United States
Environmental Protection
Agency

Office of Water
Nonpoint Source Branch
Washington DC 20460

EPA440/5-87-001
September 1986

Water

EPA

Methodology for Analysis of Detention Basins for Control of Urban Runoff Quality

LIST OF TABLES

LIST OF FIGURES

The EPA Nationwide Urban Runoff Program (NURP) supported the preparation of this manual as well as the local studies that monitored the performance of the type of urban runoff control measures that are addressed herein. The contribution of the EPA Project Officer, Dennis N. Athayde, in supporting and encouraging the development of the analysis techniques described in this report was critical to the effort.

The basic probabilistic methodology that was adapted to the two specific control techniques for urban runoff addressed here, was conceived and formulated by Dr. Dominic M. DiToro (Manhattan College and HydroQual Inc.) and further developed by Dr. DiToro and Dr. Mitchell Small (currently Carnegie Mellon University). This basic groundwork was partly independent, and partly supported by EPA's NURP program and an earlier contract for which Mr. Athayde was also the Project Officer.

The adaptation of these basic probabilistic analyses to the specific urban runoff control measures addressed here, the analysis of the NURP program data, and the preparation of this report are the work of Eugene D. Driscoll (Woodward-Clyde Consultants). Dr. DiToro provided technical consultation, and David Gaboury (Woodward-Clyde Consultants) assisted in the analysis of settling velocities. Dr. Philip E. Shelley (EG & G) assisted in the analysis of the basic NURP program data.

The contribution of the following individuals and agencies, who provided feedback on the use of these techniques for local planning purposes, is also acknowledged.

EPA Headquarters	- Carl Meyers, James Meek, Patricia Bubar, Stuart Tuller, Norman Whalen
WASHCOG, Washington, D.C.	- Thomas Schuler
SEWRPC, Milwaukee, WI	- David Kendjiorski
State WRA, Annapolis, MD	- Bruce Harrington
State DNR, Raleigh, NC	- Robert Holman

The principal focus of EPA's Nationwide Urban Runoff Program (NURP) was to develop and transfer information that would be of practical utility to planning agencies in determining the need for, and approaches to the control of, pollutant discharges from urban stormwater runoff. One of the specific objectives was to assess the performance characteristics of control techniques, and for those indicated to be feasible candidates, to provide data and analysis procedures to guide and support planning decisions.

This report describes an analysis methodology and presents graphs and example computations to guide planning level evaluations and design decisions on two techniques for urban runoff quality control. The control techniques addressed, recharge or infiltration devices, and wet pond detention devices (basins that maintain a permanent pool of water), were shown by the NURP studies to be the most consistently effective at pollutant reduction of any of the Best Management Practice (BMP) approaches considered.

The underlying theory and mathematical computations are relatively sophisticated, but the application procedures have been reduced to a number of simple, easy to use steps that do not require expertise in mathematics or statistics. The time required to perform an analysis is quite short, so that the relatively large number of alternatives that should be examined for a planning level analysis can be readily made with a very nominal investment in time and resources.

A condensed summary of the technical details of the analysis methodology is presented in Section 2. Those interested in the theoretical development are referred to the sources cited for this aspect. The fundamental equations have been solved for the range of values the controlling parameters can assume, and are summarized in a series of easy-to-use graphs. These graphs are used in the manual computations of performance. Computer programs in BASIC programming language, which execute efficiently on personal (micro-) computers, have been developed. Interested parties should contact the EPA Project Officer.

The actual performance data developed by the NURP program have been summarized in the NURP Program Final Report (December 1984), along with an analysis of cost effectiveness and an illustration of these procedures for a general planning analysis for a region. Such material is repeated here only to the extent that it supports the objective of this report to describe, illustrate, and validate the analysis procedure.

1.0
INTRODUCTION

1.1 GENERAL

Best Management Practices (BMPs) receive consideration for control of nonpoint source pollutant discharges (in this case, urban runoff) because of the favorable influence they are expected to exert on receiving water quality by reducing the mass loading of pollutants that would otherwise be carried into such waters by storm runoff. Studies conducted under the NURP program indicated detention and retention basins to be the most effective and reliable of the techniques examined for control of urban runoff pollutant loads. The principal mechanisms that influenced pollutant removals were either subsurface infiltration, or sedimentation.

A detention device installed at a specific location is necessarily of a fixed size and capacity. Storm runoff, on the other hand, is highly variable. Any installation, therefore, will exhibit variable performance characteristics, depending on the size of the storm being processed, and in general, will perform more poorly for the larger storms than for the smaller ones. When performance is influenced significantly by the storage volume available, results obtained will be modified by residual stormwater from prior events that still occupies the basin when the next event occurs. Since storm intervals are variable, this factor frequently has a significant influence on performance. For detention devices such as wet ponds, which maintain a permanent pool of water, there is a further complication to the ability to describe performance. For many storms in all basins, and for virtually all storms in large basins, the effluent displaced during a particular event represents, in fact, a volume contributed to by the runoff of some antecedent event.

The performance of any control device that treats urban runoff should therefore be characterized in such a way that the variability and intermittent nature of storm runoff is recognized and accounted for. It is also desirable that the analysis procedures used provide a basis for making reasonable projections of performance under conditions other than those tested. An obvious alternative set of conditions relates to the effect on pollutant removal of basins of different sizes; however, the important factors include performance over all storms for an area in contrast to those monitored in a test program, and performance in areas where storm patterns are different.

The methodology presented in this report is based on a probabilistic technique that accounts for the inherent variability of the situation it addresses. The analysis has a planning orientation rather than a research one, consistent with the principal focus of the NURP program. The basic objective of the analysis that has been structured is to provide a basis for establishing "first order" design specifications (size, detention time), in terms of a long-term average removal of urban runoff pollutants. A secondary objective for a useful planning tool is that it be sufficiently simple, fast, and economical to apply, so that a large number of alternative scenarios are practical to examine. The methodology presented meets both these requirements, and by comparison with actual performance data and/or projections from more elaborate simulation models, is indicated to provide sufficiently accurate performance projections for the intended purposes.

There are other analysis methods available that can accomplish the same objective. EPA's Storm Water Management Model (SWMM), and the Storage, Treatment, Overflow Runoff Model (STORM) are both well documented simulation techniques that have seen extensive use. They have, in fact, been used in some of the validation tests of the probabilistic method, where adequate performance data were not available for comparison. Since these simulators can avoid several of the simplifying assumptions of the probabilistic approach, the estimates they provide are likely to be somewhat more accurate projections. The only real restriction to their use is a practical one. The user must have convenient access to a computer on which the program is installed, and preferably experience in the use of the programs.

Although other approaches are available to a user, the methodology presented in this report is believed to have several advantages. It permits an analysis to be performed without the need for access to a computer. Analyses are simple enough to perform that there is no practical constraint to examining a large number of alternative conditions of interest. These factors and the organization of the computations (input requirements and output format) emphasize the utility for planning purposes.

1.2 ORGANIZATION OF REPORT

Section 2 describes the probabilistic methodology and discusses the rationale and use of the performance graphs, and the equations on which they are based.

Section 3 addresses recharge devices and presents a description of the methodology, an example problem, validation tests, and a discussion of the application of the methodology and some limitations and practical considerations.

Section 4 addresses wet pond detention basins using the same format.

Section 5 presents results of a series of analyses using the methodology, illustrating differences in size/performance relationships as influenced by regional differences in rainfall characteristics. These generalized results may be used as an initial screening indication, to be further refined by use of specific local parameters in the analysis.

An Appendix provides information to assist the user in estimating values for parameters used in the methodology.

2.1 GENERAL

Performance estimates for the stormwater control devices addressed in this report are computed using probabilistic analysis procedures conceived and formulated by DiToro, and developed by DiToro and Small (2,3,4). These procedures provide a direct solution for the long term average removal of stormwater and pollutants for several different modes of operation of a control technique. The variable nature of storm runoff is treated by specifying the rainfall and the runoff it produces in probabilistic terms, established by an appropriate analysis of a long-term precipitation record for an area.

Long-term average reduction in mass loading is considered an appropriate measure of performance for several reasons. It recognizes the highly variable nature of storm runoff, which for a basin of fixed size, will result in higher removal efficiencies during some storm events and lower efficiencies in others. In addition, characterizing basin performance in this manner provides a direct tie-in with the methods adopted by NURP for characterizing the intermittent and variable impacts of storm runoff on water quality and for evaluating significance in terms of protectiveness or impairment of beneficial uses.

For assessing performance, the specification of the size or design capacity of a control device is often ambiguous, because the rate and volume of individual storm runoff events vary so greatly. This is influenced by regional differences in rainfall patterns, by the size of the drainage area the device serves, and by the land use distribution of this area, which determines the degree of impervious cover and the amount of runoff that any particular storm generates. For the procedures used in this report, variable rainfall/runoff rates, volumes, durations, and intensities are specified as a MEAN and COEFFICIENT of VARIATION (CV = STANDARD DEVIATION / MEAN). A meaningful measure of device size or capacity is then the ratio of its volume or flow capacity to the volume or flow rate for the MEAN storm runoff event. This permits a convenient generalization of the analyses performed and allows results to be readily applied to various combinations of local conditions.

Analysis procedures for computing size-performance relationships for three operational modes are presented in this section. A particular stormwater control device may incorporate one or more of these modes. Estimating performance for specific devices (for which examples are presented in later sections of the report) requires selecting and combining the procedures for the modes that are appropriate, or adapting the procedures to the specific circumstances dictated by the nature of the device.

2.2 RAINFALL

A long-term record of hourly precipitation data, available from the U.S. Weather Service for many locations, may be separated into a sequence of discrete storm "events" for each of which volume, duration, average intensity, and interval since the preceding event can be readily determined. The full set of values for each of these parameters may then be statistically analyzed to determine the mean and standard deviation, as well as the probability distribution of the set of all values for a parameter. A NURP publication (1) documents a computer program (SYNOP) that computes these statistics (and other information) from a USWS hourly precipitation record.

Appendix Section 2 provides a tabulated summary of storm statistics for gages in various parts of the country, developed from analysis of rain gage data by the SYNOP program. Appendix Section 3 presents information for estimating runoff coefficient. This information is provided to assist the user in estimating appropriate values for local analyses.

Analysis of a number of rainfall records indicates that the storm parameters that are used in the analyses described in this report are well represented by a gamma distribution. This distribution has accordingly been incorporated in the probabilistic analysis procedures described in this report.

2.3 FLOW - CAPTURE

This procedure addresses the condition where a device captures 100% of all applied flows, up to its capacity QT, and bypasses all flows in excess of this. No consideration is given to what happens to the "captured" fraction, other than that it no longer discharges with the uncontrolled fraction. Some examples include the following: in a Combined Sewer Overflow situation, the amount of the total wet weather flow that is carried away from the overflow point by an interceptor sewer and conveyed to a downstream sewage treatment plant can be considered to have been "captured," or removed from the overflows that would otherwise occur. A recharge device that diverts a portion of the runoff by causing it to percolate into the ground has captured some fraction of the surface runoff that would otherwise completely flow into a surface water body.

Whether or not further consideration must be given to the storm runoff so captured is not addressed here. The technique simply determines the long-term average reduction (or capture) in stormwater volumes processed by the device, and the pollutant loads associated with them.

For storm flows that are gamma distributed, and a device that captures all inflows up to a rate, QT, the long-term fraction not captured is given (3) by :

$$f_{FC} = \frac{r_1^{r_1} \; e^{-r_1}}{G(r_1)} \int_0^{\infty} E \cdot \left[E + \frac{QT}{QR} \right]^{r_1 - 1} \exp\left[-r_1 E\right] dE \quad (1)$$

where:

f_{FC} = fraction not removed by Flow-Capture device

r_1 = $1/CV^2$ (reciprocal of square of CV of runoff flows)

$G(r_1)$ = Gamma function for r_1

E = q/QR - QT/QR

q = runoff flow rate for an event

QR = mean storm runoff flow rate

QT = flow rate capacity of device

Transformed for numerical integration by Laguerre quadrature, this performance equation becomes:

$$f_{FC} = \frac{r_1^{r_1 - 2} \; e^{-r_1 (QT/QR)}}{G(r_1)} \sum_{j=1}^{n} w_j \; f(x_j) \qquad (2)$$

where:

$f(x_j)$ = $x_j \left(x_j / r_1 + QT/QR \right)^{r_1 - 1}$

x_j , w_j = abcissas and weights for Laguerre quadrature

This equation has been solved for a range of values for normalized treatment capacity (QT/QR), and variability of storm runoff flows (CV_q). Results are presented in Figure 1 which illustrates the effect of the above variables on long-term control efficiency of a device with this mode of operation.

2.4 FLOW - TREATMENT

This procedure addresses the performance of a device under variable input flows, when the treatment or removal efficiency for a pollutant varies with the rate of applied flow. It differs from

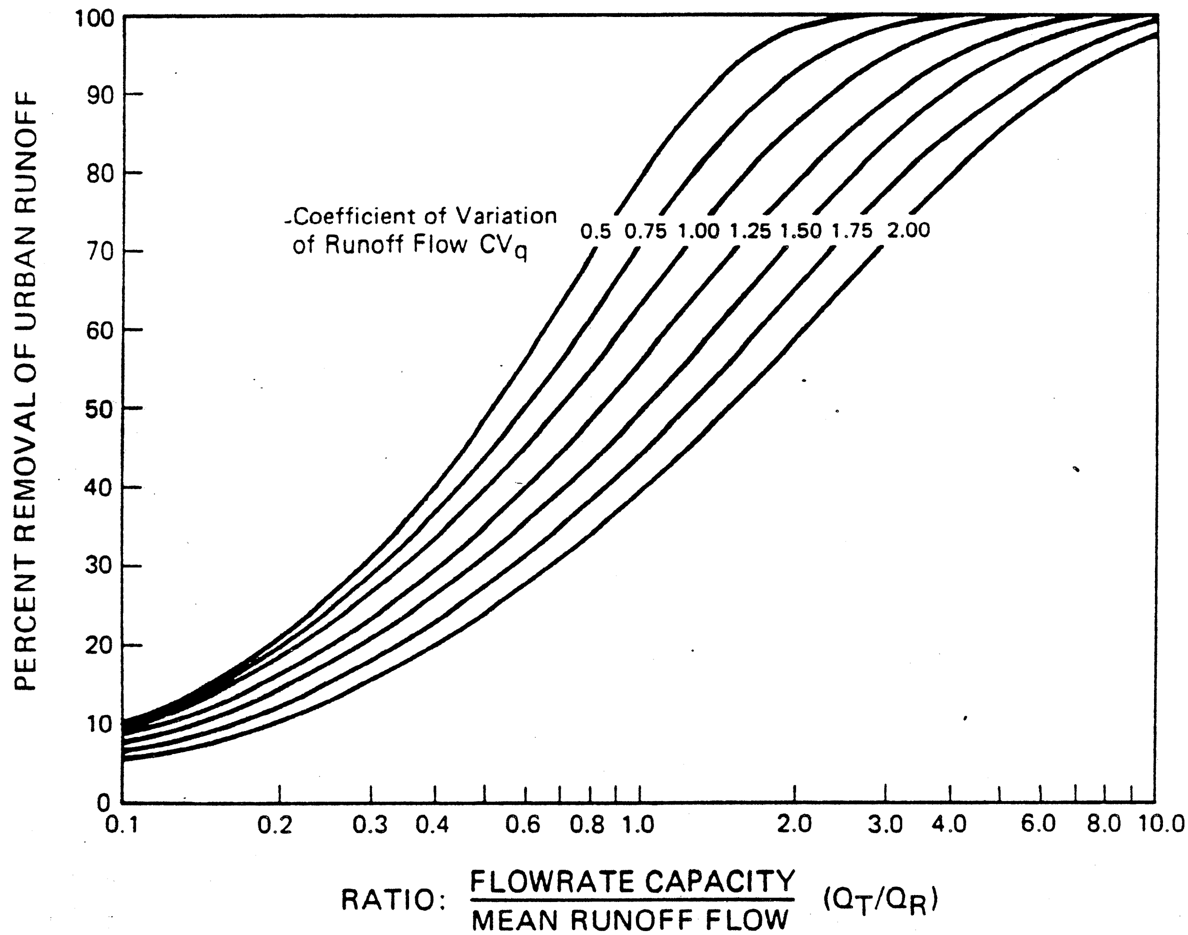


Figure 1. Average long term performance:
flow-capture device

the previous case in that the entire runoff flow is processed. An example would be a sedimentation basin which is less efficient at higher flow-through rates than it is at lower ones.

For variable runoff flows entering a treatment device that are gamma distributed and characterized by a mean flow and coefficient of variation (CV_q), the long-term average fraction of total mass removed is:

$$R_L = Z \left[\frac{r}{r - \ln\left[\frac{R_M}{Z}\right]} \right]^{r+1} \qquad (3)$$

where:

R_L = long term average fraction removed

R_M = fraction removed at mean runoff rate

r = $1/CV^2$ (reciprocal of square of CV_q)

CV_q = coefficient of variation of runoff flow rates

Z = maximum fraction removed at very low rates

A graphic solution to this equation is presented by Figure 2 and illustrates the effect on long-term performance caused by variability of stormwater flows. The analysis assumes that removal efficiency of the device is an exponential function of flow, thus:

$$\text{FRACTION REMOVED} = 1 - \exp(Q/k) \qquad (4)$$

While not exact, this relationship appears to approximate many removal relationships adequately, and is appropriate for a planning level analysis.

2.5 VOLUME - CAPTURE

This procedure addresses devices whose effectiveness is a function of the storage volume provided. This mode of operation is illustrated by a basin that captures runoff flows until it is filled and thereafter passes (untreated) all additional stormwater. The captured stormwater runoff is then removed from the basin in some manner once runoff ceases, in preparation for the next event.

The analysis does not consider what happens to the captured volume; it simply assumes it to be removed from the total discharge processed by the device. Off-line detention basins for

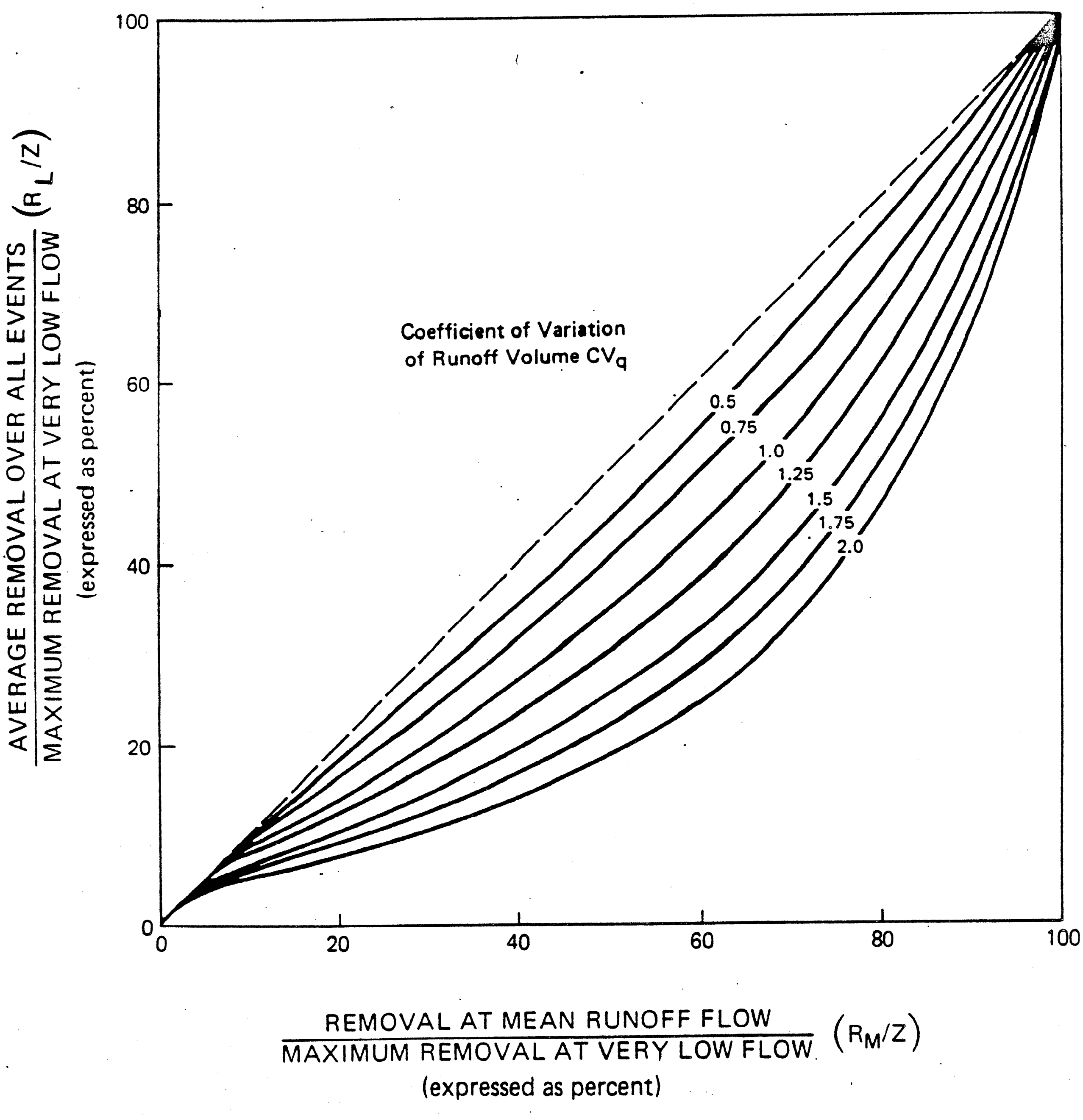


Figure 2. Long term performance of a device where removal mechanism is sensitive to flow rate

CSOs, which pump captured overflows back to the sewer system for processing at the treatment facility, provide one example of this mode of operation. Another example is a recharge basin, which (in addition to operating as a Flow-Capture device, Section 2.3) removes captured runoff volumes through percolation.

For storm volumes that are gamma distributed, the fraction not captured, over all storms, is:

$$f_v = \frac{r_1^{r_1} \; r_2^{r_2}}{G(r_1)\, G(r_2)} \int_{q=0}^{\infty} q^{r_1} \exp\left[\frac{r_2 V}{q}\right] \exp\left[-r_1 q\right] \int_{\Delta=0}^{\infty} \Delta \left[\Delta + \frac{V}{q}\right]^{r_2 - 1} \exp\left[-r_2 \Delta\right] d\Delta \, dq \qquad (5)$$

where:

r_1	=	$1 / CV_q^2$ and $r_2 = 1 / CV_d^2$
CV_q	=	coefficient of variation of runoff flow rates
CV_d	=	coefficient of variation of runoff durations
q	=	storm runoff flow rate
Δ	=	average interval between storm midpoints
V	=	basin effective volume, divided by mean storm runoff volume (VE / VR)
f_V	=	fraction of all volumes NOT captured by basin

The double integral cannot be evaluated analytically. A numerical technique using a Laguerre quadrature to approximate the integral with a weighted polynomial is applied. The basic equation transformed for solution using quadratures is:

$$f_v = \frac{r_1^{r_1} \; r_2^{r_2}}{G(r_1)\, G(r_2)} \sum_{k=1}^{n} w_k \, g[x_k] \left[\sum_{j=1}^{n} w_j \, f[x_j, x_k] \right] \qquad (6)$$

where:

$$g(x_k) = \left[\frac{x_k}{r_1}\right]^{r_1} \left[\frac{1}{r_1}\right] \cdot \exp\left[-r_1\ r_2\ V/X_k\right]$$

$$f(x_j, x_k) = \left[\frac{x_j}{r_2}\right] \left[\frac{1}{r_2}\right] \left[\frac{x_j}{r_2} + \frac{r_1 V}{x_k}\right]^{r_2 - 1}$$

n = number of orders used in integration

X_j, X_k, W_j, W_k = abcissas and weights for Laguerre Integration (from any handbook of mathematical functions)

This integral has been solved for a range of values of V (=VE/VR) and values for coefficient of variation in a range typically observed for rainfall/runoff. Results are plotted in Figure 3, which may be used instead of the equation.

From this figure, the average long-term performance of a volume device may be estimated based on the basin volume relative to the mean storm volume and the variability of individual event volumes being processed. However, the relationship is based on "effective" basin volume (VE) which may be quite different than the physical storage volume of the basin (VB). In the original CSO application, DiToro and Small (3,4) present a procedure for approximating the effective volume, based on an emptying rate ratio (E):

$$E = \frac{\Delta\ \Omega}{VR} \tag{7}$$

where:

Δ = average interval between storms (hours)

Ω = rate at which basin empties (cu ft / hour)

$\Delta\Omega$ = volume removed between storms, on average (cu ft)

VR = runoff volume from mean storm (cu ft)

The effect of the emptying rate ratio on the fraction of physical basin volume which is effective is described by Figure 4. As indicated, in cases where the volume which can be removed in the average interval between storms is small relative to the storm volume which enters on average, much of the available volume may be occupied with carryover from prior storms each time it rains. In such cases, effective volume may be considerably smaller than the physical storage volume provided.

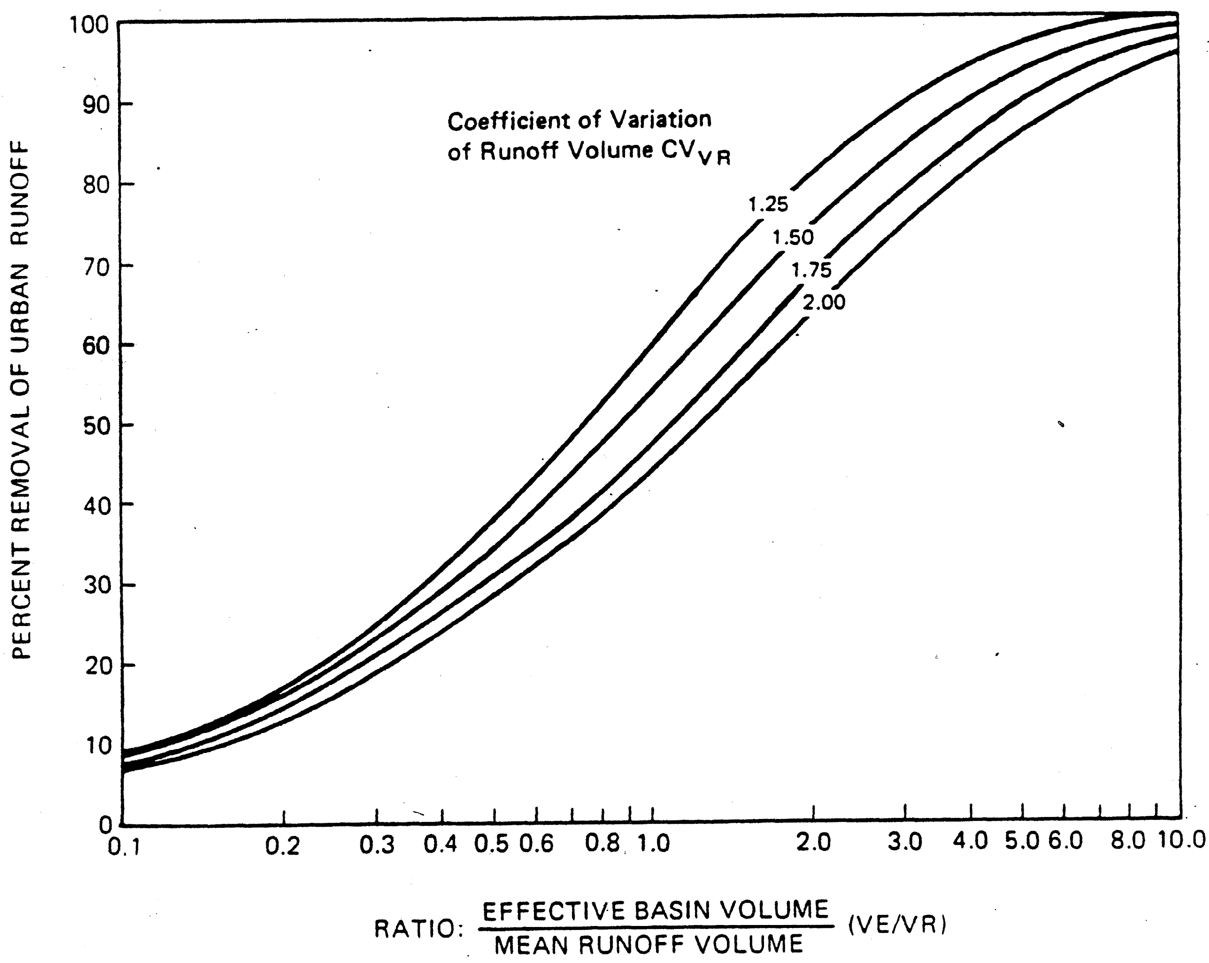


Figure 3. Average long term performance:
volume device

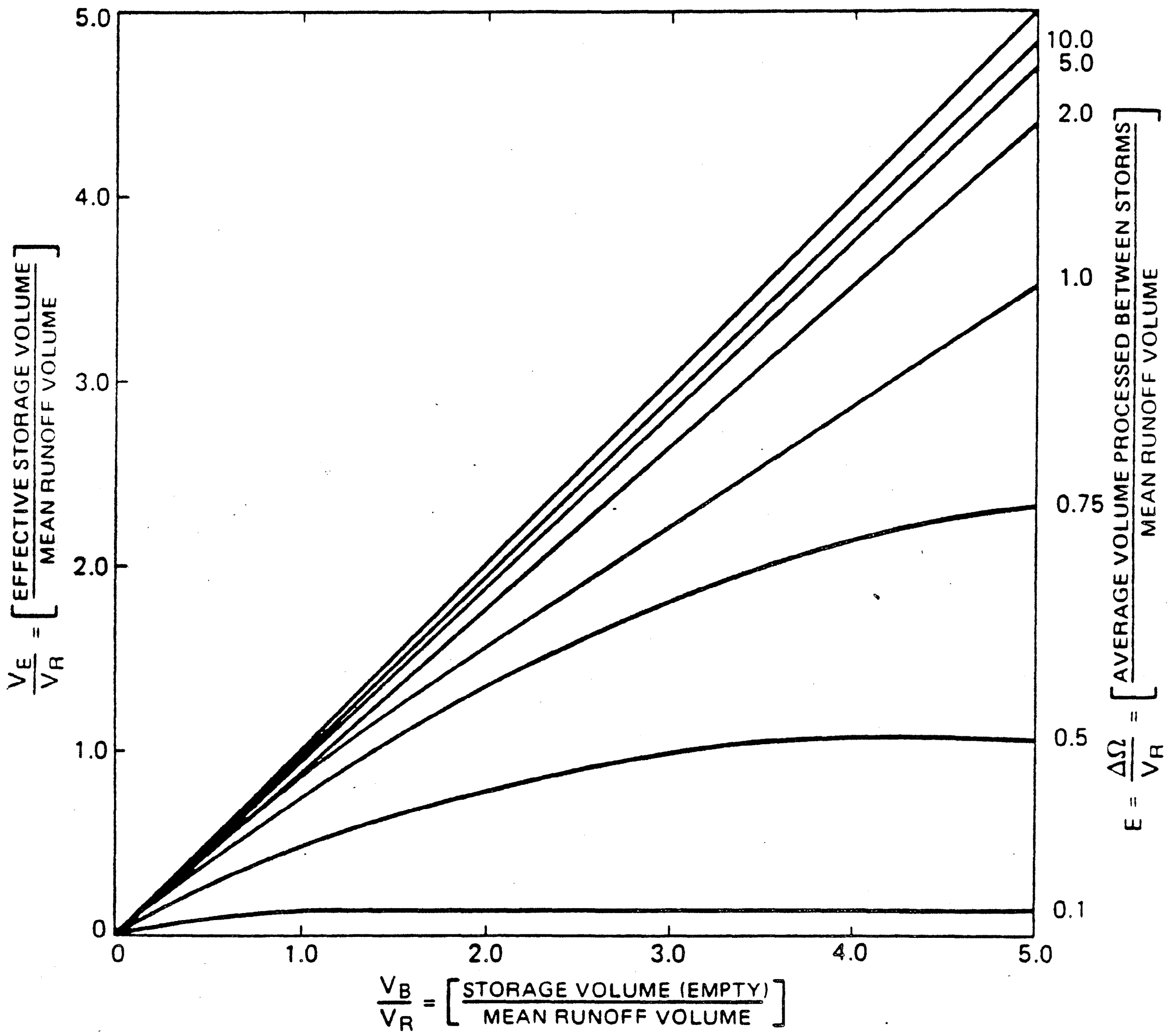


Figure 4. Effect of Previous Storms on Long-Term Effective Storage Capacity

The expression $\Delta\Omega$ may be thought of as the volume emptied from the basin during the average interval between storm events. The smaller this quantity is relative to VR, the average volume entering the basin during storms, the more likely it is that the basin will still contain leftover runoff when a storm begins, and the smaller will be the effective volume. When this ratio, E, is less than about 2, the effective volume becomes quite small compared with the physical volume provided, especially for the larger basins.

3.1 GENERAL

Recharge devices may take a variety of forms, including porous pavement, infiltration trenches, percolating catch basins, or larger basins which occupy land set aside for the purpose. There are no fundamental differences in the devices, either in the way they control storm runoff, or in the procedure for analyzing performance. The differences are in details such as the size of the basin, the configuration, and the size of the catchment area routed through a particular unit.

Given a specific surface area provided for percolation, and a unit infiltration rate defined by soil characteristics, an overall "treatment rate" can be defined for a specific device. When storm runoff is applied to the device at rates equal to or less than this rate, 100% is intercepted. At higher applied rates, the fraction of the runoff flow in excess of the treatment rate overflows to a surface water.

If the device also provides storage volume, the volume stored can be retained for subsequent percolation. Overflow to surface waters (runoff that "escapes" the device) occurs only when the available storage is exceeded. Long-term average removal is the net reduction in overflows over the long-term sequence of storms of different size, with different intervals between successive storms.

Performance will obviously vary with the basin size in relation to the area served, with the soil percolation rate, and with the characteristics of local storm patterns.

The analysis procedure described in this section permits one to either (a) evaluate the potential for a specific recharge installation to reduce pollutant loads from a particular drainage area, or (b) develop a general relationship on size or areal density for different levels of pollutant control. Examples of a site-specific approach are presented below; generalized analysis results are presented and discussed later in Section 5.

Level of control is expressed as a long-term average removal of storm runoff flows. The tacit assumption is that the urban runoff which is caused to percolate into the ground is "removed" as a discharge to surface water bodies, as are the pollutants which are present in the runoff. Any percolated waters which eventually reach surface waters through groundwater flow are assumed to percolated waters which eventually reach surface waters through groundwater flow are assumed to have had pollutants of interest removed by relevant soil processes (filtration, biological action), and hence are ignored by the analysis. The validity of this assumption will be influenced by the type of pollutant of interest and local conditions.

As with any model or computation, judgment is required in interpreting the results of this analysis, and in evaluating the overall suitability of recharge devices in a local area. Apart from the factors used in the analysis, considerations such as soil type, slope and stability, depth to water table, etc., will be important determinants of suitability at any site.

It should be noted that the analysis does not address eventual blockage of the soil. The rates assigned should be typical values which can be maintained naturally or by maintenance programs. Neither does the analysis speak to the issue of contamination of the ground water aquifer. Such considerations must be addressed in any actions or decisions related to implementation of this control approach.

The input data requirements for use of the analysis procedure consist of the following:

- Rainfall - mean and coefficient of variation of rainfall intensity. These statistics are developed by the SYNOP program. (See the Appendix for further discussion on this procedure and for a summary of data for a number of cities in different regions of the country.)
- Urban Catchment - area and runoff coefficient (ratio of runoff to rainfall).
- Device Size - surface area provided for percolation, and storage volume.
- Percolation Rate - rate of infiltration provided by local soil - usually reported in inches per hour or gallons per day per square foot. A "Treatment Rate" is defined as the product of the unit percolation rate and the surface area over which percolation occurs.

3.2 ANALYSIS METHOD

Figure 5 illustrates the operating principles involved and summarizes the terminology. The illustation is for the general case; for specific recharge device designs, only the configuration is different. For example, porous pavement would be represented as having a negligible storage volume; an infiltration trench would have the storage area filled with coarse aggregate, and available storage volume reduced to the void volume contained within the gravel or crushed stone.

It is assumed that the device is at the "downstream" end of the urban drainage area it serves, i.e., all runoff from the defined catchment area is routed through the basin.

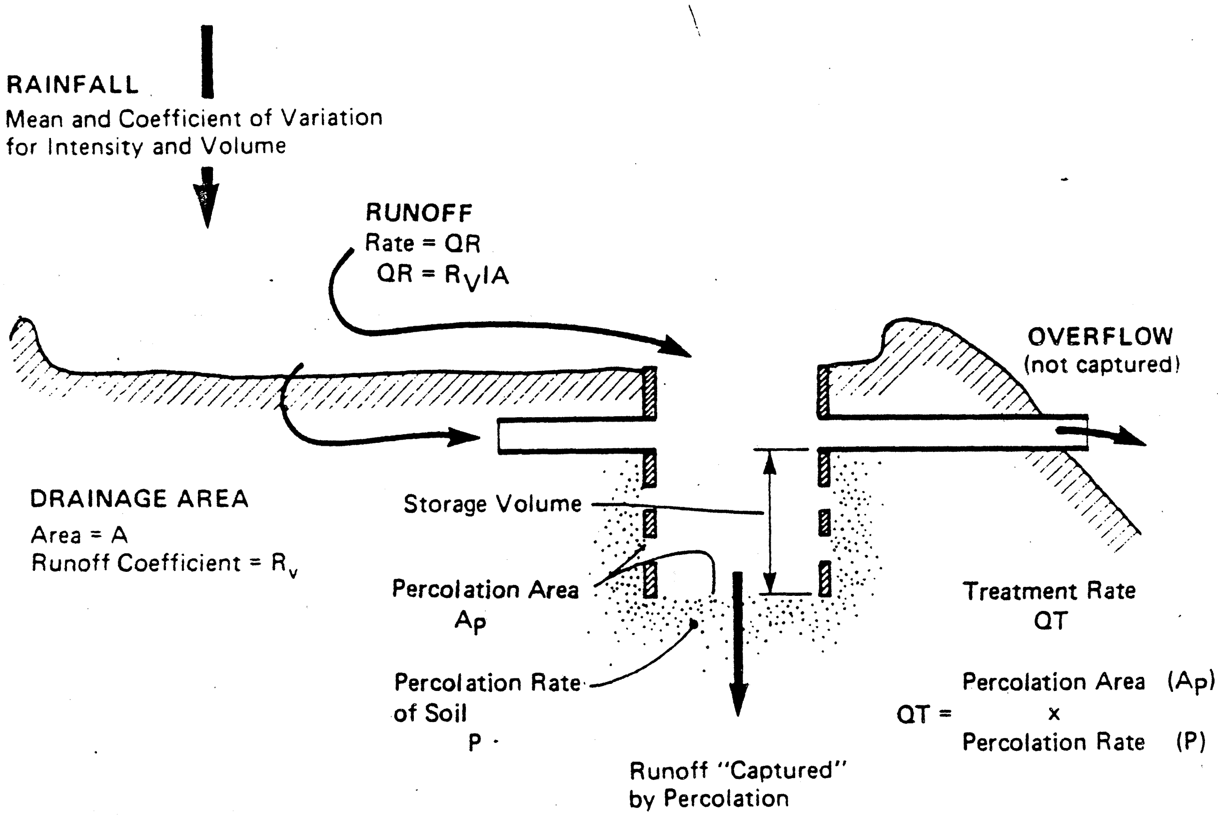


Figure 5. Schematic illustration of recharge device

Long-term performance characteristics are defined as a function of the ratio between the "treatment capacity" (QT) of the device and the runoff rate (QR) from the average storm. It is strongly influenced by the inherent variability in the rate of runoff for different storms -- which is characterized by the coefficient of variation of runoff flow rate (CV_q).

If there were no variability, i.e., if all runoff entered the device at the mean runoff rate, then performance during any event and long term average performance would be the same and would be equal to the treatment capacity provided relative to the applied rate. If treatment capacity were made equal to runoff rate (QT/QR = 1), 100% removal would be achieved. However, where treatment rate is fixed by design and runoff rate is variable, performance is reduced. The greater the variability, the poorer the performance, on average, because of the increasing number and magnitude of events which produce rates greater than the mean runoff rate.

3.3 EXAMPLE COMPUTATIONS

The performance of recharge devices can be projected using the performance curves presented in Section 2. The examples presented in this section illustrate the use of these curves.

3.3.1 Porous Pavement

A. Given

A shopping center has an area of 1 acre. It is all paved surface and runoff coefficient is estimated to be 0.9. Configuration and slopes are such that porous pavement can be installed as part of the catchment paved area and intercept all runoff produced.

The controlling rate of percolation (either porous pavement or the soil below it) is 1 inch/hour.

Storage volume in pores of pavement is assumed negligible.

The site is near Baltimore, Maryland, and rainfall statistics for the area are estimated (from tables in the Appendix) to be:

		Mean	Coef. of Variation
Volume (V)	inch	0.40	1.48
Intensity (I)	in./hr	0.069	1.21
Duration (D)	hour	6.0	1.01
Interval (Δ)	hour	82.0	1.03

B. Required

Estimate the long-term average percentage of storm runoff that would be captured if porous pavement, equal to 10% of the total area of the catchment, were installed.

C. Procedure

Step 1 - Select appropriate performance curve to use for estimate.

- Porous Pavement provides no significant amount of storage volume. Therefore, the device does not capture any volume, and Figures 3 and 4 do not apply.
- Percolation rate, and hence treatment rate (QT) is independent of applied flow rate. Thus, the treatment rate does not depend on flow and Figure 2 does not apply.
- Mode of operation corresponds to that described for FLOW - CAPTURE devices described in Section 2.3. Therefore Figure 1 describes performance.
- Performance estimates are based on QR, QT and CV_q.

Step 2 - Compute mean runoff rate (QR) in cubic feet per hour.

$$QR = (I) * (R_V) * (AREA) * (DIMENSION\ CONVERSION)$$

$$= 0.069 * 0.9 * 1 * 43560/12$$

$$= 225\ CFH$$

Step 3 - Compute treatment rate (QT) in cubic feet per hour.

Percolation rate (P) is 1 in./hr = 0.083 ft/hr

Treatment rate QT = Rate (P) * Area (A_p)

If 10% of the 1-acre catchment area is installed as porous pavement:

$$A_p = 43{,}560 * 0.10 = 4{,}356 \text{ sq ft}$$

$$QT = P * A_p = 0.083 * 4{,}356 = 362\ CFH$$

Step 4 - Compute Design Ratio (QT/QR).

QT (from step 3) = 362 CFH
QR (from step 2) = 225 CFH

QT/QR = 362 / 225 = 1.6

Step 5 - Estimate Long-term Removal.

- In Figure 1, enter horizontal axis at QT/QR = 1.6
- Extend a line vertically until it intersects the curve for the coefficient of variation (from rainfall statistics for intensity, CV_q = 1.25 approximately)
- Extend a line horizontally from this point, and read removal efficiency as approximately 72%

3.3.2 Recharge Basin

A. Given

For a 10-acre residential development, the runoff coefficient is estimated at 0.25. All stormwater runoff from the area is to be routed to a recharge basin.

Minimum basin depth must be at least 2 ft to penetrate a relatively impervious surface soil and reach a layer with good drainage properties. The subsoil has a percolation rate of 2.5 in./hr.

Rainfall statistics for the area are :

		Mean	Coef. of Variation
Volume (V)	inch	0.53	1.44
Intensity (I)	in./hr	0.086	1.31
Duration (D)	hour	7.2	1.09
Interval (Δ)	hour	85.0	1.00

Space constraints limit the basin to a bottom dimension of 25 by 50 ft, or a maximum percolation area of 1250 sq ft.

B. Required

Estimate the long-term average reduction in storm runoff that can be obtained from a recharge basin with the minimum (2 ft) depth.

C. Procedure

Step 1 - Select appropriate performance curve(s).

- Figure 1 applies in this case because treatment rate is based on percolation rate, and is independent of applied flow
- Figure 2 does not apply for the above reason
- Figures 3 and 4 also apply in this case because storage capacity is provided by the device

Step 2 - Compute runoff parameters for mean storm flow rate (QR) and volume (VR).

$$QR = (I) * (R_V) * (Area) * (43{,}560/12)$$

$$= 0.086 * 0.25 * 10 * 3630 = 780 \text{ CFH}$$

$$VR = (V) * (R_V) * (Area) * (43{,}560/12)$$

$$= 0.53 * 0.25 * 10 * 3630 = 4807 \text{ CF}$$

$CV_q = 1.31$ and $CV_v = 1.44$

Step 3 - Compute treatment rate (QT) and the design ratio for treatment (QT/QR).

Percolation rate (P) = 2.5 in./hr = 0.208 ft/hr

Percolation area (A_p) = 1,250 sq ft

$$QT = P * A_p = 0.208 * 1{,}250 = 260 \text{ CFH}$$

$$QT/QR = 260 / 780 = 0.33$$

Step 4 - Compute basin effective volume and the design ratio for storage (VE/VR).

For the minimum (2 ft depth) basin, physical basin volume (VB) is:

$$VB = 1{,}250 \text{ ft}^2 * 2 \text{ ft} = 2{,}500 \text{ cu ft}$$

$$VB/VR = 2{,}500 / 4{,}807 = 0.52$$

Emptying Rate ratio (E)

$$E = \Delta * \Omega / VR$$

Δ is the average interval between storms = 85 hr

Ω is the emptying rate of flow = QT = 260 CFH

$$E = 85 * 260 / 4{,}807 = 4.6$$

From Figure 4, enter horizontal axis at VB/VR = 0.52; extend a line vertically to intersect curve for E = 4.6; then horizontally to read VE/VR on vertical axis. Estimate that effective volume VE is essentially the same as physical volume for this case.

$$VE/VR = VB/VR = 0.52$$

<u>Step 5</u> - Estimate performance of recharge basin.

- Removal accomplished by infiltration is estimated from Figure 1 for the conditions

  QT/QR = 0.33 and $CV_q = 1.31$
  % Removed (FLOW) = 24%

- Removal accomplished by storage is estimated from Figure 3 for the conditions

  VE/VR = 0.52 and $CV_v = 1.44$
  % Removed (VOLUME) = 35%

  (This efficiency applies not to the overall runoff from the drainage area, but to the fraction that escapes the percolation process.)

- Overall removal accomplished by the combined infiltration/storage process may be computed directly from the fractions <u>NOT</u> removed by each process.

  Fraction not removed by infiltration

  $$f_Q = 1 - (\%\ \text{Removed}/100) = 0.76$$

  Fraction not removed by storage

  $$f_V = 1 - (\%\ \text{Removed}/100) = 0.65$$

  $$\%\ \text{Removed (overall)} = (1 - [f_Q * f_V]) * 100\%$$

  $$= (1 - [0.76 * 0.65]) * 100\%$$

  $$= 51\%$$

3.4 VALIDATION

Although several of the NURP sites included recharge devices, the data obtained were not sufficient in either scope or extent to provide a suitable basis for use as a validation test for the probabilistic procedure described above.

An examination of the reliability of the performance estimates provided by the procedures presented in this report was conducted by comparing projections for a range of conditions with those produced by an established deterministic simulation model. The model "STORM" was used to generate runoff for a hypothetical urban drainage area, using a long-term (approx. 20 years) hourly rainfall record. This runoff record was then processed by the Storage-Treatment block of the SWMM model, and from the long-term output produced by the simulation, the average percent reduction was computed.

This computation was performed for a variety of basin sizes and soil percolation rates.

Figure 6 compares these results with those produced by the probabilistic analysis procedures.

3.5 DISCUSSION

The procedures described for estimating performance of recharge devices on the basis of size, local soil conditions, and rainfall patterns provide estimates that compare quite favorably with those produced by accepted simulation techniques. They are simple to use and permit examination of the wide variety of alternatives usually desirable in planning activities.

The procedures described provide a basis for quantifying the performance capabilities of a variety of recharge devices, using information that will normally be readily available. However, the suitability of recharge/infiltration systems will vary with location and must be determined on the basis of local conditions.

The possibility of contributing to undesirable impacts on ground water aquifers by enhanced recharge to protect surface waters must be considered on a local basis. Situations have been identified where it has been concluded that the contaminants (and their concentrations) normally present in urban runoff, and which reach the aquifer following percolation, do not constitute a problem or a significant cause for concern. In these situations the practice is encouraged. There are, however, other situations where there are legitimate concerns with the appropriateness of this approach.

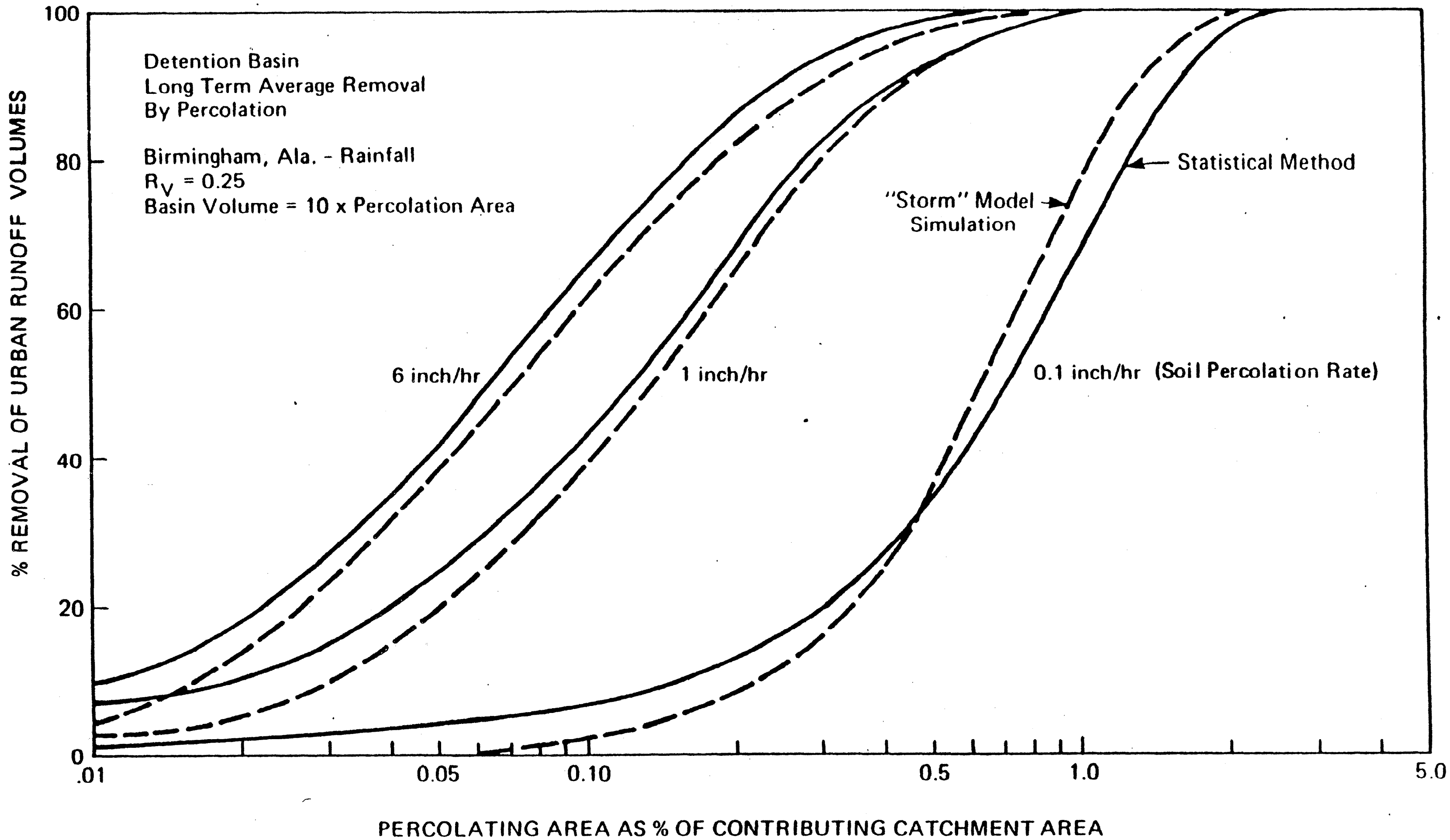


Figure 6. Detention basin performance - long term average removals by percolation - comparison of statistical and simulation methods

The approach may be unsuitable for areas with steep slopes and unstable soils, or areas with water supply wells in sufficiently close proximity to recharge areas.

A tacit assumption in the analysis is that the water table is far enough below the percolation surface that a significant interaction with the temporary mound of ground water, which may form during an event, does not take place.

A further consideration is that percolation rates assigned in the analysis are represenative of long-term conditions, and that significant soil blockage with use either does not occur or is accounted for. Historical experience with recharge basins and with land application of waste waters indicates that progressive blockage is not generally a problem when the soil can be "rested" between applications. The intermittent nature of storms, and the fact that in most areas of the country storm periods occur less than 10% of the time automatically provides such rest periods that help maintain soil permeability.

4.1 GENERAL

Detention basins that receive storm runoff, but that have negligible losses through infiltration, must rely principally on sedimentation processes for pollutant removal. Under some conditions, and to some extent, reductions attributable to other processes may influence removal of specific pollutants (e.g., natural die-off of coliform bacteria, and algal uptake of soluble nitrogen and phosphorus).

Of the variety of configurations and operational modes that have been used, stormwater detention basins that maintain a permanent pool of water, often referred to as "wet ponds," are generally considered to be the most effective for pollutant reduction.

Nine such devices in various parts of the county were actively monitored during the NURP program, as the local agencies' choice of a preferred control approach.

This section presents a procedure for projecting performance of such devices, and a comparison of results with observed performance of the NURP detention basins. A wide variety of concepts and configurations is represented by the wet ponds that were studied, ranging from oversized storm drains to natural ponds and small lakes. The size of the devices relative to the contributing drainage area varied over a wide range; the common elements for all were the maintenance of a permanent pool of water and sedimentation as the principal pollutant-removal mechanism.

The input data requirements for analysis of sedimentation devices are essentially the same as for recharge devices described in the previous section, but with the following exception. In this case the "treatment rate" is determined not by soil percolation rates, but by the settling velocity of the particulates present in the urban runoff. Represenative values for settling velocity can be assigned to urban runoff on the basis of a significant number of settling column tests conducted during the NURP program.

4.2 ANALYSIS METHOD

The probabilistic computations and performance curves presented in Section 2 can be applied to wet ponds (with appropriate adaptation and interpretation) to reflect the nature of the treatment process that occurs in detention basins of this type.

A basic aspect of such a system is that part of the time (while runoff inflows occur), stormwater is moving through the basin, and sedimentation takes place under dynamic conditions. During the considerably longer dry periods between storm events, sedimentation takes place under quiescent conditions.

4.2.1 Removal Under Dynamic Conditions

Characterization of the performance of sedimentation devices has been extensively analyzed over the years because of the important role such devices play in both water treatment and wastewater treatment systems. A method of analysis which is particularly suitable is presented by Fair and Geyer (5). Removal due to sedimentation in a dynamic (flow through) system is expressed by the following equation:

$$R = 1 - \left[1 + \frac{1}{n} \cdot \frac{v_s}{Q/A}\right]^{-n} \qquad (8)$$

where:

R = fraction of initial solids removed (R * 100 = % Removal)

v_s = settling velocity of particles

Q/A = rate of applied flow divided by surface area of basin (an "overflow velocity," often designated the overflow rate)

n = a parameter which provides a measure of the degree of turbulence or short-circuiting, which tends to reduce removal efficiency

One value of this model is that it provides a quantitative means of factoring into the analysis an expression for impaired performance due to short-circuiting (since many stormwater retention basins will not have ideal geometry for sedimentation). Fair and Geyer suggest an empirical relationship between performance and the value of "n," which is: $n = 1$ (very poor); $n = 3$ (good); $n > 5$ (very good). In addition, when a value of $n = \infty$ is assigned (ideal performance), the equation reduces to the familiar form wherein removal efficiency is keyed to detention time.

$$R = 1 - \exp\left[- \frac{v_s}{Q/A} \right] \quad \text{or} \tag{9}$$

$$R = 1 - \exp\left[- k\,t \right] \tag{10}$$

where:

k = v_S / h (sedimentation rate coefficient)

h = average depth of basin

t = V / Q residence time

V = volume of basin

The two expressions are equivalent. To use them, one must be able to identify an appropriate value for either settling velocity, or for the rate coefficient (k), which will ultimately depend on the settling velocity of the particulates present.

Solving equation (8) for a range of overflow rates and particle settling velocities and plotting the results as shown by Figure 7, indicates the wide range in removal that can be expected either (a) at a constant overflow rate for particles of different size, or (b) at different rates of flow for a specific size fraction. Both of these variable factors are present in urban runoff applications. The effect of a range of particle settling velocities is addressed by performing separate computations for a number of settling velocities and then using weighted mass fraction to compute net removal.

Storm sequences result in variable overflow rates, each event producing a different average rate, and hence, removal efficiency. The probabilistic analysis procedure described in Section 2.4 (Flow-Treatment), and summarized by the design performance curves in Figure 2, is the relevant analysis to apply. This analysis makes the following assumptions:

- The short-term variability of flows (within storm events) is small compared with the variability of average flows between storms. To the extent that this is not the case, Figure 2 will overestimate long-term performance.

- Storm flows and pollutant concentrations are independent. If flow rate and concentration are negatively correlated (high flows produce lower concentrations), performance will be better than indicated. For positive correlations, performance will be poorer than indicated.

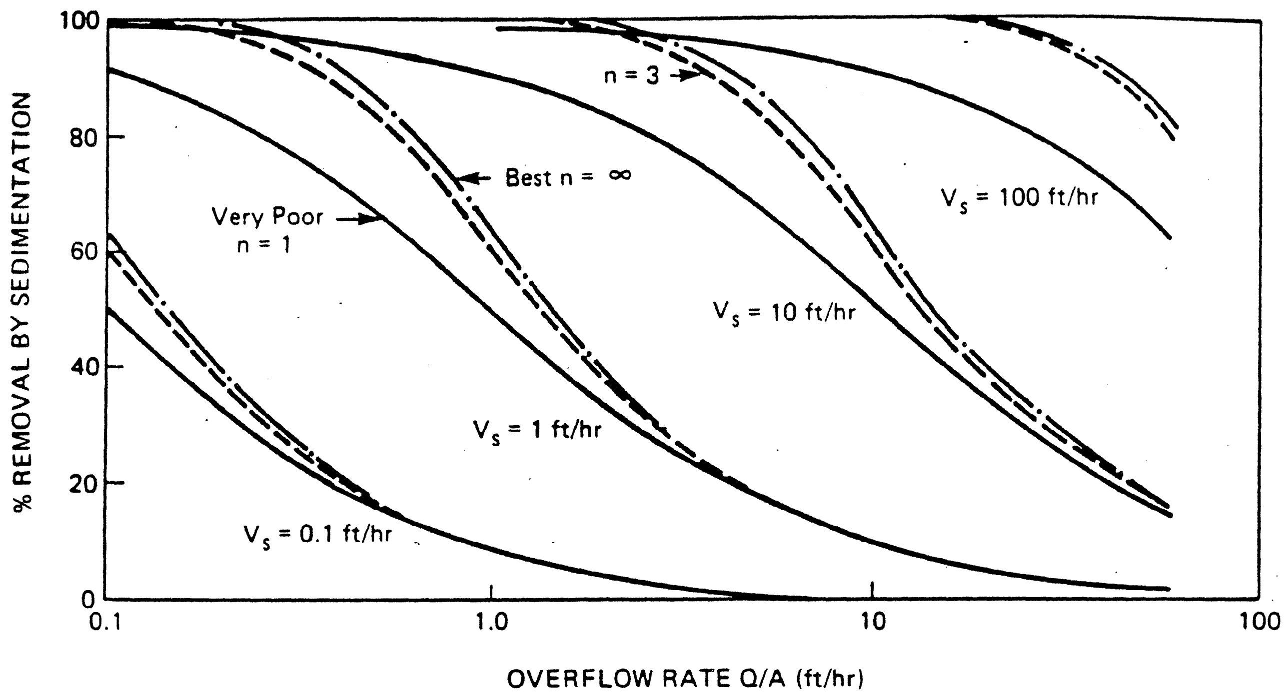


Figure 7. Effect of settling velocity and overflow rate on removal efficiency

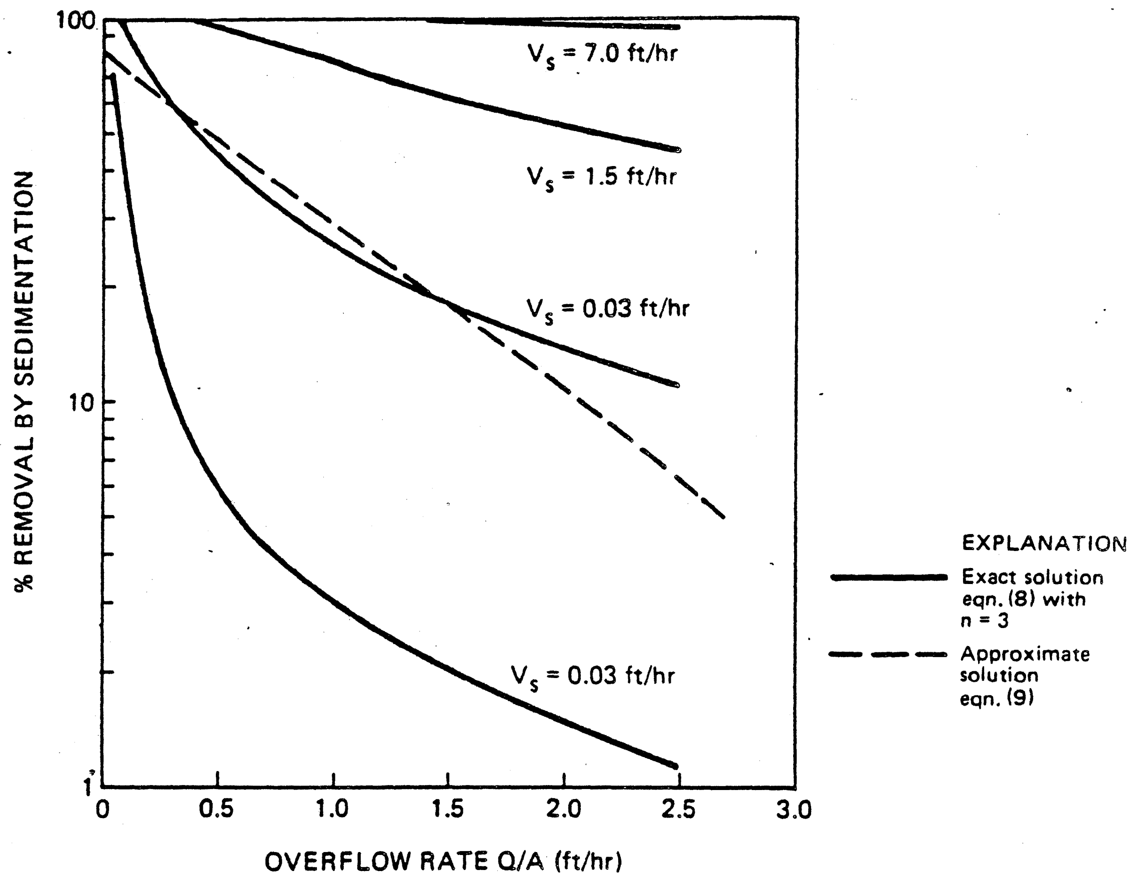


Figure 8. Flow-removal relationships for exponential approximation

- Removal efficiency is an exponential function of flow.

Available data on stormwater retention basins are not suitable to provide empirical estimates of flow rate/removal relationships. The relationship represented by equation (8) has been used instead. Removal fractions for a range of settling velocities representative of urban runoff, as computed by equation (8), are presented in Figure 8 as a semi-log plot on which the exponential approximation, equation (9), would plot as a straight line. For a site-specific analysis (for each settling velocity separately), the straight line approximation would match the exact solution at the point corresponding to the mean overflow rate (QR/A), and the slope would be adjusted to give the best match over the range of rates expected to span the bulk of the important storms. The intercept of this fitted line (Q/A = 0) provides the estimate for the factor Z in equation (3). For example, in the sample illustration shown in Figure 9, the overflow rate for the mean storm is 1.5 ft/hr. For the size fraction represented by a settling velocity of 0.3 ft/hr, removal at the mean flow rate (RM) is 0.18 and Z is estimated to be 0.8. Over the range of overflow rates of interest, the exponential approximation is within about 10%.

Long-term average removal of a pollutant under dynamic conditions can, therefore, be estimated from the statistics (mean and coefficient of variation) of runoff flows, basin surface area, and representative particle settling velocities for urban runoff.

4.2.2 Removal Under Quiescent Conditions

For much of the country, the average storm duration is about 6 hours, and the average interval between storms is on the order of 3 to 4 days. Thus, significant portions of storm runoff volumes may be detained for extended periods under quiescent conditions, until displaced by subsequent storm events. The volume of a basin relative to the volumes of runoff events routed through it is the principal factor influencing removal effectiveness under quiescent conditions.

The probabilistic computation described previously in Section 2.5 (Volume-Capture), and summarized by design performance curves in Figures 3 and 4, is used to estimate removals under quiescent conditions. This analysis assumes that physical volumes are removed from the basin during the dry periods between storms, as in the recharge basin analysis presented in the preceding section, where captured volume percolates. However, for sedimentation devices that maintain a permanent pool of water, some modification is required because there is no loss of stored volume between runoff events. Instead, it is the particulates in the detained volume that settle out under quiescent conditions. The modification required is to express this condition in terms of the parameters of the design performance curves.

The term Ω may be thought of as a "processing rate." For a recharge device, it is the rate at which volume is removed from the basin by percolation through the bottom and sides. For a sedimentation device, it may be thought of as a particle removal rate. Using this interpretation, the term $\Omega\,\Delta$ in equation (7) can be considered to represent that portion of the basin volume from which solids with a selected settling velocity have been completely removed. Instead of the TSS concentration of the entire volume diminishing with time under quiescent settling, the concentration is assumed to remain constant, while the remaining volume with which this concentration is associated diminishes with time. The solids removal rate is then:

$$\Omega = v_s * A \tag{11}$$

where:

v_s = particle settling velocity (ft/hr)
A = basin surface area (square feet)

4.2.3 Combining Dynamic and Quiescent Effects

The procedures described above can be used to compute separate long-term removal efficiencies under dynamic and quiescent conditions. Since each type of condition prevails in a detention basin at different times, the overall efficiency of a basin is the result of the combined effect of the two processes at work. The simple model used to integrate these effects is illustrated by Figure 9.

Five identical storms with an interval between event midpoints (Δ) of 3.5 days are routed through a basin, assuming plug flow. Each storm has a duration of 12 hours (0.5 day), and a volume which is 25% of the basin volume (VB/VR = 4). The plotted lines track the residence/displacement pattern in the basin for the leading edge, midpoint, and trailing edge of Storm #1. The shading highlights the fraction of the total residence time when dynamic conditions prevail. For this simplified case, and for actual conditions where both storm volumes (VR) and intervals (Δ) fluctuate, the fraction of time under dynamic conditions is estimated by:

Fraction of residence time under dynamic conditions $= D / \Delta$ (12a)

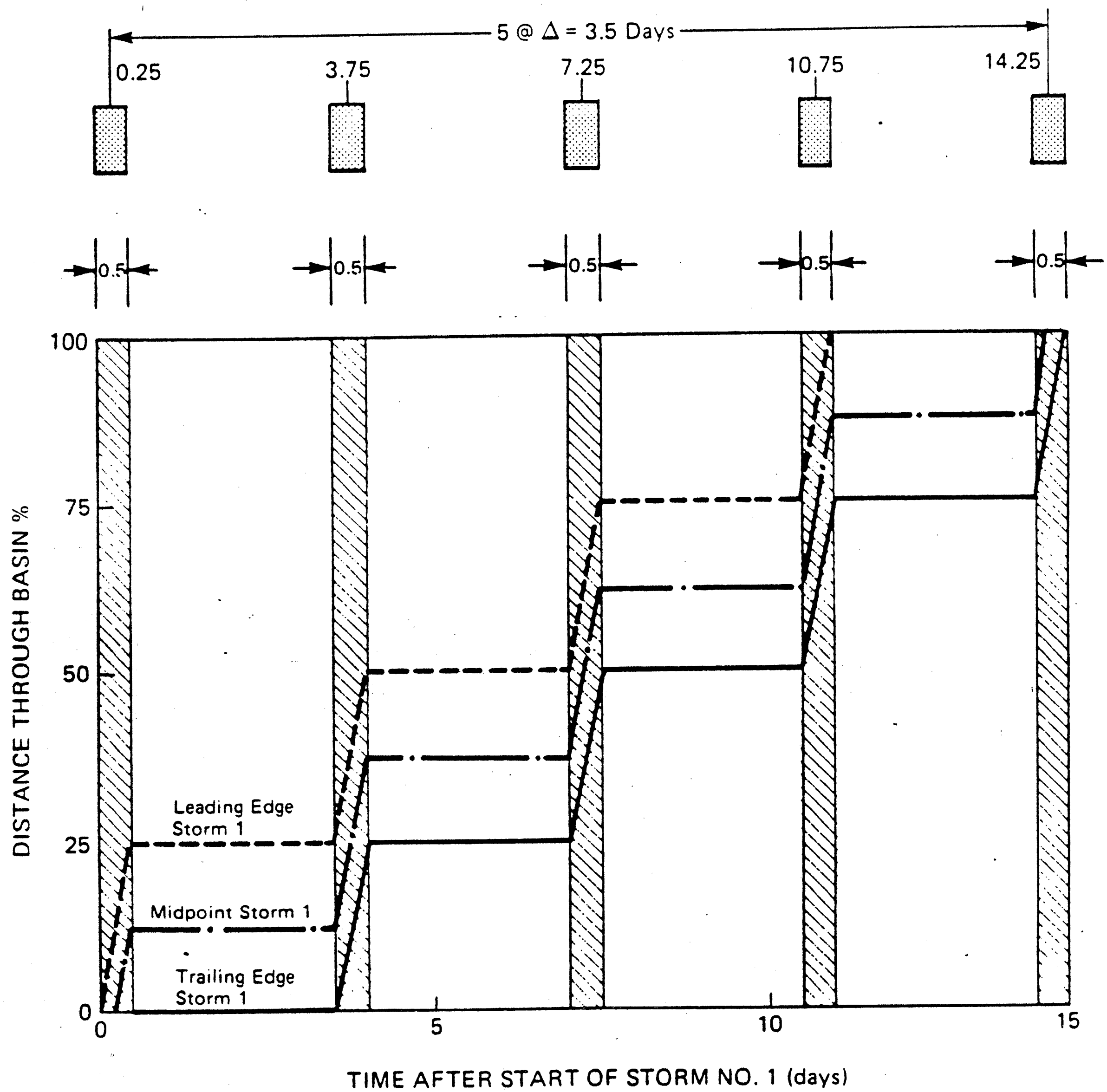
Fraction under quiescent conditions $= 1 - (D / \Delta)$ (12b)

where:

D = mean storm duration
Δ = mean interval between storm midpoints

This simple schematic illustrates several relevant features of the operation of this type of device. When the basin is as large as that indicated (which is not uncommon for current practice), the outflow volume during an event represents a different parcel of water than that for the storm that causes it to be displaced. Assessing performance by comparing paired influent and effluent loads for individual storms is less appropriate than the comparison of overall influent and effluent loads for a long-term sequence of storm events.

All runoff volumes which enter the basin undergo the dynamic removal process one or more times before discharge. For the large basin illustrated, this is broken up into four different periods of displacement. For a basin with a volume small enough that the runoff passes all the way



For Storm Midpoint Volume

Total Residence Time = 14.0 Days

Dynamic Time: (0.25) + (3 x 0.5) + 0.25 = 2.0 Days 2/14 = 0.14

Quiescent Time: 14.0-2.0 = 12 Days 12/14 = 0.86

$D/\Delta = 0.5/3.5 = 0.14$

$(1-D/\Delta) = 0.86$

Figure 9. Illustration of quiescent vs. dynamic residence time in a storm detention basin

through, there would be only one such period of dynamic removal. Performance efficiency is affected simply on the basis of the "overflow rate" that the basin size provides.

The quiescent removal process then operates on (a) those portions of the total runoff volume that remain in the basin during the dry interval that follows an event, and (b) on that fraction of the influent pollutants that remain in the water column after operation of the dynamic process. In the situation illustrated, the average runoff volume is exposed to four different periods of quiescent settling, amounting to an extended period under this condition. In a very small basin, the relative effect of the quiescent removal process may be insignificant, simply because such a small fraction of the total runoff remains in the basin at the end of each storm.

The removal efficiency for the basin under the combined effect of both dynamic and quiescent processes can be computed by applying the removal efficiency of either the dynamic or quiescent process to the pollutant fraction remaining after the operation of the other. If the fractions not removed by the dynamic and quiescent processes operating independently are f_D and f_Q, respectively:

$$\text{COMBINED \% REMOVAL} = 100 \left[1 - (f_D * f_Q) \right] \qquad (13)$$

It should be noted that in the larger basins, either process operating alone will be capable of high degrees of removal. One might consider the quiescent process to be the dominant one in large basins, because high particulate reductions can be produced even if there were no removal during dynamic periods, and because the quiescent periods provide the conditions in which the removal processes other than sedimentation can come into play. In small basins, the dynamic process will be the dominant one because only small fractions of the runoff will remain in the basin subject to the quiescent process.

4.3 VALIDATION

Performance data from nine wet pond detention basins monitored during the NURP program have been analyzed and used to test the reliability of the probabilistic methodology. These devices cover a wide range of physical types, and also provide a wide range of basin sizes relative to the contributing urban drainage area.

For the calibration effort, monitored data on storm runoff rates and volumes entering a detention basin are analyzed to define their statistical characteristics. For long-term performance projections, long-term rainfall records for the area in question are used, and the statistical properties of runoff are estimated from the rainfall record. The settling velocity of particulates in urban runoff is estimated from data obtained from settling column tests performed by a number of the NURP projects.

In addition to producing a fairly extensive data base on pollutants entering and leaving detention devices, another critically important contribution of the NURP effort was data to support estimates of the settling velocity of particles in urban runoff. Any analysis methodology for sedimentation, including that adopted for this analysis, requires information of this nature for use either directly (equation 8) or in surrogate form, as with a reaction rate (equation 10).

4.3.1 Settling Velocity of Particles in Urban Runoff

Settling tests were conducted by a number of NURP projects on samples of urban runoff. Results from these tests, and from a similar set of tests reported by Whipple and Hunter (7), have been analyzed to derive information on particle settling velocities in urban stormwater runoff. The analysis procedure used for reducing settling test data and a detailed discussion of the overall analysis results, which are summarized briefly below, are presented in the Appendix.

The analysis of 46 separate settling column tests indicates the following:

- There is a wide range of particle sizes, and hence settling velocities in any individual urban runoff sample.

- The distribution of settling velocities can be adequately characterized by a log-normal distribution.

- There is substantial storm-to-storm variability in median (or other percentiles of) settling velocity at a specific site. The range indicated is about one order of magnitude in observed values for any percentile of the distribution in a specific storm. Uncertainty in the coefficient of variation of the site-averaged settling velocity distribution (95% confidence interval) is smaller, but still appreciable (about a factor of 5).

- No significant differences between site-to-site mean distributions have been identified. The within-site variability is on the same order as potential site-to-site differences.

- Assuming the data available for analysis are representative, the foregoing indications, with regard to storm-to-storm and site-to-site differences, support the pooling of all available data to define "typical" characteristics of particle settling velocity distributions in urban runoff, and the assumption that such results are generally transferrable to other urban runoff sites. Appendix Figure A-5 illustrates best estimates (at present) for the distribution of particle settling velocities in urban runoff from any site. For the calibration tests and subsequent projections, computations are performed for five size fractions having the following average settling velocities (based on the distribution shown by Figure A-5):

Size Fraction	% of Particle Mass in Urban Runoff	Average Settling Velocity (ft/hr)
1	0 - 20%	0.03
2	20 - 40%	0.3
3	40 - 60%	1.5
4	60 - 80%	7.
5	80 - 100%	65.

4.3.2 NURP Performance Results

A total of thirteen detention basins were monitored by various NURP projects. Of these, nine may be classified as "wet basins," which maintain a permanent pool of water. Performance characteristics of these basins have been analyzed and used to compare observed removals to those predicted using the methodology described earlier.

The detention basins studied under the NURP program encompass a wide variety of physical types. They include oversized sections of a storm drain installed below street level (Grace Street sites), ponds or small lakes on streams which drain urbanized areas (Unqua Pond, Lake Ellyn), flood control basins (Traver), a converted farm pond (Westleigh), and a golf course pond through which storm drains from an adjacent urban area were routed (Waverly Hills site). In spite of this diversity, these different detention devices may be compared by the ratio of the size of the device relative to the connected urban drainage area, and the magnitude of the storms which are treated.

Table 1 summarizes such size relationships for the NURP basins, which are arranged in order of increasing performance expectations. Based on the analysis presented in the previous section, one should expect that lower overflow rates (QR/A) and higher volume ratios (VB/VR) would tend to produce better removal efficiencies by sedimentation. Therefore, these ratios are used in Table 1 as qualitative indicators of performance. The wide range provided by the NURP data set is apparent. Basin #1 has an average overflow rate during the mean storm of about six times the median settling velocity (1.5 ft/hr) of particles in urban runoff. Further, less than 5% of the mean storm volume remains in the basin after the event, to be susceptible to additional removal by quiescent settling. At the other end of the scale, the mean storm displaces only about 10% of the volume of Basin #9, and the average overflow rate is a small fraction of the median particle settling velocity.

Table 2 summarizes the observed overall average performance of the NURP detention basins over all monitored storms. Removal efficiency is determined from the sum of pollutant masses entering and leaving the device for all storms. At some sites, there were an appreciable number of events for which monitoring data were only available for either inflows or outflows. In such cases, a reduced data set (consisting of only those events for which both inlet and outlet data were available) was used in the computation. The qualitative indications of relative performance suggested by the ranking (based on size) are supported by the tabulated results. However, the variability in actual performance results tends to confuse the picture somewhat, such that the performance relationships may be better seen in the illustrations presented in the following section.

4.3.3 Calibration Results

The probabilistic methodology was used to compute the expected removal by sedimentation of a number of pollutants. The surface area and volume of each of the nine detention devices was determined from the project reports. The statistics (mean and coefficient of variation) of runoff flow rate and volume were computed from monitoring data for storms entering the basin. A value of n = 3 was arbitrarily assigned for the shortcircuiting factor for all of the analyses which follow.

Table 1. SIZE RELATIONSHIPS FOR NURP DETENTION BASINS (BASED ON MONITORED STORMS)

			Detention Basin Size		
			Relative to Mean Monitored Storm		
Code No.	Project and Site	Approx. Average Average Basin Depth (Ft)	Overflow Rate QR/A (ft/hr)	Volume Ratio VB/VR	Relative to Size of Urban Catchment (Surf Area/Drain Area X 100%)
1	Lansing Grace Street N.	2.6	8.75	0.045	0.0095%
2	Lansing Grace Street S.	2.6	2.37	0.17	0.035%
3	Ann Arbor Pitt-AA	5.0	1.86	0.52	0.09%
4	Ann Arbor Traver	4.1	0.30	1.16	0.31%
5	Ann Arbor Swift Run	1.5	0.20	1.02	1.15%
6	Long Island Unqua	3.3	0.08	3.07	1.84%
7	Washington, D.C. Westleigh	2.0	0.05	5.31	2.85%
8	Lansing Waverly Hills	4.6	0.09	7.57	1.71%
9	Northern Illinois Lake Ellyn	5.2	0.10	10.70	1.76%

TABLE 2. OBSERVED PERFORMANCE OF WET DETENTION BASINS
REDUCTION IN PERCENT OVERALL MASS LOAD

Site No.	Project and Site	No. of Storms	Size Ratios		Average Mass Removals - All Monitored Storms (Percent)									
			QR/A	VB/VR	TSS	BOD	COD	TP	Sol.P	TKN	NO_{2+3}	T.Cu	T.Pb	T.Zn
1	Lansing Grace St. N.	18	8.75	0.05	(-)	14	(-)	(-)	(-)	(-)	(-)	(-)	9	(-)
2	Lansing Grace St. S.	18	2.37	0.17	32	3	(-)	12	23	7	1	(-)	26	(-)
3	Ann Arbor Pitt-AA	6	1.86	0.52	32	21	23	18	(-)	14	7	•	62	13
4	Ann Arbor Traver	5	0.30	1.16	5	(-)	15	34	56	20	27	•	•	5
5	Ann Arbor Swift Run	5	0.20	1.02	85	4	2	3	29	19	80	•	82	(-)
6	Long Island Unqua	8	0.08	3.07	60	(TOC=7)		45	•	(-)	(-)	•	80	•
7	Washington, D.C. Westleigh	32	0.05	5.31	81	•	35	54	71	27	•	•	•	26
8	Lansing Waverly Hills	29	0.04	7.57	91	69	69	79	70	60	66	57	95	71
9	NIPC Lake Ellyn	23	0.10	10.70	84	•	•	34	•	•	•	71	78	71

Notes: (-) Indicates apparent negative removals.

• Indicates pollutant was not monitored.

Because of the wide variability in particle settling velocities, and their important effect on removal by sedimentation, independent removal efficiency computations were performed for separate size fractions and results combined for the overall removals indicated. All five size fractions (Section 4.3.1) were assigned for TSS, total lead, and total P computations. For the other heavy metals (Cu, Zn), for TKN, and for BOD and COD, it was assumed that there would be no significant association with the largest size fraction, and computations were performed using four size fractions.

Most analyses of pollutant concentrations measured the total quantity, and did not distinguish between soluble and particulate fractions. Sedimentation computations are based on the particulate or settleable fraction. However, overall removal is expressed in terms of total quantities of pollutant, which is both the most relevant way to express results for control decisions as well as the basis for reporting observed results to be used for comparison with computations. For the analysis, therefore, it is necessary to assign the fraction of the total concentration or load which is settleable. For TSS, total P, and total lead, there is a reliable basis for doing so. Suspended solids are particulates by definition. Data developed through the NURP program indicate that lead consistently exhibits very high particulate fractions. Thus, although no specific measurements of soluble and particulate forms were made at detention basin sites, a particulate fraction of 0.9 can be assigned to lead with confidence. All but one of the sites (Basin #6) monitored both total and soluble phosphorus, and the actual particulate fraction for the site was used in the computation. A settleable fraction of 0.6 was assigned for Basin #6, guided by results from the entire NURP data base.

For these three pollutants, for which reliable estimates of particulate fractions are available and for which a significant fraction of the total is settleable, the comparison between observed removal efficiency and removals computed by the methodology described earlier is presented in Figure 10. There are a few obvious outliers; however, in general, predictions are within 10% to 15% of observed performance results. Additional confidence is derived from the fact that both observed and computed results span the entire range of performance possibilities, from less than 5% to 10%, to 90% or better.

Four significant outliers were identified and investigated. In all cases, actual monitored percent removal was much less than that projected.

- Site #4 (see Table 2) shows almost no TSS removal, although a substantial (~60%) removal is projected. At this newly installed basin, the project report indicates that significant bank erosion at the outlet structure occurred during the test program. Lead was not monitored, but observed/predicted Total P removals compare quite favorably at this site.

- Site #5 data show almost no Total P removal, although about 50% reduction is projected. On the other hand, both TSS and lead projections compare favorably with observed data. The basin is a shallow, vegetated area, characterized by the local project as a wetland. The possibility of the basin outlet discharging phosphorus from internal sources, rather than influent runoff, is suggested.

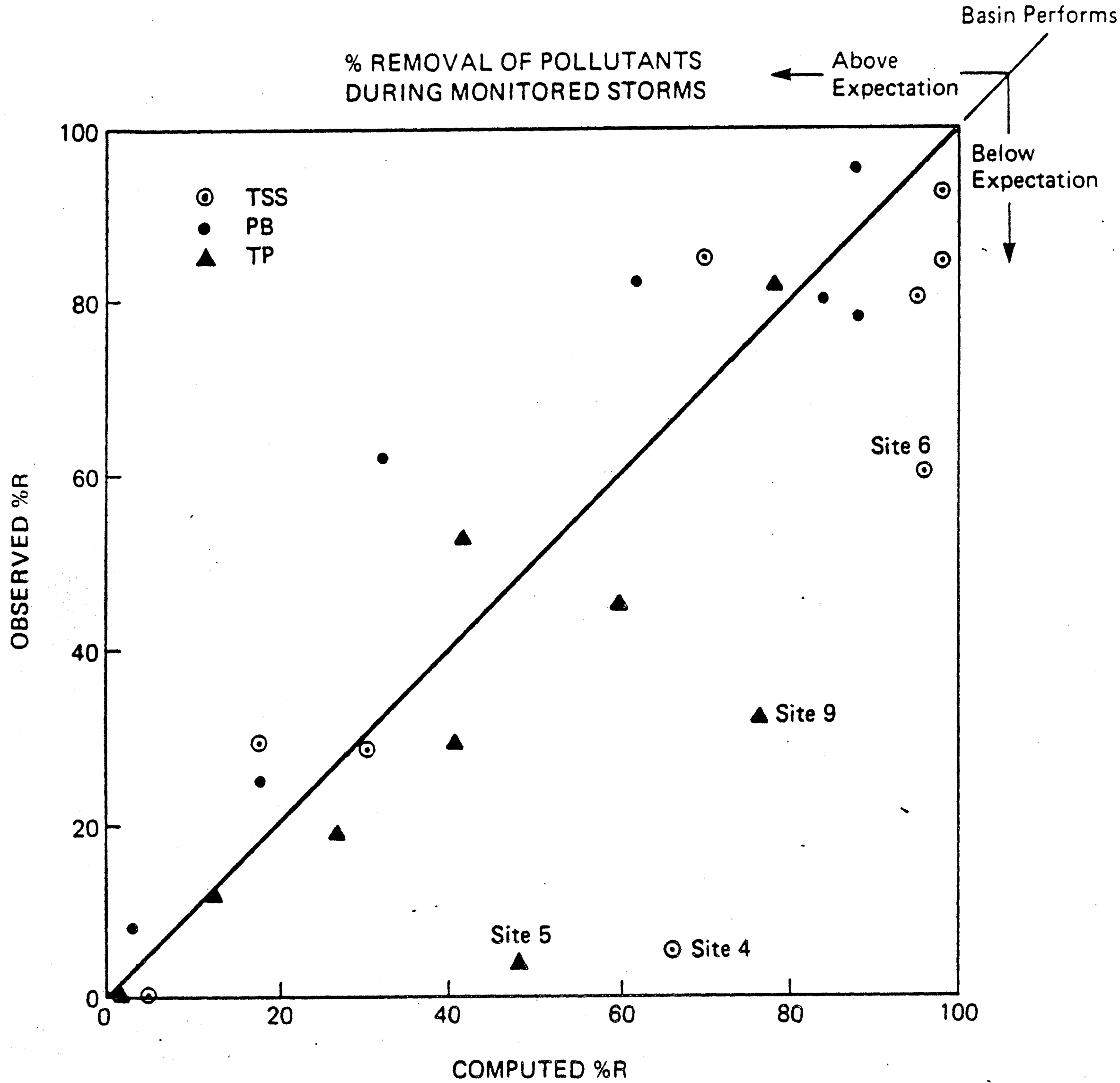


Figure 10. Comparison of observed vs. computed removal efficiencies (site numbers given for outliers—see text)

- Site #9 shows Total P removal projections that are significantly in excess of observed removals. However, as with Site #5, projected removals compared quite favorably with observed performance for both lead and TSS. This rather large basin, actually a five-acre lake, supports significant algal growth. The observed significant reductions for soluble phosphorus and nitrogen are attributed to algal uptake, since they could not have resulted from sedimentation.

  Conversion of soluble nutrients to algal cells would tend to add a source of TSS and Total P to basin outflows that are not associated directly with the particulate forms entering with the stormwater. Such processes tend to reduce the apparent sedimentation efficiency.

- Site #6 is a natural pond (with surrounding park) in a stream system draining an urban area, and it supports an appreciable population of ducks fed by local residents. Lead and Total P removals compare favorably to projections. Removal of TSS is appreciably less than projected. A comprehensive analysis of removal efficiency for coliform organisms was conducted at this site. This was not incorporated into the methodology calibration due to the lack of similar data at other sites. It is instructive to note, however, that despite the duck population, average removals for the monitored storms were on the order of 90% for total coliforms, fecal coliforms, and fecal strep.

4.4 EXAMPLE COMPUTATION

A. Given

A 10-acre residential development has a runoff coefficient (Rv) estimated at 0.25. All stormwater runoff from the area is to be routed to a wet pond detention basin.

Space constraints limit the basin dimensions to 25 by 50 ft, or a surface area of 1250 square feet. The basin will have an average depth of 4 feet. Physical storage volume is 5000 cubic feet (CF).

Rainfall statistics for the area are:

			mean	coef. of variation
Volume	(V)	inch	0.53	1.44
Intensity	(I)	in./hr	0.086	1.31
Duration	(D)	hr	7.2	1.09
Interval	(Δ)	hr	85.0	1.00

Particle settling velocities as tabulated in Section 4.3.1 are assumed to apply for this site.

B. Required

Estimate the long-term average reduction in total suspended solids (TSS) in storm runoff that can be obtained from the specified basin size.

C. Procedure

Step 1 - Select appropriate performance curve to use.

- Figure 1 does not apply because removal efficiency by sedimentation varies with flow through rate, as illustrated by Figures 7 and 8
- Figure 2 applies for removal under dynamic conditions
- Figure 3 and 4 apply in this case because storage capacity is provided by the device, and removal by sedimentation also occurs during quiescent conditions between storm events

Step 2 - Compute runoff parameters for mean storm - flow rate (QR) and volume (VR).

$$QR = (I) * (R_V) * (Area) * (43{,}560 / 12)$$
$$= 0.086 * 0.25 * 10 * 3630 = 780 \text{ CFH}$$

$$VR = (V) * (R_V) * (Area) * (43{,}560 / 12)$$
$$= 0.53 * 0.25 * 10 * 3630 = 4807 \text{ CF}$$

Assume that the variability of runoff parameters is the same as for the corresponding rainfall parameters.

$$CV_q = 1.31 \quad \text{and} \quad CV_v = 1.44$$

Step 3 - Compute the removal under DYNAMIC conditions.

The overflow rate during the mean storm (QR / A) is

$$QR / A = 780 / 1250 = 0.62 \text{ ft / hr}$$

Each of the selected size fractions will have a different removal efficiency at the mean flow. Use the appropriate settling velocity in equation (8), or scale from Figure 8 to estimate R_M, the removal at the mean oveflow (QR / A = 0.62).

Fit a straight line approximation for each removal curve in Figure 8 so that it intersects the exact curve at the mean overflow rate (QR/A = 0.62). Estimate the removal efficiency at very low rates (Z in equation 3) from the point where the fitted line intersects the vertical axis.

Then, for each size fraction, use the values obtained above in equation (3), together with the estimate of coefficient of variation of runoff flows to estimate the long-term average removal (R_L).

Alternatively, if estimates of "Z" are 100% for all size fractions (a reasonable estimate in this case), the long-term average removals (R_L) can be scaled directly from Figure 2.

Since the size fractions are mass weighted, the overall TSS removal will be the average of the five size fractions.

Results using the graphic approach are as follows:

Size Fraction	Average Settling Velocity (ft/hr)	R_M (%) (Fig. 8)	R_L (%) (Fig. 2)
1	0.03	5	5
2	0.3	40	23
3	1.5	90	77
4	7.	100	100
5	65.	100	100

OVERALL AVERAGE REMOVAL = 61

fraction NOT removed $f_D = (100 - 61)/100 = 0.39$

Step 4 - Compute the removal under QUIESCENT conditions.

Basin Volume ratio (VB / VR)

$$(VB / VR) = 5000 / 4807 = 1.04$$

The long-term average removal efficiency is defined by Figure 3. This is based on the coefficient of variation of runoff volumes (estimated at 1.44 in Step 2) and the "Effective" Volume ratio (VE/VR), rather than the volume ratio computed immediately above, which is based on physical size of the basin.

The desired ratio (VE/VR) is scaled from Figure 4 using the ratio VB/VR = 1.04 computed above, and the Emptying Rate ratio.

$$E = \Delta * \Omega / VR$$

Δ is the average interval between storms = 85 hr

VR is the mean storm runoff volume = 4807 CF

Ω is the solids removal rate as defined by equation (11) in Section 4.2.2, and is the product of basin surface area (1250 sq ft) and the settling velocity (v_s).

$$\Omega = v_s A$$

Each of the five size fractions has a different settling velocity, and therefore different values for Ω, E, the effective volume ratio VE/VR, and finally the quiescent removal efficiency. The table below lists the results of the foregoing procedure for estimating removals under quiescent settling.

SIZE NO.	FRACTION Vs (ft/hr)	Ω (= Vs A)	E (= $\Delta\Omega$/VR)	VE / VR (Fig. 4)	% REM (Fig. 3)
1	0.0	38	0.7	0.50	35
2	0.3	375	6.6	1.00	54
3	1.5	1875	33.2	1.04	56
4	7	8750	154.7	1.04	56
5	65	81250	1436.7	1.04	56

OVERALL AVERAGE REMOVAL = 51

fraction NOT removed f_Q = (100 - 53) / 100 = 0 .49

Step 5 - Compute the COMBINED removal under both dynamic and quiescent conditions.

Overall removal accomplished by the combination of dynamic and quiescent processes is computed directly from the fractions NOT removed by each process.

Fraction NOT removed by quiescent settling $f_Q = 0.49$

Fraction NOT removed by dynamic settling $f_D = 0.39$

$$\% \text{ Removed (overall)} = [1 - (f_Q * f_V)] * 100\%$$

$$= [1 - (0.49 * 0.39)] * 100\%$$

$$= 81\ \%$$

A careful examination of the results is instructive. As the following summary table indicates, the quiescent process has a lesser effectiveness for the removal of particles with the higher settling velocities, compared with dynamic removals. This is not because the process provides less efficient sedimentation. It is a result of the fact that for a basin volume about equal to the mean storm runoff volume (VB/VR = 1.04), a significant percentage of storm event runoff volumes are greater than the basin capacity. The indicated quiescent removals reflect the fact that some fraction of the total runoff does not remain in the basin to undergo quiescent settling.

The efficiency and importance of the quiescent process is reflected by its significantly higher effectiveness in removing the slower settling fractions.

SIZE NO.	FRACTION Vs (ft/hr)	% REMOVAL DYNAMIC	% REMOVAL QUIESCENT	% REMOVAL COMBINED
1	0.0	5	35	38
2	0.3	23	54	65
3	1.5	77	56	90
4	7	100	56	100
5	65	100	56	100
	ALL	61	51	81

4.5 DISCUSSION

On the basis of the comparisons between observed and predicted performance (presented in Figure 10) the analysis methodology described earlier appears to provide sufficiently reliable estimates of performance for use in planning activities. More refined computations, which do not require some of the approximations and assumptions used in the probabilistic methodology, are certainly possible. SWMM and some other deterministic models have this capability, and it would be interesting and useful to compare projections. It should be noted however, as a close scrutiny of observed performance (Table 2) will indicate, that because of either limited data sets or complex site-specific factors, or both, actual observed performance does not conform to a consistent pattern. It is suggested that other, more refined computations are likely to reflect similar levels of uncertainty when compared with actual performance data.

The discussion of the outliers in the comparison between observed and computed performance serves two purposes. First, by identifying site factors that can reasonably be expected to cause anomalous results, it adds credibility to the analysis methodology. Second, it highlights the fact that competing processes are at work in wet pond detention basins that may enhance or degrade removal of specific pollutants.

It is tempting to consider an extension of this methodology (or other analysis methodologies) to incorporate biological or other processes that are also obviously at work in at least some stormwater detention basins. The available data were considered inadequate to support a meaningful extension of the analysis at this time, although the means for doing so are clear. Biological or other decay mechanisms are typically expressed as rate coefficients with units of the reciprocal of time (e.g., 1/day). Such rates, for which reasonable estimates can be derived from the literature or specific studies, can be converted to a psuedo-settling velocity (or vice-versa per equation 10). With additional data, this would be a worthwhile effort due to the significance of mechanisms other than sedimentation in stormwater basins.

5.0
GENERAL PERFORMANCE PROJECTIONS

The analysis methodology described in Section 2 provides a basis for relating the size of a retention basin to its average performance as a stormwater quality control device, accounting for the intermittent and highly variable character of urban stormwater runoff. The calibration results presented indicate that performance projections, while not precise, are quite adequate approximations for use in planning activities. Because the calibration analysis covered a very wide range of physical basin types and sizes relative to the hydraulic loads applied, it is reasonable to consider the model suitable for use in a generalized analysis.

A generalized analysis is desirable because it addresses the following issues:

- Transferability: If information derived from a limited set of site specific monitoring data can be extended to other areas and other situations, its value is greatly enhanced. Transferrability of data and information was an important objective of the NURP effort.
- Adjustment: Monitoring programs appropriately emphasize conditions of higher stress which maximize the information content of a set of data. In this context, the storms monitored were consistently biased toward more severe events. Thus, for all test sites, the average of monitored storm events was significantly larger than the long-term average for all storms each particular basin can expect to treat. As a result, long-term performance will be better (perhaps appreciably) than performance under test conditions.
- Utility: NURP's emphasis was on planning tools, as opposed to a design or research emphasis. Accordingly, the information which can be developed should be structured in a format which assists planning activities.

In the results presented below, the analysis methodology is applied using rainfall characteristics as the basic input because long-term records are available for all areas of the country. Rainfall is converted to runoff parameters by applying a runoff coefficient, estimates of which are available from both NURP data and prior literature.

There are regional and local differences in rainfall patterns. Depending on the size and development of an urban area, runoff coefficients will vary. Feasible local options for basin surface area and depth will vary. Further, soluble fractions of certain pollutants may vary from site to site, as may typical particle sizes and settling velocities in urban runoff. Because of the foregoing, local analyses using site specific conditions are the most appropriate approach. Some general perspectives are possible, however, provided that it is recognized that local factors may modify results.

There are local differences in rainfall patterns within a region; however, based on rainfall records for 50 or more cities analyzed under the NURP program, fairly typical regional rainfall characteristics can be assigned (see Appendix Figure A-2). Detention basin performance for these rainfall patterns, for basins which have an average depth of 3.5 feet, and catchments which have a runoff coefficient of 0.2 are illustrated by Figure 11. The comparisons are based on TSS removal. The depth value shown is an average value: in effect, it defines the relationship between surface area and volume and is typical of the units in the NURP data base which has been analyzed. The runoff coefficient used is estimated, based on NURP data analyzed, to be fairly typical of the average for a large urbanized area. This figure, therefore, illustrates the order of differences in performance characteristics which can result from regional differences in rainfall patterns.

In Figure 11, and the other figures which follow, basin size is expressed as a (percentage) ratio between the surface area of the basin and the contributory urban drainage area. For example, an area ratio of 0.10% on the horizontal axis reflects a basin with a surface area of 0.64 acres serving a 1-square-mile (640-acre) urban drainage area. The performance relationships could alternatively be expressed in terms of basin volumes, although depth would also have to be shown in such a case because performance depends on both area and volume provided.

Figure 12 illustrates the effect of increasing average basin depth, and hence volume, using the Rocky mountain area rainfall statistics. Comparisons are based on TSS removal. Note that, for basins which provide area ratios in the order of 0.10%, doubling the volume (7 versus 3.5 foot depth) may improve removal efficiency as much as 20%. However, for relatively large basins, increased depth improves performance only marginally.

Since detention basin performance depends on runoff, rather than the rainfall which must be used for long-term projections, the runoff coefficient assigned (ratio of runoff to rainfall) is quite important. The value of 0.2 assigned in Figure 12 is estimated to be a representative value of an average for broad urbanized areas, and hence useful in providing an estimate of overall areawide requirements. However, the procedure may also be used to identify detention basin requirements for smaller, specific urban areas. In such cases, the runoff coefficient may either be lower (low density residential areas) or higher (commercial, very high density residential). The significant effect of runoff coefficients on performance in shown by Figure 13, using rainfall characteristics typical of the Northeast, and TSS removal for the comparison.

A set of detention basin performance charts may be developed using the NURP analysis methodology, and appropriate local factors, to provide a working guide for planning decisions. The previous performance charts were based only on TSS removal to simplify the comparisons which were made. For planning activities, however, estimates of removals for other pollutants of interest would be desireable.

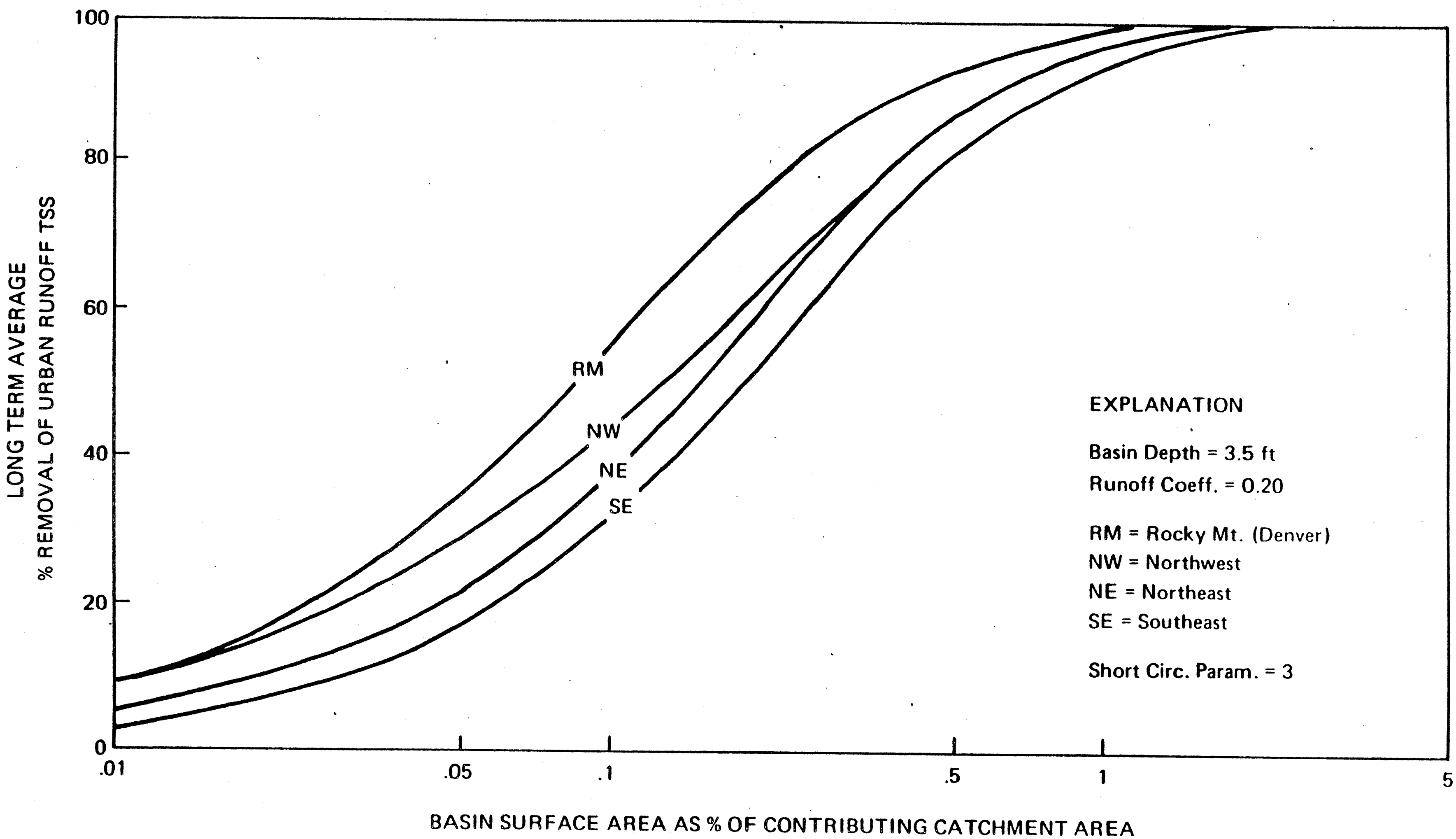


Figure 11. Regional differences in detention basin performance

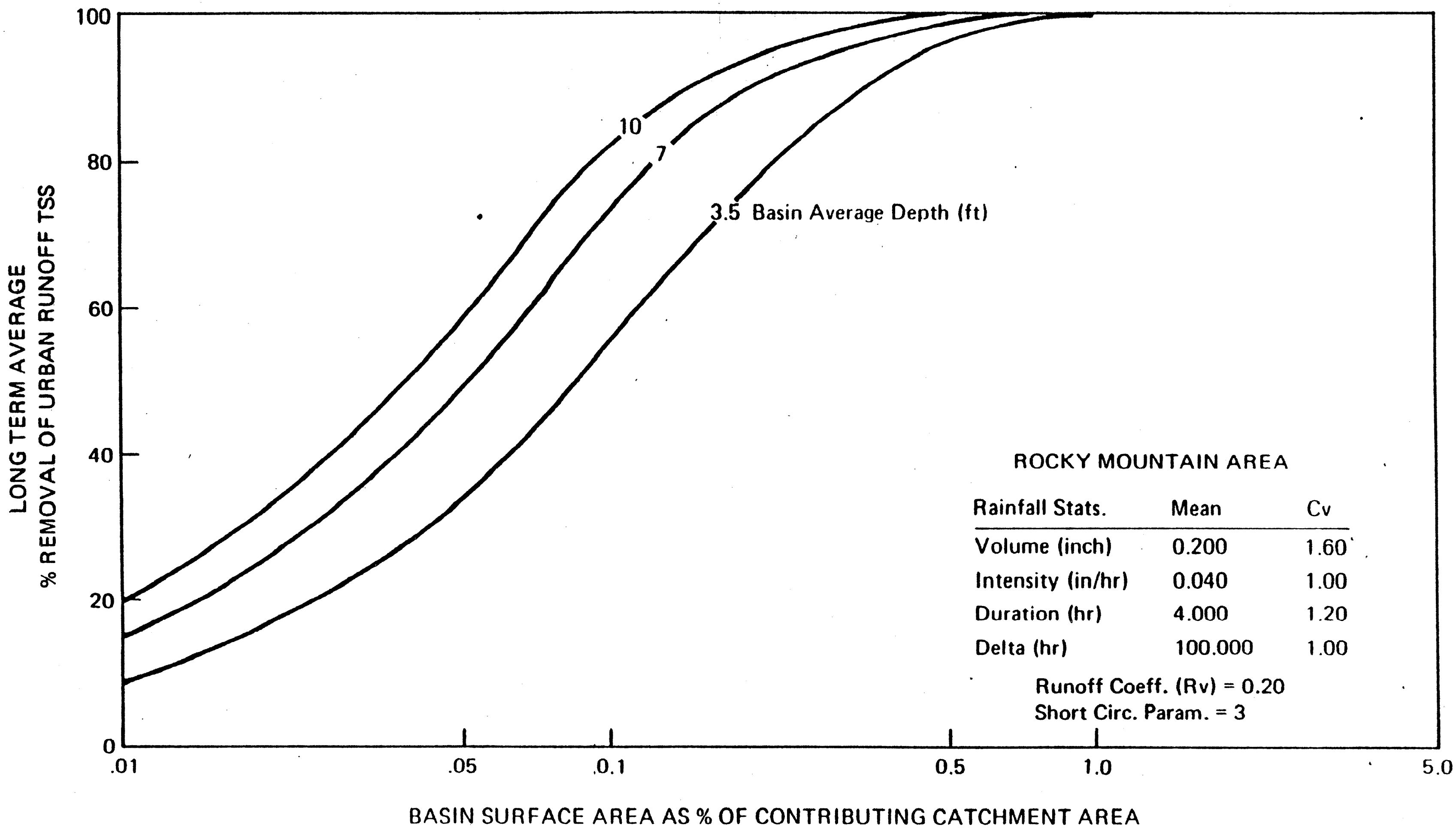


Figure 12. Effect of depth (volume) on performance

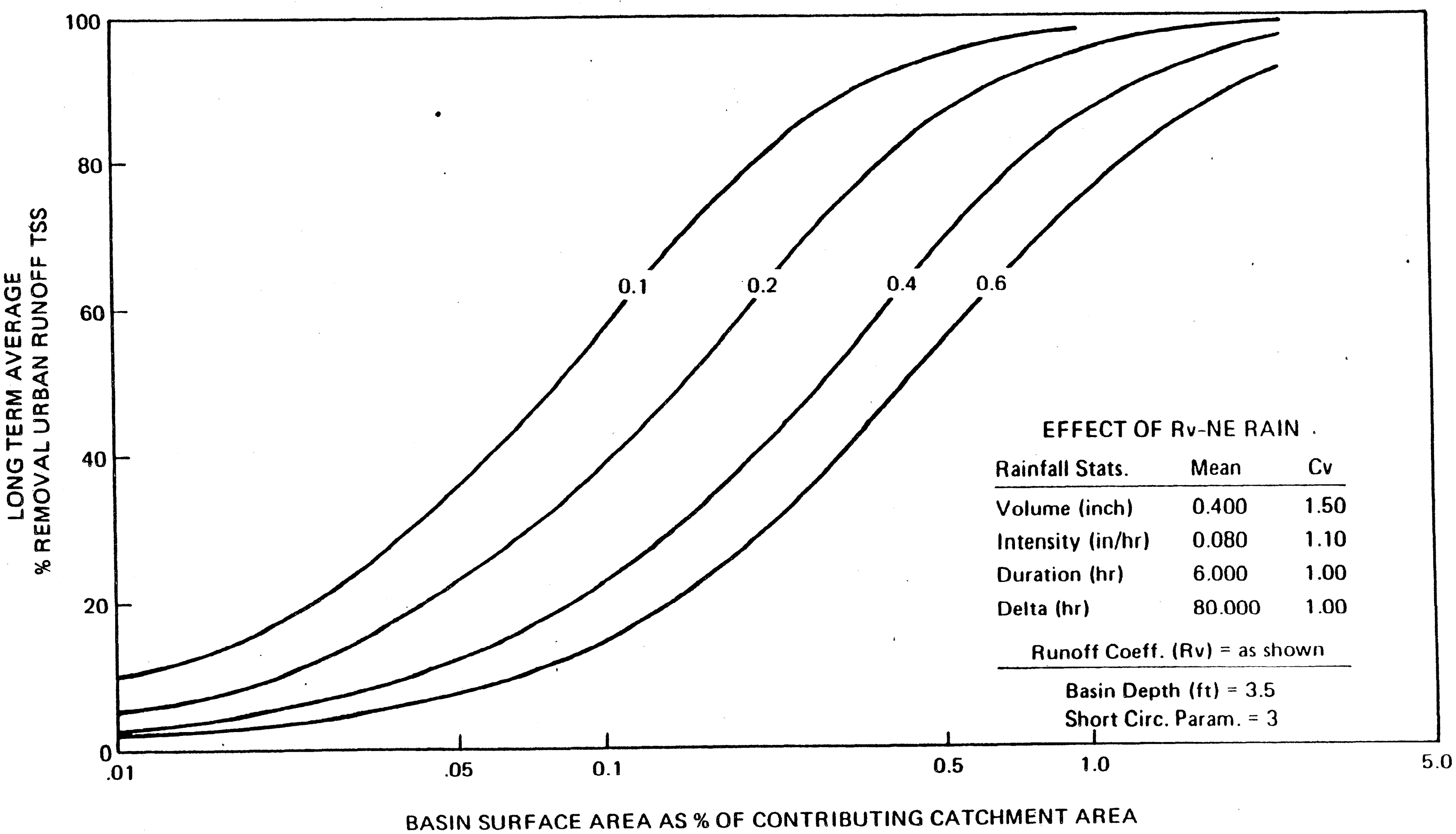


Figure 13. Effect of runoff coefficient on performance

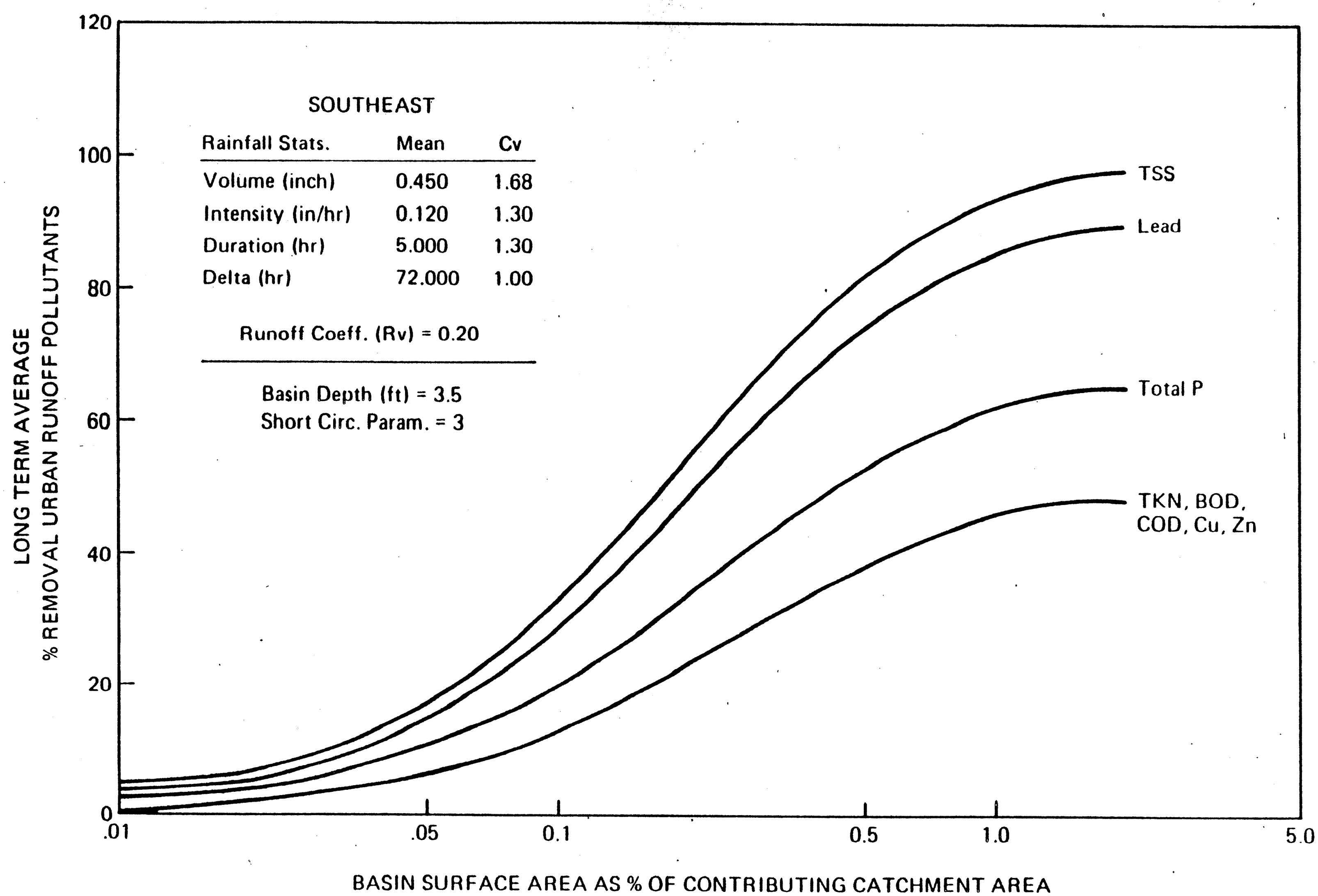


Figure 14. Detention basin performance

An illustration of such a chart is presented by Figure 14, using Southeast rainfall patterns, a basin average depth of 3.5 feet, a runoff coefficient of 0.20, and the particulate fraction of specific pollutants developed in the calibration analysis. The particulate fractions for lead (0.9) and total P (0.67) employed for this projection are typical values for urban runoff, based on the NURP data base. For TKN, Cu, Zn, BOD and COD, the estimates of particulate fraction (0.5) are based on more limited NURP data and are less certain.

In the absence of appropriate local data, the NURP estimates derived from a very large data base would provide the best estimate. However, where a local monitoring program is planned, such estimates and performance projections can be refined if the relevant analytical determinations are incorporated into the monitoring program.

1. USEPA. 1980. Data Management Procedures Manual, NURP document, Washington, D.C., March.

2. Hydroscience, Inc. 1979. A Statistical Method for the Assessment of Urban Stormwater, for USEPA NonPoint Sources Branch, EPA 440/3-79-023, May.

3. DiToro, D.M. and M.J. Small. 1979. Stormwater Interception and Storage, Journal of the Environmental Engineering Division, ASCE, Vol. 105, No. EE1, Proc. Paper 14368, February.

4. Small, M.J. and D.M. DiToro. 1979. Stormwater Treatment Systems, Journal of the Environmental Engineering Division, ASCE, Vol. 105, No. EE3, Proc. Paper 14617, June.

5. Fair, G.M. and J.C. Geyer. 1954. Water Supply and Waste Water Disposal, John Wiley and Sons.

6. USEPA. 1982. Detention Basins for Control of Urban Stormwater Quality, Washington, D.C., Water Planning Division, First Draft, September.

7. Whipple, W., Jr. and J.V. Hunter. 1981. Settleability of Urban Runoff Pollution, Water Pollution Control Federation Journal, 53(12), pp. 1726-1731, December.

1.0 GENERAL

This Appendix presents information on representative values for parameters used in the computations. It is intended to serve as a reference that will permit the user to make preliminary estimates for use in a screening analysis, and for comparing local values against those developed from a broader data base.

2.0 RAINFALL STATISTICS

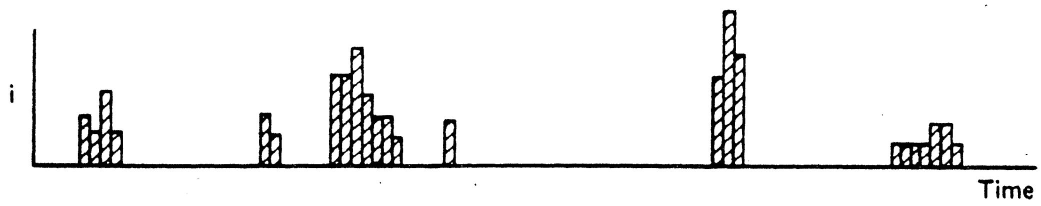
Long-term rainfall patterns for an area are recorded in the hourly precipitation records of rain gages maintained by the U.S. Weather Service (USWS). The analysis procedures used in this manual are based on the statistical characteristics of storm "events." As illustrated by Figure A-1, the hourly record may be converted to an "event" record by the specification of a minimum number of dry hours that defines the separation of storm events. Routine statistical procedures are then used to compute the statistical parameters (mean, standard deviation, coefficient of variation) of all events in the record for the rainfall properties of interest.

A computer program, SYNOP, documented in a publication of EPA's Nationwide Urban Runoff Program (NURP), computes the desired statistics from rainfall data tapes obtainable from USWS. It generates outputs based on the entire record, and also on a stratification of the record by month, which is convenient for evaluating seasonal differences.

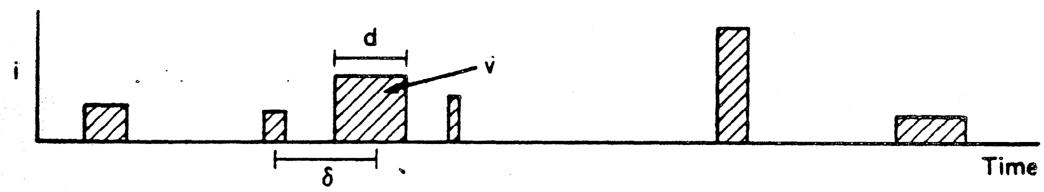
Table A-1 summarizes the statistics for storm event parameters for rain gages in selected cities distributed throughout the country. These data may be used to guide local estimates, pending analysis of specific data based on a site-specific rain gage. The tabulations provide values for mean and coefficient of variation for storm event volumes, average intensities, durations, and intervals between storm midpoints. The cities for which results have been tabulated are grouped by region of the country. Results are presented for both the long-term average of all storms, and for the June through September period that is often the critical period for receiving water impacts.

Figure A-2 provides initial estimates of storm event characteristics for broad regions of the country, based on data in the foregoing table.

(a) HOURLY RAINFALL VARIATION



(b) STORM EVENT VARIATION



	PARAMETER			
	For each storm event		For all storm events	
			Mean	Coef Var
Volume	v	(inches)	V	ν_v
Duration	d	(hours)	D	ν_d
Average intensity	i	(inch/hour)	I	ν_i
Interval between event midpoints	δ	(hours)	Δ	ν_δ

Figure A-1. Characterization of a rainfall record

Table A-1. RAINFALL EVENT CHARACTERISTICS FOR SELECTED CITIES

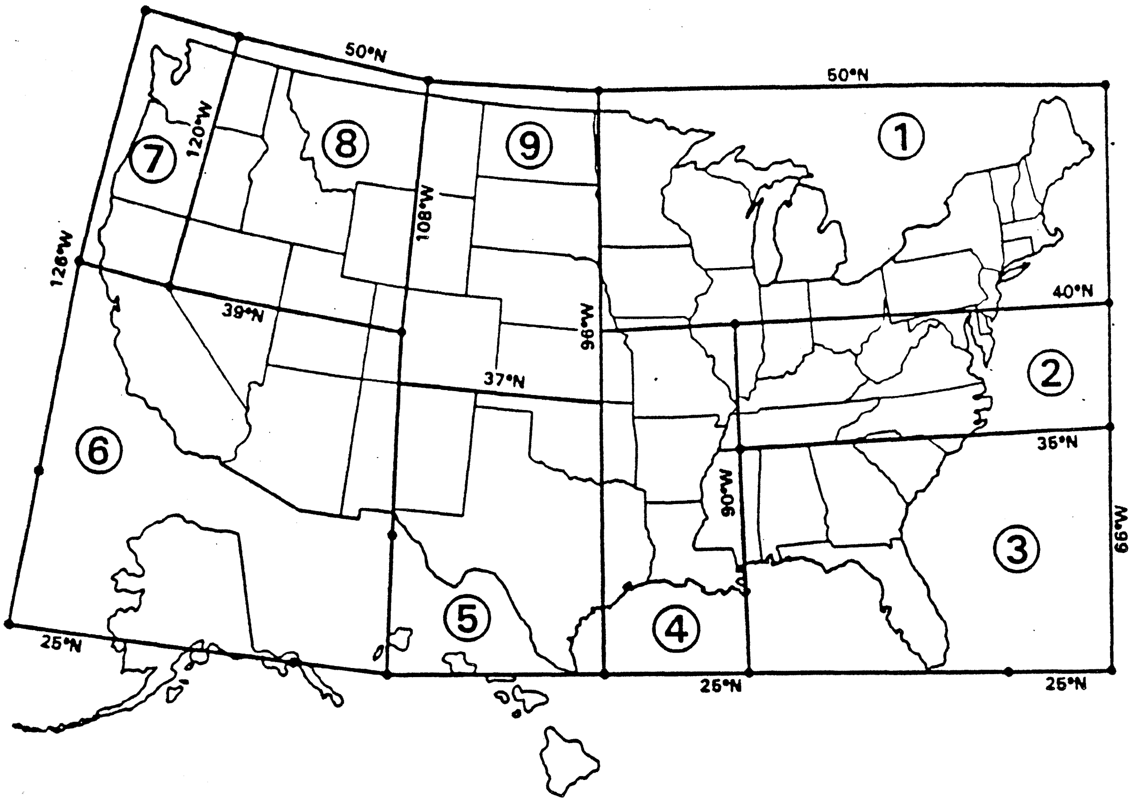
	Annual								June to September							
	Mean				Coefficient of Variation				Mean				Coefficient of Variation			
Location	V	I	D	Δ	ν_v	ν_i	ν_d	ν_Δ	V	I	D	Δ	ν_v	ν_i	ν_d	ν_Δ
Great Lakes																
Champaign-Urbana, IL	0.35	.063	6.1	80	1.47	1.37	1.02	1.02	0.45	.102	4.6	87	1.44	1.22	1.01	1.05
Chicago, IL (3)	0.27	.053	4.4	62	1.44	1.58	1.06	1.12	0.33	.091	6.2	67	1.49	1.37	1.00	1.13
Chicago, IL (5)	0.27	.053	5.7	72	1.59	1.54	1.08	1.00	0.37	.090	4.5	76	1.42	1.37	1.04	1.02
Davenport, IA	0.38	.077	6.6	98	1.37	1.24	1.40	1.01	0.49	.112	5.3	91	1.32	1.14	1.22	0.94
Detroit, MI	0.21	.050	4.4	57	1.59	1.16	1.02	1.07	0.27	.095	3.1	64	1.43	1.32	0.82	1.14
Louisville, KY	0.38	.064	6.7	76	1.45	1.42	1.08	1.00	0.36	.094	4.5	78	1.40	1.31	1.01	1.04
Minneapolis, MN	0.24	.043	6.0	87	1.48	1.22	1.08	0.98	0.34	.075	4.5	74	1.34	1.26	1.00	0.92
Steubenville, OH	0.31	.057	7.0	79	1.28	1.03	1.39	1.00	0.39	.094	5.9	88	1.28	1.27	1.76	0.95
Toledo, OH	0.22	.048	5.0	62	1.52	1.16	0.99	1.03	0.29	.083	3.7	69	1.43	1.37	1.93	1.06
Zanesville, OH	0.30	.061	6.1	77	1.24	1.01	0.93	1.03	0.36	.100	4.3	80	1.23	1.11	0.95	1.06
Lansing, MI (5)(30 yr)	0.21	.041	5.6	62	1.56	1.55	1.10	1.02	0.29	.073	4.2	71	1.39	1.25	0.98	1.00
Lansing, MI (5)(21 yr)	0.26	.047	6.2	87	1.42	1.42	0.95	1.00	0.34	.078	5.1	89	1.25	1.13	0.90	0.98
Ann Arbor, MI (5)																
Lower Mississippi Valley																
Memphis, TN	0.52	.086	6.9	89	1.36	1.31	1.07	1.01	0.44	.112	4.7	88	1.35	1.28	1.12	1.06
New Orleans, LA (8)	0.61	.113	6.9	89	1.46	1.40	1.24	1.02	0.53	.142	5.0	65	1.40	1.42	1.34	1.08
Shreveport, LA (9)(17 yr)	0.54	.080	7.8	110	1.39	1.27	1.09	0.99	0.49	.105	5.3	109	1.50	1.27	1.28	1.09
Lake Charles, LA (10)	0.66	.108	7.7	109	1.64	1.40	1.26	0.99	0.63	.130	5.9	86	1.90	1.41	1.43	0.99
Average	0.58	.097	7.3	99	1.46	1.35	1.17	1.00	9.52	.122	5.2	87	1.54	1.35	1.29	1.06
Texas																
Abilene, TX	0.32	.083	4.2	128	1.52	1.24	1.01	1.45	0.42	.121	3.3	114	1.56	1.32	0.98	1.46
Austin, TX	0.33	.078	4.0	96	1.88	1.53	1.06	1.44	0.38	.106	3.3	108	1.82	1.71	1.02	1.49
Brownsville, TX	0.27	.072	3.5	1.09	2.02	1.43	1.20	1.50	0.33	.104	2.8	101	1.94	1.33	1.30	1.67
Dallas, TX	0.39	.079	4.2	100	1.64	1.23	1.00	1.32	0.38	.100	3.2	111	1.65	1.24	1.01	1.44
Waco, TX	0.36	.086	4.2	1.06	1.66	1.40	1.08	1.36	0.40	.117	3.3	124	1.60	1.34	1.07	1.39
Average	0.33	0.080	4.0	108	1.74	1.37	1.07	1.41	0.38	.110	3.2	112	1.71	1.39	1.08	1.49

Table A-1. RAINFALL EVENT CHARACTERISTICS FOR SELECTED CITIES (continued)

	Annual								June to September							
	Mean				Coefficient of Variation				Mean				Coefficient of Variation			
Location	V	I	D	Δ	ν_v	ν_i	ν_d	ν_Δ	V	I	D	Δ	ν_v	ν_i	ν_d	ν_Δ
Northeast																
Caribou, ME	0.21	.034	5.8	55	1.58	0.97	1.03	1.03	0.24	.054	4.4	55	1.64	1.15	1.00	1.01
Boston, MA	0.33	.044	6.1	68	1.67	1.02	1.03	1.06	0.30	.063	4.2	73	1.80	1.20	1.12	1.12
Lake George, NY	0.23	.067	5.4	76	1.26	1.98	0.91	1.48	0.27	.076	4.5	72	1.25	1.61	0.86	1.44
Kingston, NY	0.37	.052	7.0	80	1.35	1.01	0.91	0.98	0.35	.073	5.0	79	1.46	1.27	1.00	1.08
Poughkeepsie, NY	0.35	.052	6.9	81	1.31	0.95	0.87	0.95	0.36	.081	4.9	82	1.48	1.16	0.96	1.00
New York City, NY	0.37	.053	6.7	77	1.37	1.04	0.93	0.89	0.30	.076	4.8	75	1.51	1.28	1.03	0.95
Mineola LI, NY	0.43	.088	5.8	89	1.34	1.14	1.30	0.99	0.41	.114	4.5	88	1.42	1.17	1.48	1.03
Upton LI, NY	0.43	.076	6.3	81	1.42	1.06	1.09	0.99	0.42	.101	4.6	88	1.56	1.10	1.23	1.02
Wantagh LI, NY (2 YR)	0.40	.075	5.6	83	1.54	1.24	1.03	1.03	0.34	.091	4.0	74	1.59	1.08	1.28	0.99
Long Island, NY	0.41	.126	4.2	93	1.35	1.30	1.12	1.72	0.41	.127	3.4	99	1.52	1.15	1.21	1.57
Washington, D.C.	0.36	.067	5.9	80	1.45	1.18	1.03	1.00	0.41	.107	4.1	78	1.67	1.38	1.10	1.06
Baltimore, MD (3)	0.40	.069	6.0	82	1.48	1.21	1.01	1.03	0.43	.107	4.2	79	1.66	1.49	1.08	1.08
Southeast																
Greensboro, NC	0.32	.067	5.0	67	1.40	1.44	1.11	1.18	0.34	.093	3.6	62	1.67	1.43	1.20	1.19
Columbia, SC	0.38	.102	4.5	68	1.55	1.59	1.13	1.18	0.41	.153	3.4	58	1.59	1.68	1.25	1.13
Atlanta, GA	0.50	.074	8.0	94	1.37	1.16	1.11	0.93	0.45	.100	6.2	87	1.43	1.27	1.31	0.97
Birmingham, ALA	0.53	.086	7.2	85	1.44	1.31	1.09	1.00	0.45	.111	5.0	76	1.47	1.33	1.18	1.01
Gainesville, FLA	0.64	.139	7.6	106	1.35	1.14	1.66	1.06	0.65	.161	6.6	70	1.41	1.13	1.65	0.92
Tampa, FLA	0.40	.110	3.6	93	1.63	1.21	1.11	1.10	0.44	.138	3.1	49	1.70	1.28	1.28	1.01
Average	0.49	.102	6.2	89	1.47	1.28	1.22	1.05	0.48	.133	4.9	68	1.52	1.34	1.33	1.01

Table A-1. RAINFALL EVENT CHARACTERISTICS FOR SELECTED CITIES (concluded)

	Annual								June to September							
	Mean				Coefficient of Variation				Mean				Coefficient of Variation			
Location	V	I	D	Δ	ν_v	ν_i	ν_d	ν_Δ	V	I	D	Δ	ν_v	ν_i	ν_d	ν_Δ
Rocky Mountains																
Denver, CO (3) 8 YRS	0.15	.033	4.3	97	2.00	1.58	1.24	1.25	0.18	.053	3.2	82	1.90	1.44	1.20	1.26
Denver, CO (3) 25 YRS	0.15	.033	4.8	101	1.73	1.07	1.20	1.15	0.15	.055	3.2	80	1.85	1.51	1.20	1.05
Denver, CO (13) 24 YRS	0.22	.032	9.1	144	1.49	1.13	1.15	0.92	0.22	.053	4.4	101	1.78	1.53	1.35	0.23
Rapid City, SD (3)	0.15	.039	4.0	86	1.81	1.63	1.21	1.33	0.20	.063	3.0	75	1.63	1.36	1.08	1.20
Rapid City, SD (12)	0.20	.033	8.0	127	1.46	1.09	1.24	0.95	0.25	.059	6.1	101	1.50	1.46	1.39	0.94
Salt Lake City, UT (3)	0.14	.031	4.5	94	1.42	0.91	0.92	1.39	0.14	.041	2.8	125	1.51	1.13	0.80	1.41
Salt Lake City, UT (3) (2 GAGES)	0.18	.025	7.8	133	1.32	1.06	0.85	0.97	0.16	.031	6.8	164	1.43	1.06	1.01	0.98
Average (2)	0.15	.036	4.4	94	1.77	1.35	1.20	1.24	0.18	.059	3.1	78	1.74	1.44	1.14	1.13
California																
Oakland, CA	0.19	.033	4.3	320	1.62	0.74	1.03	1.60	0.11	.020	2.9	756	1.63	0.56	1.00	1.09
San Francisco, CA (75)	0.78	.017	59	515	1.45	0.89	1.37	0.72	0.14	.017	11.2	830	1.46	0.70	1.67	0.75
Southwest																
El Paso, TX	0.15	.047	3.3	226	1.54	1.12	1.07	1.43	0.19	.069	2.6	142	1.68	1.28	1.20	1.44
Phoenix, AZ	0.17	.055	3.2	286	1.38	1.26	0.97	1.42	0.21	.090	2.4	379	1.51	1.64	0.84	1.25
Average	0.17	.045	3.6	277	1.51	1.04	1.02	1.48	0.17	.060	2.6	425	1.61	1.16	1.01	1.26
Northwest																
Portland, OR (3) 25 YRS	0.17	.017	5.4	60	1.60	0.85	1.00	1.47	0.15	.019	4.5	109	1.45	0.99	0.95	1.64
Portland, OR (10) 10 YRS	0.36	.023	15.5	83	1.51	0.79	1.09	1.32	0.22	.027	9.4	179	1.32	1.33	1.13	1.20
Eugene, OR (6)	0.39	.030	10.9	73	1.85	0.87	1.25	1.74	0.21	.033	6.3	167	1.32	1.01	1.05	1.49
Eugene, OR (15)	0.63	.026	23.1	118	1.88	0.88	1.35	1.30	0.28	.029	12.0	226	1.28	1.07	1.72	1.20
Eugene, OR (20)	0.72	.025	29.2	136	1.85	0.91	1.34	1.19	0.31	.027	15.0	250	1.24	1.15	1.19	1.11
Seattle, WA (15)	0.46	.023	21.5	101	1.45	0.86	1.26	1.02	0.29	.024	12.7	159	1.45	0.92	1.24	1.04
Average	0.48	.024	20.0	101	1.61	0.84	1.23	1.21	0.26	.027	11.4	188	1.35	1.11	1.20	1.15



ZONE	PERIOD	RAINFALL STATISTICS							
		VOLUME (IN)		INTENSITY (IN/HR)		DURATION (HR)		INTERVAL (HR)	
		MEAN	C.V.	MEAN	C.V.	MEAN	C.V.	MEAN	C.V.
1	ANNUAL	0.26	1.46	0.051	1.31	5.8	1.05	73	1.07
	SUMMER	0.32	1.38	0.082	1.29	4.4	1.14	76	1.07
2	ANNUAL	0.36	1.45	0.066	1.32	5.9	1.05	77	1.05
	SUMMER	0.40	1.57	0.101	1.37	4.2	1.09	77	1.08
3	ANNUAL	0.49	1.47	.102	1.28	6.2	1.22	89	1.05
	SUMMER	0.48	1.52	.133	1.34	4.9	1.33	68	1.01
4	ANNUAL	0.58	1.46	.097	1.35	7.3	1.17	99	1.00
	SUMMER	0.52	1.54	.122	1.35	5.2	1.29	87	1.06
5	ANNUAL	0.33	1.74	.080	1.37	4.0	1.07	108	1.41
	SUMMER	0.38	1.71	.110	1.39	3.2	1.08	112	1.49
6	ANNUAL	0.17	1.51	.045	1.04	3.6	1.02	277	1.48
	SUMMER	0.17	1.81	.060	1.16	2.6	1.01	425	1.26
7	ANNUAL	0.48	1.61	0.024	0.84	20.0	1.23	101	1.21
	SUMMER	0.26	1.35	0.027	1.11	11.4	1.20	188	1.15
8	ANNUAL	0.14	1.42	.031	0.91	4.5	0.92	94	1.39
	SUMMER	0.14	1.51	.041	1.13	2.8	0.80	125	1.41
9	ANNUAL	0.15	1.77	.038	1.35	4.4	1.20	94	1.24
	SUMMER	0.18	1.74	.059	1.44	3.1	1.14	78	1.13

Figure A-2. Representative regional values for preliminary estimates

From the statistics of the storm event parameters, other values of interest may be determined.

The ratio of mean storm duration (D), to the mean interval between storms (Δ), reflects the percent of the time that storm events are in progress:

$$\% \text{ time that it is raining} = \frac{D}{\Delta}$$

The average number of storms during any period of time is defined by the ratio between the total number of hours in the selected period and the average interval between storms (Δ). For example, on an annual basis:

$$\text{Avg. number of storms per year} = \frac{365 * 24}{\Delta}$$

The storm event parameters of interest have been shown to be well represented by a gamma distribution, and the results listed in Table A-1 indicate that the coefficient of variation of the event parameters generally falls between 1.0 and 1.5. Figure A-3 plots the probability distribution of gamma distributed variables with coefficients of variation of 1.0, 1.25, and 1.5, in terms of probability of occurrence as a function of the magnitude, expressed as a multiple of the mean. This plot can be used to approximate the magnitude of an event with a specified frequency of occurrence.

For example, consider a site where storm events have volume statistics for mean and coefficient of variation of 0.4 inch, and 1.5 respectively. Figure A-3 can be used to estimate that 1 percent of all storm events have volumes that exceed about 7.5 times the mean (or 7.5 * 0.4 = 3 inches). If the same location has an average interval between storms (Δ) of 87.5 hours, there will be an average of:

$$(365 * 24) / 87.5 = 100 \text{ events/year}$$

and the 1 percentile event (3 inches) reflects a storm volume exceeded on average, once per year.

3.0 RUNOFF COEFFICIENT (Rv)

Runoff coefficient is defined as the fraction of rainfall that appears as surface runoff. The substantial data base developed under EPA's NURP program indicated that runoff coefficient varied from event to event at any site. Variations were not significantly correlated with storm size or intensity and can be treated as random. The median value for a site was best estimated by the percent of impervious surface in the drainage area.

Figure A-4 illustrates the relationship between the median runoff coefficient observed at an urban site and the percent of impervious area in the catchment.

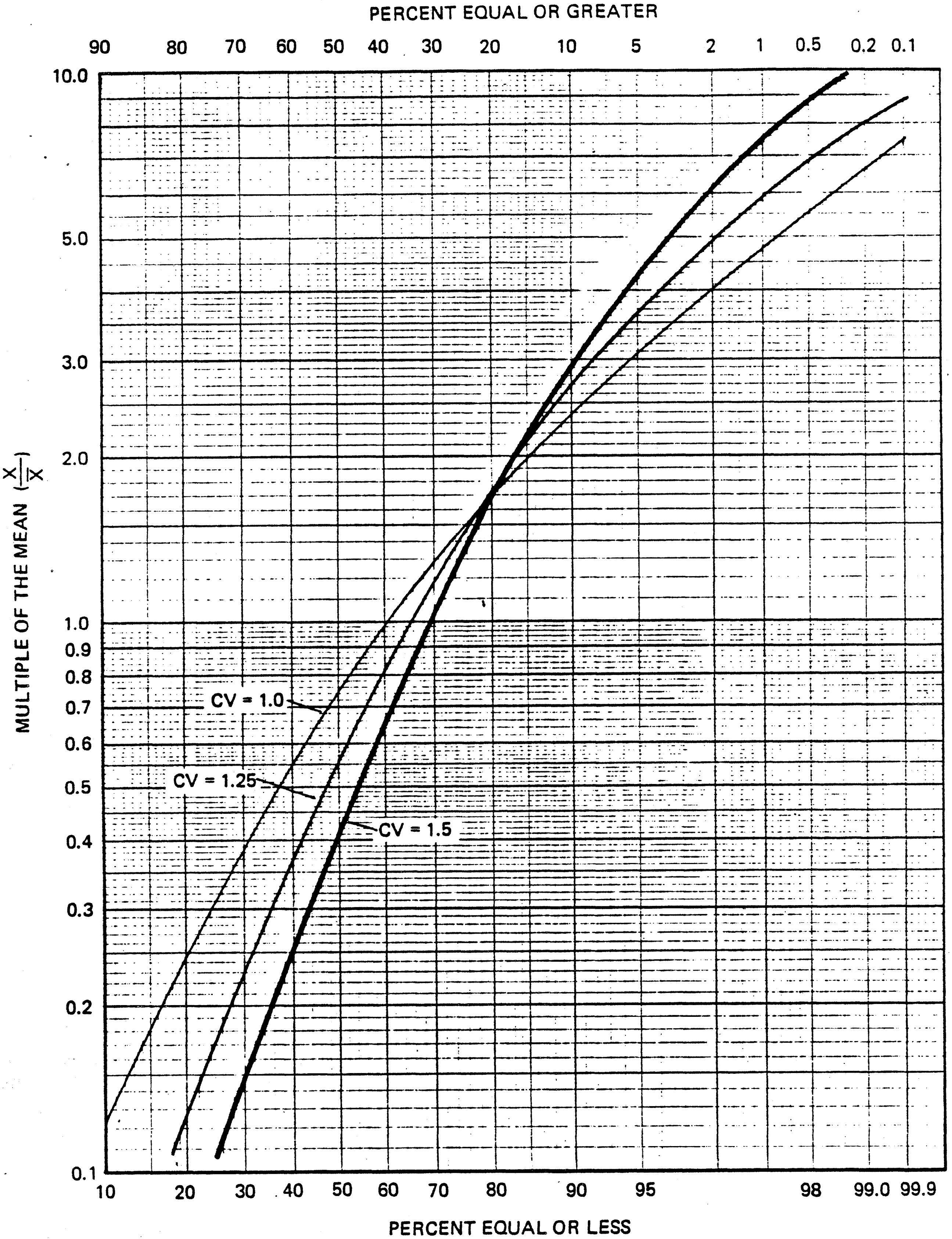


Figure A-3. Probability distribution for a variable with a gamma distribution

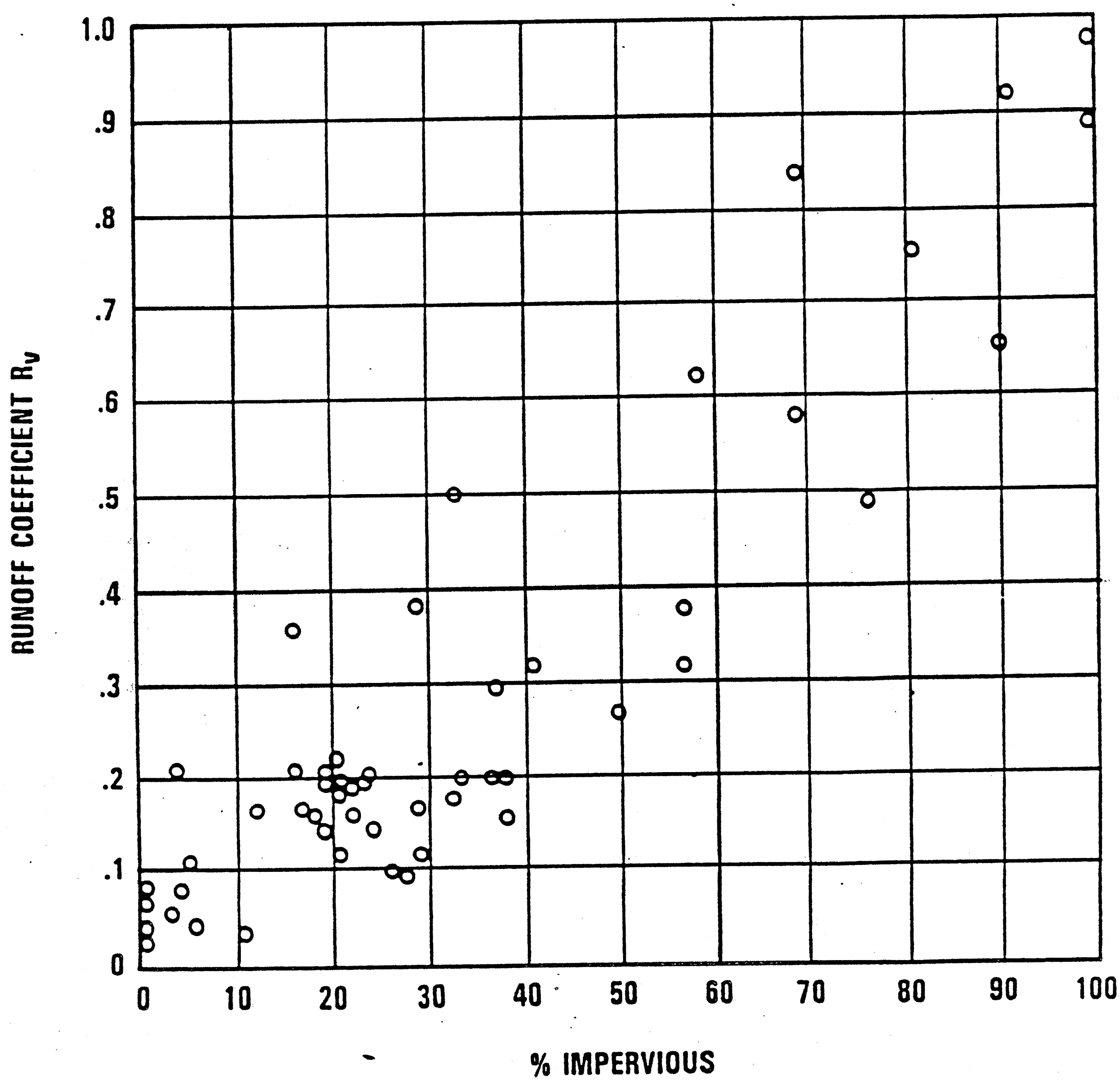


Figure A-4. Relationship between percent impervious area and median runoff coefficient

This information may be used to guide estimates of the surface runoff routed to a detention basin during storm events.

4.0 SETTLING VELOCITIES

The settling velocity of particulates in urban runoff is a key determinant of the efficiency of pollutant removals by sedimentation. Settling velocity measurements were conducted on approximately 50 different runoff samples from seven urban sites. These data may be used to guide estimates in the absence of local settling column study results.

There is a wide range of particle sizes, and hence settling velocities, in any sample of stormwater runoff. This range can be described by a probability distribution of pollutant settling velocities and determined by an appropriate analysis of the data obtained from standard settling column tests, as described further below. When the settling velocity distributions obtained from the NURP studies were analyzed, it was found that there were differences between separate storms at a site, and differences between individual storms at different sites. Site-to-site differences were of the same order as storm-to-storm variations at a particular site, justifying the combination of all data. The result of such an analysis, illustrated by Figure A-5, indicated that it is reasonable to make estimates of "typical" urban runoff settling characteristics and expect that, in an appropriate analysis, short-term variations will average out. This assumption and the relationship shown, proved to work out quite well in the analysis of the performance of nine different detention basins in different parts of the country and differing radically in size.

For analysis purposes, the indicated range of settling velocities can be broken down into five equal fractions that have the characteristics listed in Section 4 of this document.

While the "typical" values provided here are considered to be satisfactory for initial estimates, and for screening analyses, additional settling column studies are encouraged to expand the data base and improve site-specific estimates. The test procedure is quite simple, and utilizes equipment and procedures that have been in general use for many years and frequently applied in water and waste treatment applications. The only difference is the technique suggested for analyzing the data to increase its utility for stormwater runoff applications.

The equipment and procedure are shown schematically by Figure A-6. The settling column, typically lucite and about 6 inches in diameter by 6 feet high, is fitted with a series of sample ports. It is filled with the runoff sample, then small samples are withdrawn from the ports at scheduled intervals of time. Concentrations of pollutants of interest are compared with the initial concentration and the pattern of percent removal versus port depth (H) and time (T) is determined. Since each port depth and sample time corresponds to a settling velocity, each measurement (expressed as percent removal) can be interpreted as the percent of the total that have settling velocities equal to or greater than that characterized by port location and sampling time.

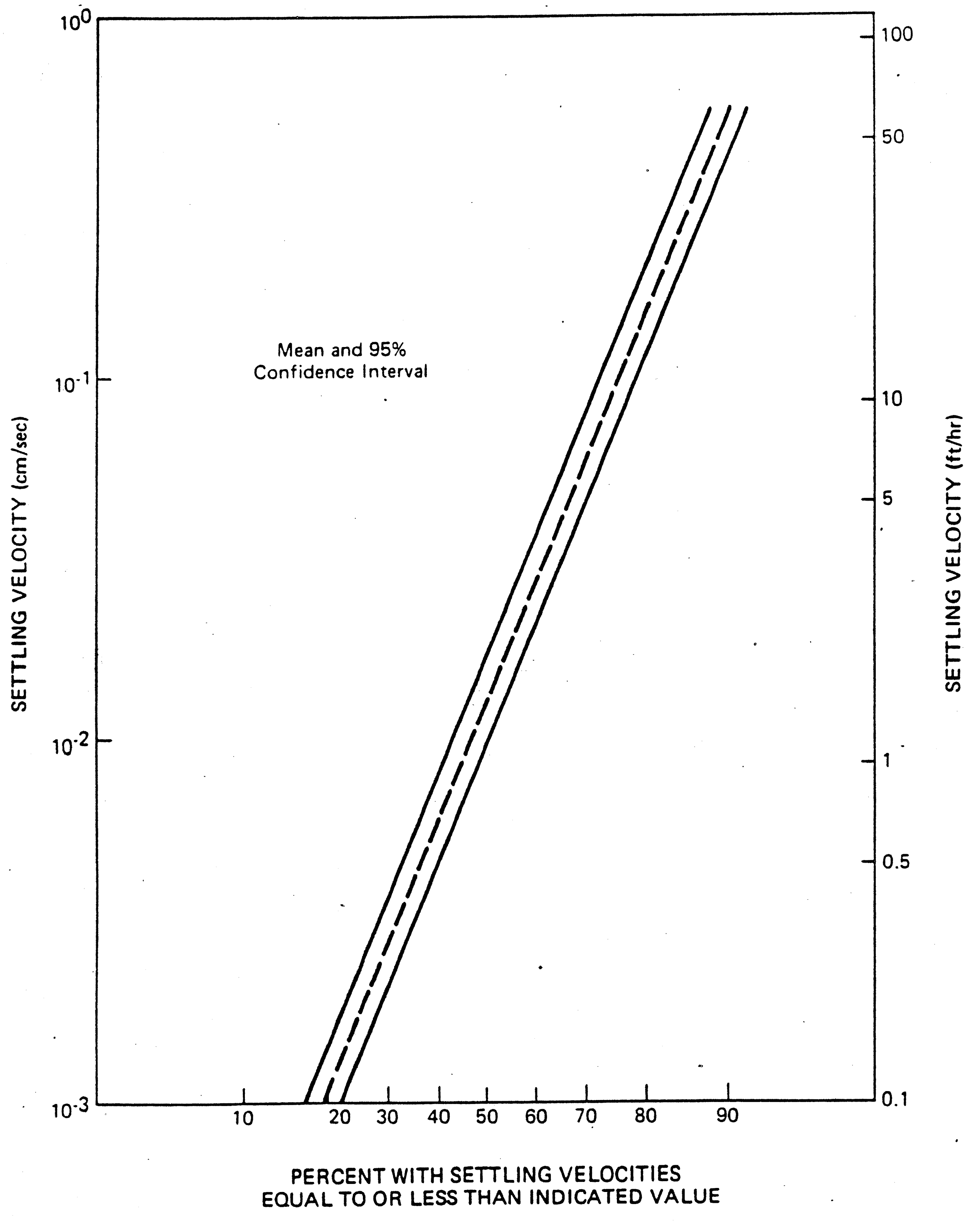
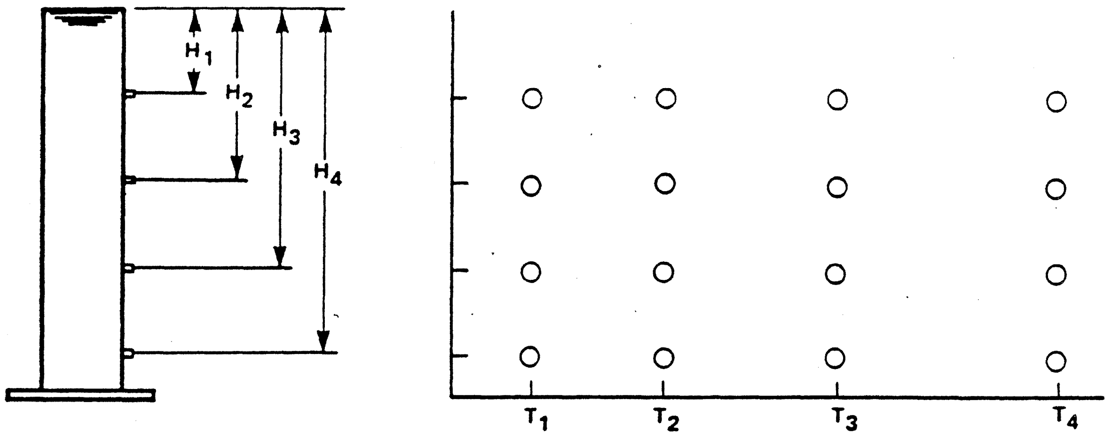


Figure A-5. Probability distribution of settling velocities in urban runoff—typical based on pooled data



SETTLING COLUMN

ELAPSED TIME TO SAMPLE WITHDRAWAL

O = Data Point - Record % removed based on observed vs. initial concentration

Settling velocity (V_s) for that removal fraction is determined from the corresponding sample depth (h) and time (t)

$$V_s = H/T$$

Observed % removed reflects the fraction with velocities equal or greater than computed V_s

A probability plot of results from all samples describes the distribution of particle settling velocity in the sample

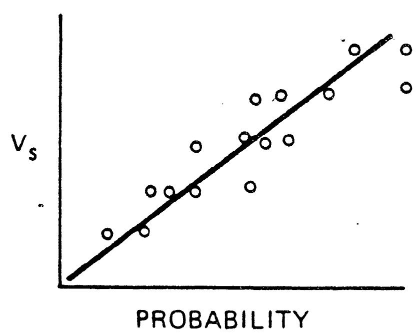


Figure A-6. Estimating settling velocity distributions from settling column tests

Test results are often somewhat erratic because of the sensitivity of analytical tests (especially TSS at low concentrations) and thermal currents and other disturbances in the column. The use of multiple ports and settling times provides data on a range of settling velocities, and provides duplicate measurements for many settling velocities and therefore an opportunity to average out variations inherent in the test procedure.