the depth. The resulting stage-surface area relationship is integrated over the basin depth to yield the stage-volume relationship.

BASIN also calculates the stage-discharge relationship based on gravity drainage.

BASIN

```
 10 INPUT "WHAT ARE THE DEPTH, BASE, LENGTH, AND SLOPES?";D,B,L,
    S1,S2,S3,S4
 20 INPUT "WHAT ARE THE DIMENSIONS OF THE OUTLET DIAMETER AND
    HEIGHT?";PI,HT
 30 FOR D1=0 TO D
 40 VOL=B*D1*L+D1*D1*D1/3*(S1*S2+S2*S3+S3*S4+S4*S1)+
    D1*D1/2*(B*(S1+S3)+L*(S2+S4))
 50 SA=B*L+D1*D1*(S1*S2+S2*S3+S3*S4+S4*S1)+D1*(B*(S1+S3)+L*(S2+S4))
 60 IF (D1-HT)<0 GOTO 90
 70 Q=SQR(D1-HT)*PI*PI*6.3027
 80 GOTO 100
 90 Q=0
100 PRINT "FOR DEPTH=";D1 :  PRINT "THE SURFACE AREA=";SA :
    PRINT "AND THE VOLUME=";VOL
110 PRINT "THE OUTFLOW RATE (CFS)=";Q
120 D1=D1-.5
130 NEXT
140 END
```



$$SA = BL + k_1 d + k_2 d^2$$

$$k_1 = B(s_1 + s_3) + L(s_2 + s_4)$$

$$k_2 = s_1 s_2 + s_2 s_3 + s_3 s_4 + s_4 s_1$$

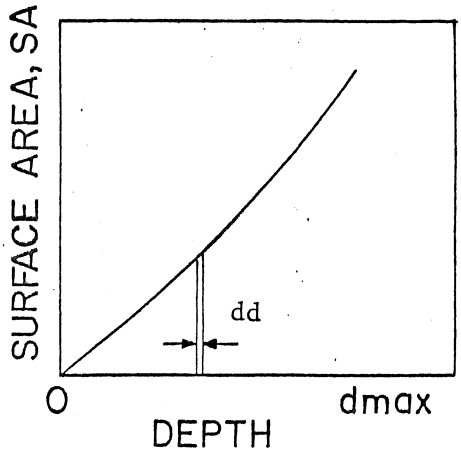
$$dV = SAdd$$

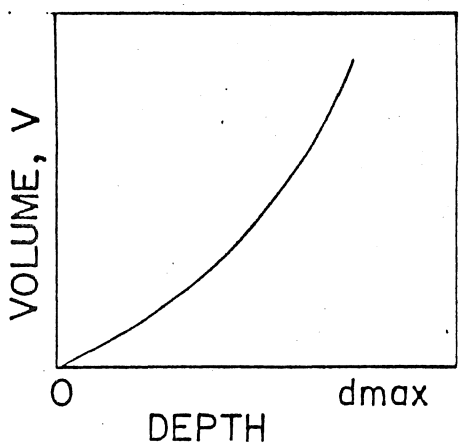
$$V = \int_o^{dmax} SAdd$$

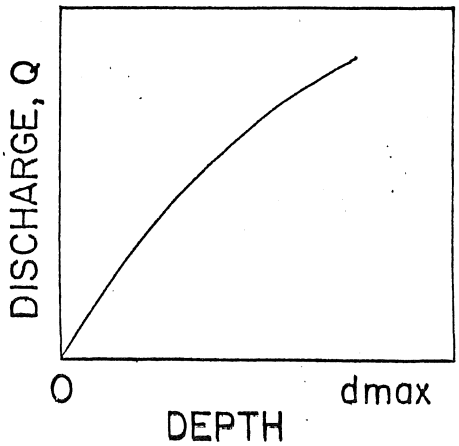
$$V = BLD + k_3 d^2 + k_4 d^3$$

$$k_3 = k_1/2$$

$$k_4 = k_2/3$$







$$Q = k_6 (d - d_0)^{1/2}$$

$$k_6 = (CE) SA_{outlet} (2g)^{1/2}$$

Figure B-1. Development of BASIN.

APPENDIX C

Detention Time

The concept of hydraulic detention time is widely applied in many areas of science and engineering. Typically, storage/treatment facilities are designed under steady state and uniform flow conditions. In practice, detention facilities present difficulties in estimating a hydraulic detention time due to characteristics which violate ideal assumptions. A simplification frequently employed in reservoir design and lake studies is averaging the flow over a relatively long time interval, usually a year, and assuming that the change in volume over the time interval is zero, implying a steady state condition. The flows and storages in urban stormwater detention facilities are characterized by a smaller (less than one year) time scale, due to quicker turnovers. As a result, there is a need to examine the flow routing process on a smaller time scale in order to analyze the treatment characteristics of the control unit.

Behind td=V/Q: Steady-State Conditions

Under uniform flow conditions, the velocity of water in a control unit is equal to

$$v=Q/A \tag{C-1}$$

where v is the velocity [L/T],

Q is the flow [L3 /T], and

A is the cross-sectional area of flow [L2].

The length of time (t) required to travel a distance (L) is

$$t=L/v \tag{C-2}$$

Assuming that the same steady and uniform flow conditions prevail in a detention basin, the travel time (detention time) is

$$td = L/v = L/(Q/A) = (LA)/Q = V/Q \tag{C-3}$$

where V is the basin volume in [L3]

Figure C-1 shows some common configurations of control units at steady state conditions. For a system composed of basins in series (C-1b), the total detention time is the sum of the individual basin detention times. For parallel basins (C-1c), the average detention time for the system is the sum of the product of the percentage of total flow traversing each circuit and the detention time of that circuit. In Figure C-1c, the percentage of flow passing through basin 1 is (a1Q/Q), and the detention time is

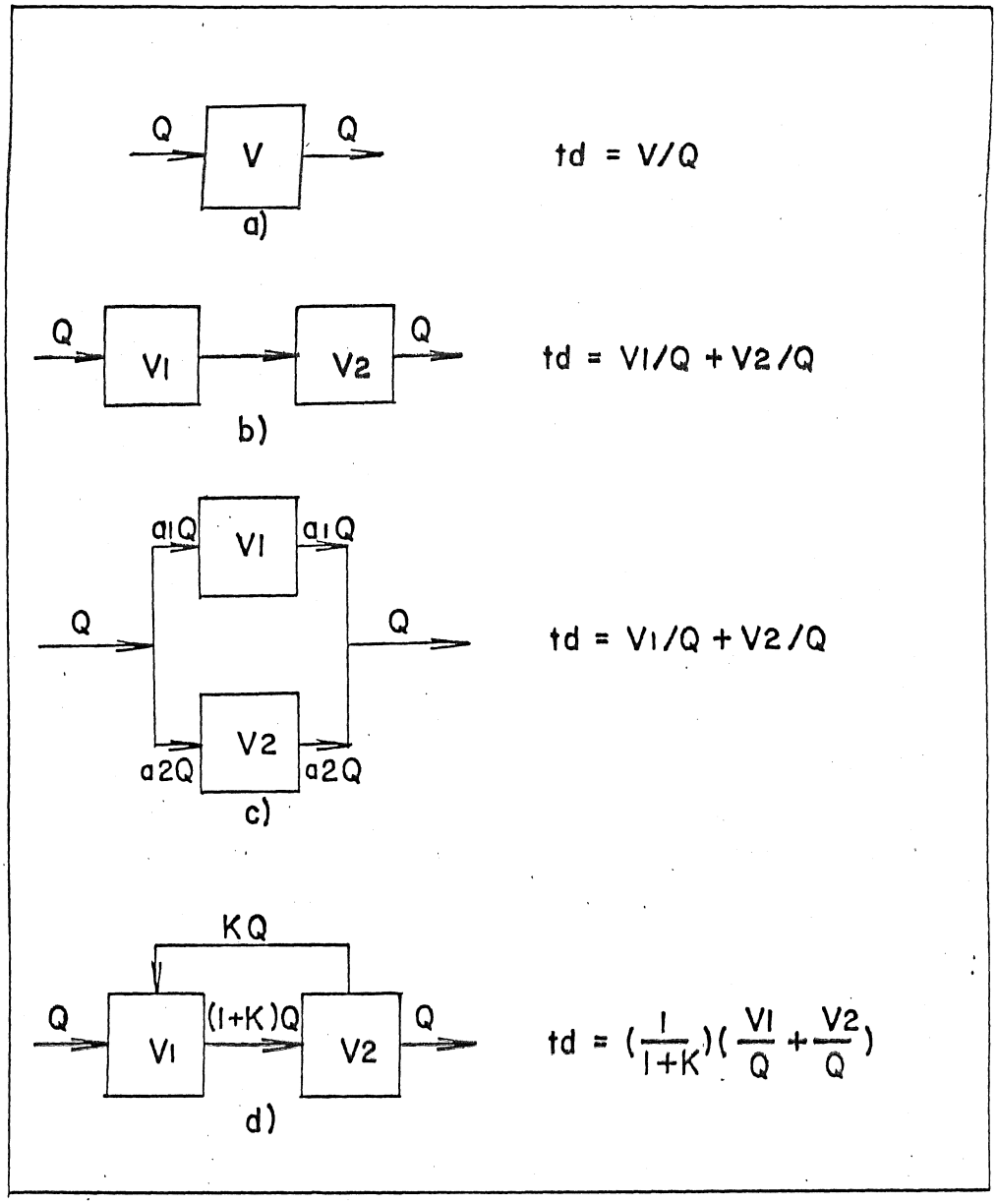


Figure C-1. Various control unit configurations: steady-state conditions.

(V1/a1Q). The contribution from 1 to the system detention time is therefore

$$(a1Q/Q)\ (V1/a1Q) = V1/Q \quad . \qquad \text{(C-4)}$$

Similar calculations for basin 2 yields the system detention time

$$td = V1/Q + V2/Q \quad . \qquad \text{(C-5)}$$

Expanding this concept to multiple tank parallel circuits results in

$$td = \text{summation from } i=1 \text{ to } n \ \ (Vi/Q) \quad . \qquad \text{(C-6)}$$

For a system containing a recycle loop, as in Figure C-1d, the detention time is equal to the flow weighted sum of the tanks' detention times and the recycle detention time. The percentage of flow passing through the system without recycle is

$$Q/(1+k)Q = 1/(1+k) \qquad \text{(C-7)}$$

where k is the recycle ratio, while the recycle percentage is

$$kQ/(1+k)Q = k/(1+k) \quad . \qquad \text{(C-8)}$$

The system detention time is therefore

$$td = (1/(1+k))(V1/Q+V2/Q) \quad . \qquad \text{(C-9)}$$

Notice that the flow in the recycle system receives less treatment time than the flow through basins in series. Hence, settling basins are rarely designed with recycle.

Analysis of Nonsteady-State Conditions

By employing finite difference techniques, equations for detention times of the systems in Figure C-1 may be developed for nonsteady-state conditions. Using the average volume during the time interval dt, the expression dv/dt by definition is zero. The flow passing through a tank is described by both an inflow and a outflow term. By defining the instantaneous flow as the mean of the inflow and outflow magnitudes, the average flow over a time interval can be determined as the mean instantaneous flow.

$$Q = (in + out)/2 \qquad Q(ave) = (Q \text{ at } t + Q \text{ at } t+dt)/2 \qquad (C-10)$$

Expanding this concept to average detention time,

$$td(ave) = V(ave)/Q(ave) \qquad (C-11)$$

Control unit configurations under nonsteady but linear flow conditions are presented in Figure C-2, where the effluent is proportional to the amount of storage. The detention time may be represented as

$$td = (2V) / (Q+kV) \qquad (C-12)$$

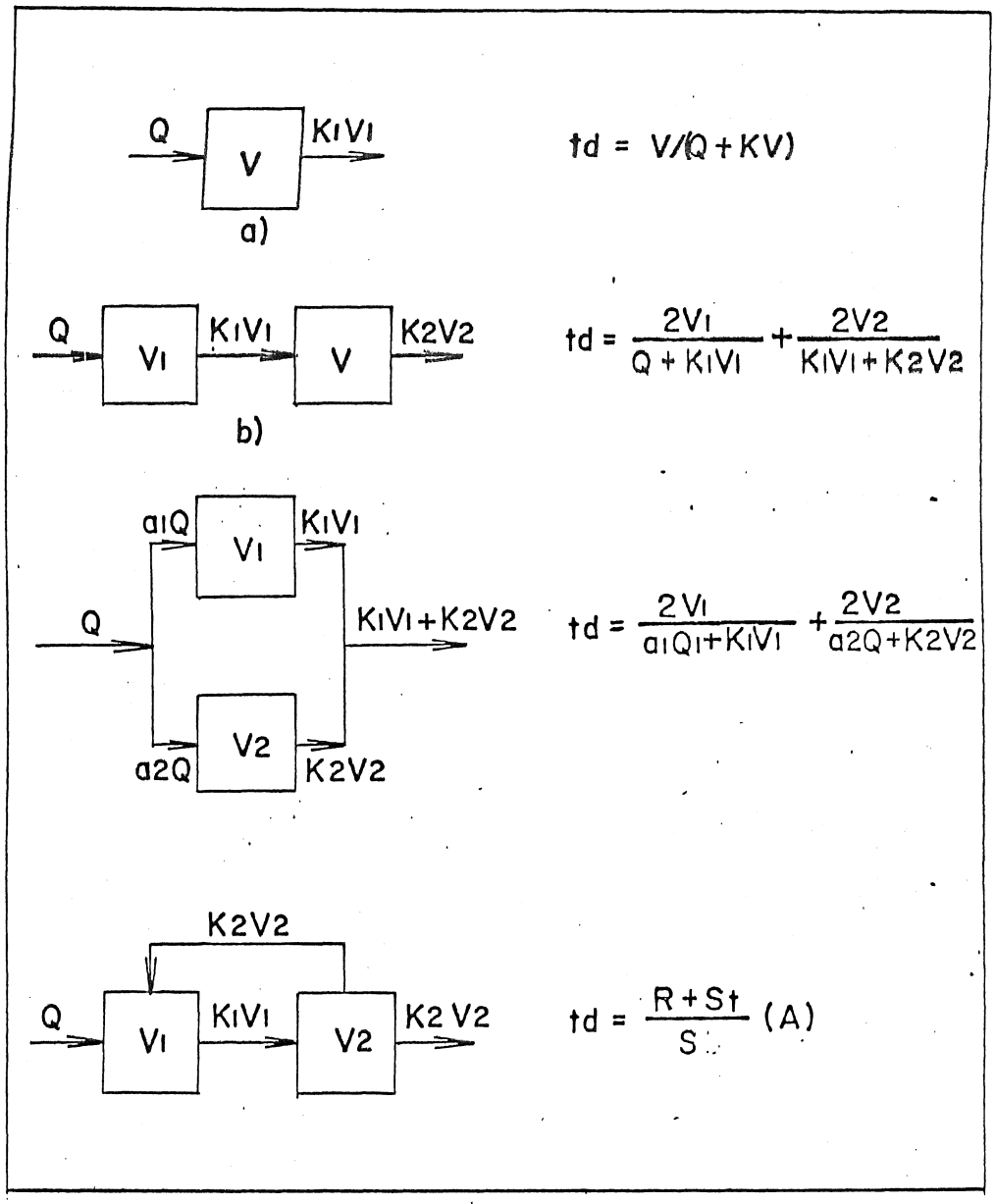


Figure C-2. Various control units configurations: nonsteady-state conditions.

For a series of linear units, the system detention time is equal to the sum of the components' detention times.

$$td = \text{summation from } i=1 \text{ to } n \left[(2V(i)) / (k(i-1)V(i-1)+k(i)V(i))\right] \quad \text{(C-13)}$$

For parallel tanks the detention time is

$$td = \text{summation from } i=1 \text{ to } n \left[(2a(i)V(i)) / (a(i)Q + k(i)V(i))\right] \quad \text{(C-14)}$$

For tanks with recycle, the flow recycled is proportional to the volume in basin 2. The amount flowing in the system (S) is

$$S = (Q+2k1V1+2k2V2+k3V2)/4 \quad \text{(C-15)}$$

The amount passing straight through the system (St) is (k3V2), and the recycle flow (R) is (k2V2). The detention time is

$$td = ((St+R)/S)(A) \quad \text{(C-16)}$$

where $A = ((2V1/(Q+k2V2+k1V1)) + (2V2)/(k1V1+k2V2+k3V2))$

It is emphasized that these equations will describe steady state systems and will reduce to the equations in Figure C-1 when inflows equal outflows. Also, these equations are equally valid for any system which can be described by similar (linear) kinetics.

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BIOGRAPHICAL SKETCH

Gary Frank Edmon Goforth was born on December 12, 1956, just outside the prison walls in Huntsville, Texas. He grew up in Texas and Indiana and has lived in Florida since 1966. He enrolled at the University of Florida in 1975, and received a Bachelor of Science in Engineering (Environmental Engineering) in 1979. He subsequently entered the Graduate School at the University of Florida, working on a Master of Engineering Degree in Environmental Resources Management.

He was a member of Tau Beta Pi and Kappa Phi Kappa honorary societies, the Water Pollution Control Federation, the American Water Works Association, and the Aircraft Owners and Pilots Association.

To support his academic habit, he worked throughout his undergraduate and graduate career. Beginning as a haberdasher, he has monitored the effects of thermal effluent from a nuclear power plant on a saltwater marsh, researched tertiary treatment of domestic waste in a freshwater marsh, and assessed the impact of urbanization on the water quality of an urban stream. He has coauthored several annual reports associated with these projects.

In 1980, he married a wonderful little Polish girl, Karen.